Service Life Design for Bridges (R19A) Academic Toolbox

May 9, 2018

Contents

Section **Page**

Tables

Figures

Definitions

$(s/kg/m^3)^{0.5}$	square root of seconds per kilogram per cubic meter
ACC	Accelerated Carbonation Test
AAR	Alkali-Aggregate Reactions
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ACR	alkali-carbonate reaction
ASR	alkali-silica reaction
ASTM	American Society for Testing and Materials
CDF	Cumulative Distribution Function
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
in^2/yr	square inch per year
kg/m^3	kilogram per cubic meter
lb/in^3	pound per cubic inch
LRFD	Load and Resistance Factor Design
m^2/s	square meter per second
mm	millimeters
mm^2/yr	square millimeter per year
N/A	Not Applicable
NACE	National Association of Corrosion Engineers
OCP	Open Circuit Potential
PDF	Probability Density Function

PMF	Probability Mass Function
R19A	*Service Life Design for Bridges*
RH	Relative Humidity
SHRP2	Strategic Highway Research Program
SI	International System of Units
w/c ratio	water/cement ratio

1.0 Introduction

1.1 Overview

It is widely recognized that the service life of bridges and their structures can be more economically extended with robust and durable initial design and construction than by future rehabilitation. To address limited future operating budgets, bridge owners are focusing increasingly on life-cycle costs and asset management, rather than just initial construction costs. To reduce life-cycle costs, new major structures are being designed for service lives of 100 years or more; longer than the 75 years typically assumed in bridge design codes. The Izmit Bay Bridge in Turkey, as well as the new New York (Tappan Zee) Bridge and the Abraham Lincoln Bridge across the Ohio River in the United States are examples of three recent major infrastructure projects that share a common design requirement: each must be designed to achieve a 100-year service life for non-replaceable components.

North American structural design codes do not explicitly consider the durability and service life of structures. Prescriptive requirements in these codes result in oversimplified deemed-to-satisfy rules that cannot quantify the service life of a structure in a given environment. Reliable long-term performance of structures in aggressive environments is not achieved and serviceability failures occur earlier than the assumed design life.

To confidently achieve the required service life, engineers must go beyond current structural design codes and assess deterioration mechanisms and mitigation measures specific to each structure and its environment.

For concrete structures, as with modern structural design codes, a rational, reliability-based approach can be used. Analogous to structural design, durability-related loads and resistances are assessed and quantified. For bridges subject to chloride-induced reinforcement corrosion, a durability load is the chloride exposure level and corresponding durability resistances are the concrete permeability and cover thickness. The durability resistances are measured during construction to verify that the required material properties are achieved, just as concrete compressive strength is measured as a means of verifying the structural performance.

1.2 Academic Toolbox

The Federal Highway Administration (FHWA), in partnership with the American Association of State Highway and Transportation Officials (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.

One of the goals of *Service Life Design* is to focus on developing quantitative methods for evaluating the anticipated service life of bridges, in much the same way that the current AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specification addresses application of external loads and sizing members based on material strengths and dimensions.

The objective is to produce performance and design requirements that owners and designers can use and apply on projects.

The first step is to define the term 'service life' that will be used on the project. The following definitions are currently being used in the following structural design codes and guides and can be adapted for the project needs:

- AASHTO LRFD (AASHTO, 2017): Service Life: The period of time in which the bridge is expected to be in operation. This definition is not to be confused with the Design Life, which is the period of time on which the statistical derivation of transient loads is based: 75 years for the current version of AASHTO LRFD.
- CSA Group (CSA, 2014a+b) A23.1-14 and S6-14: Service life is the time during which the structure performs its design function without unforeseen maintenance or repair.
- American Concrete Institute (ACI) 365 (ACI, 2017): Service life is the period after placement during which all the properties exceed the minimum acceptable values when routinely maintained.
- International Federation for Structural Concrete (*fib*) Bulletin 34 - Model Code for Service Life Design (*fib*, 2006): Design Service Life is the assumed period for which a structure or a part of it is to be used for its intended purpose.
- FHWA Bridge Preservation Guide (FHWA, 2018): The service life is the period for which a component, element, or bridge provides the desired function and remains in service with appropriate preservation activities. This definition is not to be confused with the design life, which is the is the period for which a component, element, or bridge is expected to function for its designated purpose when designed, constructed, and maintained as per standards.

The service life of the different components of a bridge structure can be different. For example, nonreplaceable components such as foundations, substructure, and decks can be designed for a 100-year service life. Replaceable components such as bearings and joints can be designed for shorter service life (30 to 50 years for example) and designed to be replaceable. The target service life for each component will be clearly stated, as well as the maintenance operations required to achieve the target service life. The next chapters will present service life design methodologies that can be implemented.

An academic toolbox was developed as part of the implementation activities for *Service Life Design*. The objective of this toolbox is to provide training materials for an instructor in a college- or university-level course or for a professional development course on bridges, concrete materials, or reliability and probability in engineering. The material provided is aimed to be incorporated in whole or in part into the instructor's own teaching materials.

The toolbox is divided into the following chapters:

1. Introduction
2. Probability and Reliability Analysis
3. Service Life Design of Concrete Structures
4. Service Life Design of Steel Structures

A list of references for the instructor or students is provided at the end of each chapter for further study.

1.3 List of References

American Association of State Highway and Transportation Officials (AASHTO). 2017. *LRFD Bridge Design Specifications*. Washington, D.C., 8th Edition, September, 1,780 pp.

American Concrete Institute (ACI). 2017. *Report on Service-Life Prediction* ACI 365.1R-17. Farmington Hills, MI, 49 pp.

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.

CSA Group (CSA). 2014a. *Concrete materials and methods of concrete construction / Test methods and standard practices for concrete*, CSA A23.1-14/A23.2-14, Toronto, 690 pp.

CSA Group (CSA). 2014b. *Canadian Bridge Highway Code*, CSA S6-14, Toronto, 894 pp.

Federal Highway Administration (FHWA). 2018. *Bridge Preservation Guide – Maintaining a Resilient Infrastructure to Preserve Mobility*, Publication No. FHWA-HIF-18-022, 28 pp.

International Federation of Structural Concrete (*fib*). 2006. *Model Code for Service Life Design fib* Bulletin 34, Lausanne, Switzerland, 1st edition, 110 pp.

2.0 Probability and Reliability Analysis

Engineers have always recognized the presence of uncertainty in the analysis and design of engineering systems. To simplify the problem for structural design, older design codes assumed uncertain parameters as deterministic and safety factors were used to consider the uncertainties. Safety factors, however, did not provide any information on how the different parameters of the system influence safety.

Probabilistic analysis provides the required information for an optimum design. Several design guidelines or codes have recently been revised to incorporate probabilistic analysis. For example, the AASHTO LRFD bridge design specification states that "the design provisions of these specifications employ the Load and Resistance Factor Design (LRFD) methodology. The factors have been developed from the theory of reliability based on current statistical knowledge of loads and structural performance." Load factors and resistance factors are based on a reliability analysis to provide a uniform level of reliability across the multiple limit states (serviceability, ultimate, and fatigue). For various limit states, the load factors are determined based on an acceptable probability of the factored loads being exceeded during a specified period.

In this chapter, basic concepts of probability distributions and reliability analysis, required to understand and perform service life design, are reviewed. This chapter is not meant to provide in-depth knowledge on these topics. The reader should refer to the references provided at the end of this section for additional information.

2.1 Probability Distribution

A distribution function indicates how probabilities are distributed over possible values of X. Depending on the type of a variable, discrete or continuous, the probability distribution can be defined as follow:

- Probability Mass Function (PMF) => $Pr(X=x_i)=f(x)$ (for discrete random variables)
- Probability Density Function (PDF) => $Pr(a \leq X \leq b)=\int_a^b f(x)dx$ (for continuous random variables)

Cumulative Distribution Function (CDF) for both the continuous and discrete cases can be shown as: $F(x)=Pr(X \leq x)$.

Several probability distributions play useful roles in engineering as in many other disciplines; beta, normal, and lognormal distributions are briefly introduced in this section as a few examples of such distributions. The normal distribution is used to describe sets of data varying around a mean value; for example, the concrete cover thickness is typically modeled as a normal distribution since it is expected that as-built cover measurements will be higher or lower than the nominal specified cover given that a construction tolerance is allowed. The lognormal distribution is used to capture a long tail on the higher values and to avoid the occurrence of negative values; for example, a lognormal distribution is typically used to model the chloride surface concentration where high values can be considered. The beta distribution has the advantage of being bounded by a lower and higher bound; this distribution is typically used to model critical chloride thresholds as this avoids considering very low or very high threshold values in the model.

2.1.1 Normal or Gaussian Distribution

Normal or Gaussian distribution is one of the most commonly used distributions. The PDF of this distribution can be expressed as Equation 1:

$$f_X(x)=\frac{1}{\sigma_X\sqrt{2\pi}}\exp[-\frac{1}{2}(\frac{x-\mu_X}{\sigma_X})^2] \qquad -\infty<x<\infty \qquad \text{Equation 1}$$

where the mean μ_x and the standard deviation σ_x are the two parameters of the distribution. Figure 1 shows examples of normal probability density functions.

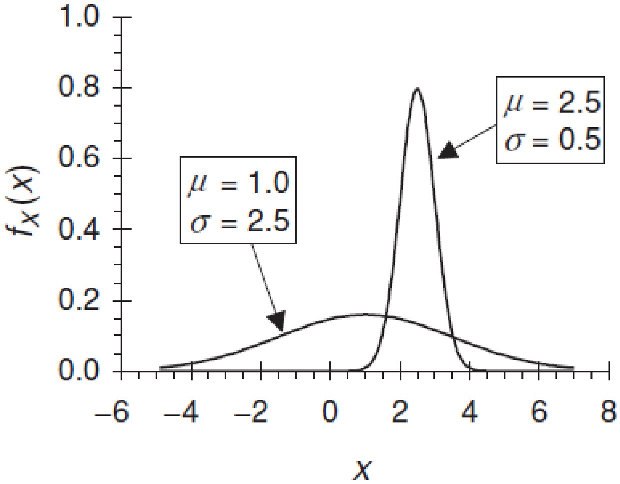


Figure 1: Standard normal density function (Kottegoda and Rosso, 2008).

The corresponding CDF can be expressed as Equation 2:

$$F_X(x)=\int_{-\infty}^{x}\frac{1}{\sigma_X\sqrt{2\pi}}\exp[-\frac{1}{2}(\frac{x-\mu_X}{\sigma_X})^2]\, d_x \qquad \text{Equation 2}$$

A normal random variable with a mean and standard deviation of μ and σ is denoted as N(μ,σ). Generally, it is not simple to estimate the probability by integrating Equation 2. The problem can be addressed by transforming the original random variable X into a standard normal variable with zero mean and unit standard deviation as shown in Equation 3:

$$S=\frac{X-\mu_x}{\sigma_x} \qquad \text{Equation 3}$$

By replacing the standard normal variable in the PDF formula of normal distribution, $f_S(s)$ can be expressed as Equation 4:

$$f_s(s)=\frac{1}{\sqrt{2\pi}}\exp[-\frac{1}{2}s^2] \quad -\infty<s<\infty \qquad \text{Equation 4}$$

The corresponding CDF of S is defined as Equation 5:

$$F_S(s)=\int_{-\infty}^{s}\frac{1}{\sqrt{2\pi}}\exp[-\frac{1}{2}s^2]d_s \qquad \text{Equation 5}$$

The standard normal distribution is denoted as N(0,1), and its CDF is denoted as:

$$\Phi(s)=F_S(s) \qquad \text{Equation 6}$$

As the normal distribution is perfectly symmetrical, Φ(-s) is equal to 1-Φ(s). The CDF of the standard normal distribution is widely available in tabulated format.

2.1.2 Lognormal Distribution

The distribution of a phenomenon arising from the multiplicative effect of numerous uncorrelated factors tends to be lognormal. One example is the chloride surface concentrations that can be measured on bridge decks.

If X is a positive random variable and Y=ln(X) has a normal distribution $N(\mu_Y,\sigma^2_Y)$, then X has a lognormal distribution where PDF is defined as Equation 7. Examples of lognormal distribution curves can be seen in Figure 2.

$$f_X(x)=\frac{1}{x\sigma_{\ln(x)}\sqrt{2\pi}}\exp\left\{-\frac{1}{2}\left[\frac{\ln(x)-\mu_{\ln(x)}}{\sigma_{\ln(x)}{}^2}\right]^2\right\}, \text{ for } 0\leq x<\infty \qquad \text{Equation 7}$$

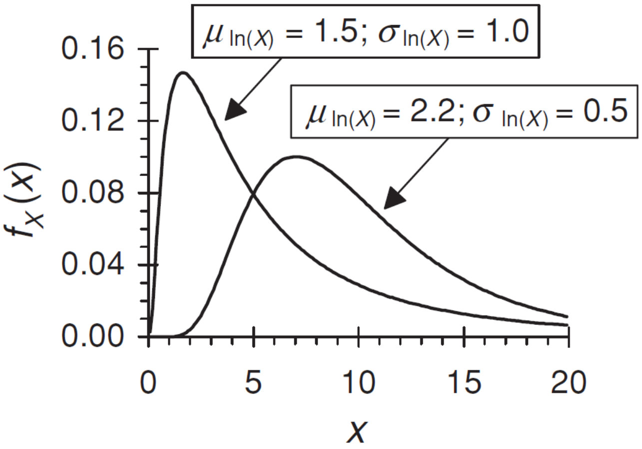


Figure 2: Lognormal density functions (Kottegoda and Rosso, 2008).

2.1.3 Beta Distribution

A random variable taking place on the interval given by 0-1 can be modeled by beta distribution where PDF is defined as Equation 8. Examples of beta distribution curves can be seen in Figure 3.

$$f_x(x/\alpha,\beta)=\frac{1}{B(\alpha,\beta)}x^{\alpha-1}(1-x)^{\beta-1}\text{, for } 0<x<1,\ \alpha>0,\ \beta>0 \qquad \text{Equation 8}$$

where:

$$B(\alpha,\beta)=\int_0^1 x^{\alpha-1}(1-x)^{\beta-1}dx \qquad \text{Equation 9}$$

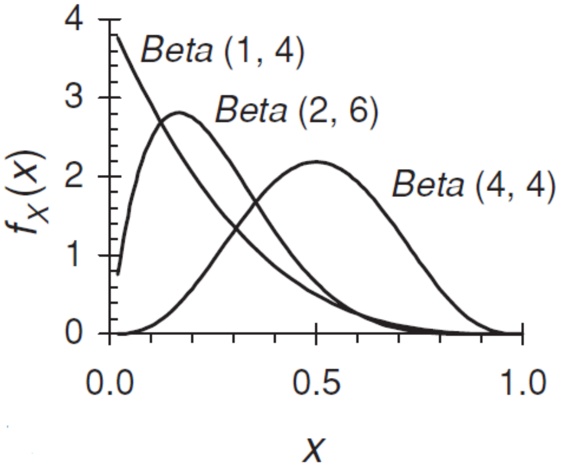


Figure 3: Beta density functions (Kottegoda and Rosso, 2008).

2.2 Reliability

Generally, to provide satisfactory performance, an engineering system should be designed so that the capacity or resistance is greater than the demand. As there is a certain amount of uncertainty, however, satisfactory performance cannot be absolutely ensured. Instead, assurance can only be given in terms of the probability of success in satisfying some performance criteria, which is referred to as reliability. The probability of failure to satisfy some performance criteria is referred to as risk.

2.2.1 Reliability Evaluation

In the simplest case, the limit-state function g(x) (also known as performance function) is the difference between resistance (R) and demand (S, from French word *solicitation*) such that g(x)=R-S. Hence, the probability of failure can be defined as shown in Equation 10.

- $g(x)<0$: Failure
- $g(x)>0$: Safe
- $P_f=P(g(x)\leq 0)$
- $g(x)=0$: The limit-state surface

Equation 10

If R and S are normal independent variables with mean values μ_R and μ_S, respectively, and standard deviations σ_R and σ_S, respectively, g(x) is also a normal random variable that is shown as $N(\mu_R-\mu_S, \sqrt{\sigma_R^2+\sigma_S^2})$. In this case, the probability of failure is defined as Equation 11.

$$P_f = \Phi\left(\frac{0-(\mu_R-\mu_S)}{\sqrt{\sigma_R^2+\sigma_S^2}}\right)=1-\Phi\left(\frac{\mu_R-\mu_S}{\sqrt{\sigma_R^2+\sigma_S^2}}\right) \qquad \text{Equation 11}$$

The reliability index, β, is defined as Equation 12.

$$\beta = \frac{\mu_g}{\sigma_g} = \frac{\mu_R-\mu_S}{\sqrt{\sigma_R^2+\sigma_S^2}} \qquad \text{Equation 12}$$

The probability of failure and the reliability index are directly related, as shown in Equation 13:

$$P_f = \Phi(-\beta) \qquad \text{Equation 13}$$

P_f can also be approximated using the following approximation provided in Equation 14:

$$P_f = 10^{-\beta} \qquad \text{Equation 14}$$

If β is large, P_f will be small, implying that the underlying risk is small. This relationship is visualized in Figure 4, where β is plotted against P_f in the range of typical β-values. The exact relationship is shown by a solid line, and the gross approximation $P_f = 10^{-\beta}$ is shown as a dashed line on Figure 4.

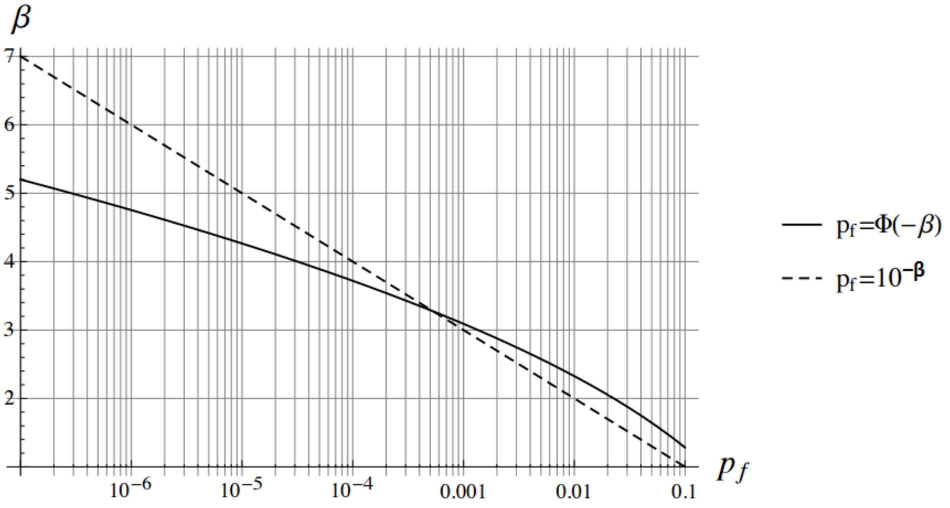


Figure 4: Relationship between β and P$_f$ (Haukaas, 2014).

A general representation of P_f is presented in Equation 15.

$$P_f = \int \dots \int f_x(x_1 x_2 \dots x_n) d_{x_1}\, d_{x_2} \dots d_{x_n} \qquad \text{Equation 15}$$

Where f(x) is the joint PDF for the random variables x_1, x_2, …, x_n and the integration is over the failure region, which is g(x)<0. It is impossible to solve the multi-fold integral in Equation 15 analytically, except for a few special cases. Therefore, numerical methods like the First Order Reliability Method, Second Order Reliability Method, and sampling methods can provide approximate solutions. More information about these methods can be found in the reference list provided at the end of this chapter. Several sampling schemes are available to estimate the failure probability. The simplest and most popular approach is called the Monte Carlo simulation, which is discussed in the following section.

2.2.2 Monte Carlo Simulation Technique

The Monte Carlo simulation uses random samples of parameters or inputs to explore the behavior of a complex system or process. It calculates results over and over, each time using a different set of random values from the probability distributions. A Monte Carlo simulation could involve thousands of recalculations before it is complete, depending on the number of uncertainties and the ranges specified for them. Monte Carlo simulation produces distributions of possible outcome values. Essential elements of its techniques are summarized in Figure 5.

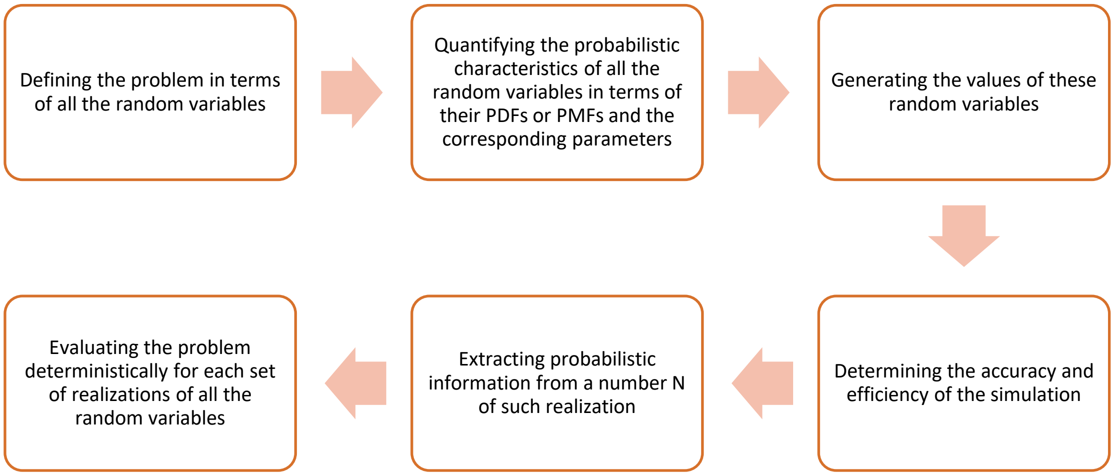


Figure 5: Essential elements of Monte Carlo simulation technique.

If N_f is defined as the number of samples falling into the failure domain or satisfying the condition of $g(x) \leq 0$ and if N is the total number of samples, the probability of failure is defined in Equation 16:

$$P_f = N_f/N \qquad \text{Equation 16}$$

2.3 Exercises

Example 2.3.1

To study chloride-induced corrosion of black reinforcing steel in a bridge deck, a chloride profile was measured from concrete cores selected from the bridge deck. Table 1 shows the measured chloride contents, mean, and variance (μ_{ch} and σ^2_{ch}), at three different depths. Data shows that at each depth, the chloride contents follow a normal distribution. The chloride content required for the depassivation of black reinforcing steel (critical chloride threshold) is assumed to follow a normal distribution with a mean of μ_{crit}=1.00 % (wt/wt cement) and a coefficient of variation of 0.45. If the deck concrete cover is assumed to be exactly 2 inches, what is the reliability index and probability of corrosion initiation?

(Note: Chloride ion contents can be measured at various depth increments of a concrete sample based on the procedure described in ASTM C1152 (ASTM, 2012).)

Table 1: Mean and Variance of Measured Chloride Content at Three Different Levels of Bridge Deck Cores.

Depth (inches)	Chloride Contents Mean μ_{ch} (wt% of cement)	Chloride Contents Variance σ^2_{ch}
1	0.42	0.15
2	0.31	0.12
3	0.28	0.09

Answer:

This problem is solved from a reliability evaluation as outlined in Section 2.2.1. As the cover is 2 inches, the mean and variance of the chloride content (the demand, S) at the reinforcing steel level is:

$\mu_{ch} = 0.31$ (wt% of cement)

$\sigma^2_{ch} = 0.12$

The mean and variance of the critical chloride threshold (the resistance, R) are:

$\mu_{crit} = 1.00$ (wt% of cement)

$\sigma^2_{crit} = [COV \times \mu_{crit}]^2 = (0.45 \times 1.00)^2 = 0.20$

Hence, the mean and variance of the limit-state function, g(x), are:

$\mu_Z = \mu_{crit} - \mu_{ch} = 1.00 - 0.31 = 0.69$

$\sigma^2_Z = \sigma^2_{crit} + \sigma^2_{ch} = 0.12 + 0.20 = 0.32$

The reliability index is determined as shown in Equation 12:

$$\beta = \frac{\mu_Z}{\sigma_Z} = \frac{0.69}{\sqrt{0.32}} = 1.22$$

The probability of failure is determined through Equation 13 by looking up the probability in a standard normal table based on the determined β-value:

$P_f = \Phi(-1.22) = 0.1112$

Or, 11% probability of corrosion initiation.

Example 2.3.2

The chloride content (the demand, S) at the reinforcing steel level of a concrete footing is estimated to follow a normal distribution with statistical properties as follows:

$\mu_{ch} = 0.45$ (wt% of cement)

$\sigma^2_{ch} = 0.4$

According to Section 2.1.1, this can also be written as N(0.45,0.4) because the variables are normal random variables. In the same way, the critical chloride threshold (the resistance, R) is estimated to be N(0.6,0.15).

What probability of failure (corrosion initiation) is estimated using the Monte Carlo method if R and S are independent?

Answer:

As stated in Section 2.2.2, the Monte Carlo simulation uses random samples of parameters as input for the analysis in question. A Monte Carlo simulation typically requires numerous recalculations using different random parameters to make the analysis accurate. However, for convenience, only 10 random sample variates representing the resistance (R) and another 10 random sample variates representing the demand (S) will be used for this example. Table 2 summarizes the demand and resistance parameters used in this example.

Table 2: Demand and Resistance Parameters.

Parameter		Distribution	Mean, μ	Std. dev, σ
Chloride content	Demand, S	N(0.45,0.4)	0.45	0.4
Critical chloride threshold	Resistance, R	N(0.6,0.15)	0.6	0.15

Table 3 on the next page shows the random numbers selected for the analysis and the calculations of probability of corrosion initiation.

Table 3: Random Numbers Generated for this Example and Calculations of Probability of Corrosion Initiation.

Random Number z_i	Resistance r_i	Random Number z_i	Demand s_i	r>s?(*)
0.9311	0.82	0.4537	0.40	1
0.7163	0.69	0.1827	0.09	1
0.4626	0.59	0.2765	0.21	1
0.7895	0.72	0.6939	0.65	1
0.8184	0.74	0.8189	0.81	0
0.3008	0.52	0.9415	1.08	0
0.3989	0.56	0.4967	0.45	1
0.0563	0.36	0.2097	0.13	1
0.1770	0.46	0.4575	0.41	1
0.2036	0.48	0.4950	0.44	1
(*) 0 = failure, 1 = success				
Number of samples				10
Number of failures				2
P_f				20%

The random numbers in Table 3 have all been selected from a table of random numbers between 0-1. The next step in the Monte Carlo analysis is to transform these values, z_i, into the values of random variables following a given distribution $F_X(x)$. In this example, the demand, S, and the resistance, R, both follow a normal distribution with $N(\mu_S,\sigma_S)$ and $N(\mu_R,\sigma_R)$, respectively, as given in Table 2. Combining Equation 3 and Equation 6 generates:

$$z = F_X(x) = \Phi\left(\frac{x-\mu_x}{\sigma_x}\right)$$

where $F_X(x)$ is the CDF of the random variable considered (Equation 2). By isolating x, the following expression is obtained:

$$x = \mu_x + \sigma_x \, \Phi^{-1}(z)$$

This transformation is called the inverse transformation method and the random variable is generally given by:

$$x_i = F_{X_i}^{-1}(z_i)$$

where $F_{X_i}^{-1}$ is the inverse of the CDF of the random variable X_i, which in this case is either the demand, S, or the resistance, R, which are both normally distributed.

For this example, $z_1 = 0.9311$ is one of the random numbers chosen for the resistance, R, as seen in Table 2. Hereby, the random variable parameter can be determined from the equation above, as:

$$r_1 = \mu_R + \sigma_R\, \Phi^{-1}(z_1) = 0.6 + 0.15 \cdot 1.49 = 0.82$$

where the factor $\Phi^{-1}(z_1) = \Phi^{-1}(0.9311) = +1.49$ has been obtained from a standard normal distribution table. Similar calculations are performed for the remaining r_i ($i = 2 \ldots 10$). The normal random variable $R = (r_1, r_2, \ldots, r_{10})$ then defines the chloride resistance. As for the demand, S, the calculations are similar to those for the resistance; however, in the above equation, the mean value is replaced with $\mu_S = 0.45$ and the standard deviation with $\sigma_S = 0.4$ according to Table 2. The normal random variable $S = (s_1, s_2, \ldots, s_{10})$ then defines the chloride content.

The probability of corrosion initiation is equal to the number of failures ($r<s$) to the total number of samples (Equation 16). In this example, that means the probability of corrosion initiation is 20 percent. As stated previously, however, many more random numbers should be included to get a more accurate probability.

2.4 List of References

ASTM International (ASTM). 2012. *Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete.* C1152. West Conshohocken, PA.

Haukaas, T. 2014. *Reliability Analysis*. February http://www.inrisk.ubc.ca/probabilities/.

Kottegoda, N.T., and Renzo R. 2008. *Applied Statistics for Civil and Environmental Engineers*, Second Edition. Wiley-Blackwell, 718 pp.

3.0 Service Life Design of Concrete Structures

This section presents a brief description of common deterioration mechanisms of concrete structures and the strategies that can be used to mitigate the deterioration.

3.1 Sulfate Attack

Deleterious reactions occur when Portland cement with a moderate-to-high tricalcium aluminate (C_3A) content is used in concrete in contact with sulfate bearing soil or groundwater. Effects include extensive cracking, expansion, loss of bond between the cement paste and aggregates, and alteration of the paste composition that will cause an overall loss of the concrete strength (Figure 6). Another form of sulfate attack is the thaumasite form of sulfate attack that can occur under certain conditions of low temperatures and exposure to sulfates and carbonates. A thaumasite form of sulfate attack can result in significant degradation of the concrete paste and consequent loss of concrete strength and durability.

Figure 6: Sulfate attack of concrete piers in contact with sulfate bearing water (FHWA, 2010).

Mitigation methods include using Portland cement with a low tricalcium aluminate content, providing a concrete with low permeability and a low water-cement ratio, and the use of supplementary cementitious materials.

ASTM has cements types that are known to provide resistance against sulfates such as cement Type II (Moderate Sulfate Resistance) and Type V (High Sulfate Resistance). ACI 318 (ACI, 2014) provides guidance on mitigation methods to use to prevent damages from sulfate attack.

3.2 Delayed Ettringite Formation

Delayed ettringite formation (DEF) is a form of internal sulfate attack that can occur in concrete cured at elevated temperatures such as in precast units or mass concrete placements. It can be affected by concrete composition, curing conditions, and exposure conditions. Risk of DEF is avoided through proper temperature control during concrete placement and curing.

Maximum temperatures allowed during curing to mitigate risks of DEF are typically 150 to 160 degrees Fahrenheit (^{o}F).

Figure 7 shows a concrete wall damaged by DEF. Because the formation of ettringite causes a volume expansion, extensive concrete cracking is often a result of DEF.

Figure 7: Delayed ettringite formation in concrete wall (*Characterization of Delayed Ettringite Formation in Maryland Bridges*, dissertation by Micah Shalom, 2007).

3.3 Alkali-Aggregate Reactions

Alkali-aggregate reaction (AAR) is a chemical reaction between certain minerals, such as reactive, non-crystalline silica present in some aggregates and the alkalis present in the concrete that can cause expansion and cracking of the concrete. Figure 8 shows a concrete barrier attacked by AAR. AAR-induced cracking in unrestrained concrete typically has a random orientation referred to as map cracking. AAR can be subdivided into two types of reactions: alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR). ASR is the most common type and is due to a chemical reaction between alkalis present in the concrete and certain siliceous rocks or minerals present in aggregates. ACR is less common and is due to a chemical reaction between alkalis present in the concrete and certain carbonate rocks.

Figure 8: Concrete barrier attacked by AAR (FHWA, 2010).

Measures to minimize the risk of AAR include the appropriate selection of non-reactive aggregates as determined by standard test methods (such as ASTM C1260 (ASTM, 2014), ASTM C1293 (ASTM, 2018), ASTM C295 (ASTM, 2012a)). The use of low alkali cement (less than 0.6 percent equivalent sodium oxide [Na_2O]) and supplementary cementitious materials can also mitigate AAR if reactive aggregate must be used.

AASHTO R 80 (AASHTO, 2017) provides guidance on test methods to implement that can mitigate risks related to AAR.

3.4 Freeze-Thaw

In a cold environment, all parts of the concrete structure above the design frost depth are typically exposed to freeze-thaw cycles. Freeze-thaw cycles can cause deterioration when the concrete is critically saturated: the water in the pores freezes to ice and expands. Typical signs of freeze-thaw damage include cracking, spalling, and scaling of the concrete surface and exposure of the aggregates. Figure 9 shows a concrete pavement exposed to internal cracking due to freeze-thaw.

Figure 9: Internal cracking of concrete pavement exposed to freeze-thaw (FHWA, 2015).

The frost resistance of concrete depends on the mix design and concrete permeability: concrete with high water content and high water-to-cement ratio is less resistant. Damage from freezing and thawing can be avoided by using freeze-thaw resistant aggregates and providing air entrainment in the concrete. Typically, a fresh concrete air content of 6 percent, plus or minus 2, is specified to mitigate the effects of freeze-thaw action. This percentage can vary depending on the size of the aggregate.

Another factor that affects the freeze-thaw durability is the air-void-spacing factor that indicates the distance between the air voids where water can freeze and expand freely. The hardened air-void system in the concrete paste can be tested in accordance with ASTM C457 (ASTM, 2017), which provides guidance on acceptance test values.

The presence of de-icing salts can lower the freeze-thaw resistance of the concrete. 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 as seen in Figure 10.

Scaling can be mitigated by appropriate mix design, air entrainment, and proper methods of curing and finishing. Scaling tests can also be used to test the concrete performance to scaling such as the ASTM C672 (ASTM, 2012b).

Figure 10: Scaling of concrete pavement exposed to freeze-thaw and de-icing salts (FHWA, 2015).

Left: The far left shows the intact surface and the right part shows scaling of the surface. Right: Close-up of scaling of the surface.

3.5 Ice Abrasion

For concrete elements located in water that freezes, ice flow (floes) can impact and rub against concrete components in the river, abrading the surface. Ice sheets forming around concrete components can rise and fall with water fluctuations, abrading the surface. These effects can cause a loss of concrete cover over time.

No design standards exist regarding abrasion loss because of ice for bridge structures. One option to consider consists of providing a sacrificial cover thickness based on observations from other structures located in a similar environment.

3.6 Corrosion

For concrete structures, a two-phase service life model can be used to represent the development of corrosion over time as illustrated in Figure 11. The concept of the two-phase model was first proposed by Tuutti (1982) and later adopted with or without modifications by many researchers. Often, the nominal service life is assumed equal to the corrosion initiation time, which is at the end of the initiation phase and is very conservative as also discussed in Section 3.6.3. The structure does not become obsolete at the end of the initiation period. This definition of the limit state, however, is consistent with the objective of having concrete structures with minimal maintenance requirements over the service life.

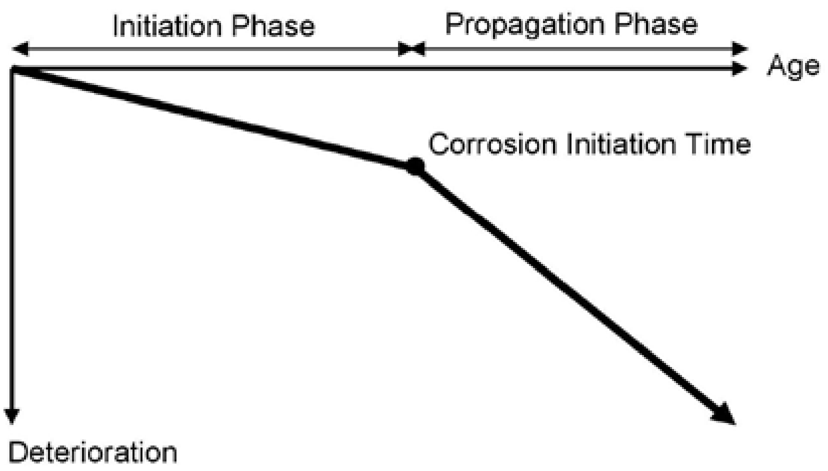


Figure 11: Two-phase modelling approach of corrosion deterioration.

The Initiation Phase

For concrete not attacked by carbonation or chloride ingress (Sections 3.6.1 and 3.6.2), the pH value of the pore solution is typically 12.5-13.5 due to the high content of calcium hydroxide in the cement paste. In such alkaline environments, reinforcing steel is protected from corrosion by a passive layer of iron oxides on the surface of the reinforcement. During the initiation phase, the passive layer is broken down either due to carbonation or chloride ingress. The dissolution of the passive layer and the decrease of the concrete pH promote the initiation of corrosion. The length of time necessary for this process to occur can be highly variable depending on the concrete properties, depth of cover on the reinforcement, and the exposure conditions. During the initiation phase, no noticeable weakening of the material or the function of the structure occurs.

The Propagation Phase

At the start of this phase, the protective barrier on the steel surface is broken down and critical levels of chlorides are reached, so that during the propagation phase an active deterioration develops, and accumulation of damage commences. In many cases, corrosion develops at an increasing rate with time.

Mitigation methods for chloride-induced corrosion include the use of low-permeability concrete, adequate concrete cover, use of corrosion-resistant reinforcing, and proper control of cracking.

3.6.1 Carbonation-Induced Corrosion

Carbonation-induced corrosion is caused by carbon dioxide from air penetrating the concrete and reacting with calcium hydroxide to form calcium carbonate. Concrete carbonation begins at the external surface and progresses inward at a rate that decreases with time. This is a slow and continuous process that lowers the alkalinity of the concrete, which is essential for corrosion protection of steel reinforcement. When the pH value decreases, the steel passivation layer is dissolved and corrosion can occur when moisture and oxygen are present. The reaction of carbon dioxide (CO_2) and calcium hydroxide ($Ca(OH)_2$) only occurs in solution; thus, in very dry concrete, carbonation will be slow. Carbonation-induced corrosion leads to uniform corrosion around the steel reinforcement and usually develops later and at slower rates than chloride-induced corrosion.

Carbonation can be the principal driver of degradation for elevated concrete bridge components not exposed to chlorides. Highest carbonation rates are expected inside where concrete is not directly exposed to rain, such as in hollow cross beams and pylon legs. CO_2 penetrates faster in sheltered concrete compared to concrete exposed to rain. Carbonation ceases in saturated concrete and proceeds very slowly in near-saturated concrete because water-filled concrete pores prevent ingress of carbon dioxide. Although CO_2 can penetrate in dry or low-relative humidity concrete, the corrosion rate in such dry environments is negligible. For low water/cementitious material ratios, the carbonation rate is also very low.

Mitigation methods for carbonation-induced corrosion include low-concrete permeability and adequate concrete cover. American and Canadian experience has shown that for structural elements exposed to the atmosphere, mitigation methods for chloride-induced corrosion will also prevent carbonation-induced corrosion for a similar or longer period of time.

3.6.1.1 Design Parameters for the Mathematical Modelling

Carbonation-induced corrosion is rarely explicitly considered in design codes. For bridge structures, this deterioration mechanism is not critical for most components since chloride-induced corrosion typically prevails. Hence, on most bridge projects, detailed modeling for carbonation is not performed as the service life design will be governed by the requirements derived from chloride-induced corrosion.

The rate of carbonation from the concrete surface depends on the following factors:

- Concrete cover and permeability
- Cement type and content
- Supplementary cementitious materials
- Water/cementitious material ratio
- Curing period of concrete
- Temperature
- Humidity of environment
- Concentration of carbon dioxide

Keeping all other factors equal, carbonation occurs most rapidly when the relative humidity of the concrete is between 50 and 75 percent. Based on the limit-state equation, Equation 17, a full probabilistic design approach was developed during the publicly funded DuraCrete research project for the modelling of carbonation-induced corrosion in uncracked concrete (DuraCrete, 2000). The DuraCrete project was a European project carried out between 1996-1999 as a collaboration between universities and industry. The objective of DuraCrete was to use available knowledge of concrete deterioration through scientifically based modelling of the transport and deterioration mechanisms governing the service life performance of concrete structures. The project resulted in the publication of a design guide for service life. The DuraCrete model for carbonation-induced corrosion is based on Fick's 1st law of diffusion as the prevailing transport mechanism for carbon dioxide within the concrete.

$$g(a,x_c(t)) = a-x_c(t) = a - \sqrt{2\times k_e\times k_c\times(k_t\times R^{-1}_{ACC,0}+\varepsilon_t)\times C_s}\ \sqrt{t}\times W(t) \qquad \text{Equation 17}$$

where:

$$k_e=\left(\frac{1-\left(\frac{RH_{real}}{100}\right)^{f_e}}{1-\left(\frac{RH_{ref}}{100}\right)^{f_e}}\right)^{g_e} \qquad \text{Equation 18}$$

$$k_c:=\left(\frac{t_c}{7}\right)^{b_c} \qquad \text{Equation 19}$$

$$R^{-1}_{ACC,0}:=\left(\frac{x_c}{\tau}\right)^{2} \qquad \text{Equation 20}$$

$$R^{-1}_{NAC,0}:=k_t\cdot R^{-1}_{ACC,0}+\varepsilon_t \qquad \text{Equation 21}$$

$$C_S:=C_{s,atm}+C_{s,emi} \qquad \text{Equation 22}$$

$$W:=\left(\frac{t_0}{t}\right)^{\frac{(p_{SR}\cdot ToW)^{b_w}}{2}}:=\left(\frac{t_0}{t}\right)^{w} \qquad \text{Equation 23}$$

$$ToW:=\frac{\text{days per year with rainfall } h_{Nd}\geq 2.5\text{ mm}}{365} \qquad \text{Equation 24}$$

Although the CO_2 diffusion coefficient for a concrete during service life may be a function of numerous variables, it is assumed that the diffusion coefficient for carbon dioxide through the material is a constant material property. shows a summary of input parameters for the probabilistic models for carbonation-induced corrosion in uncracked concrete included in International Federation for Structural Concrete (*fib)* Bulletin 34 (*fib*, 2006). Further information on each input parameter can be found in this reference. Parameters marked with an asterisk in the table are to be determined specifically for the project.

Table 4: Input Parameters for Full Probabilistic Design Method for Carbonation-Induced Corrosion - Uncracked Concrete.

Parameter	Description	Units	Distribution Function
a^*	Concrete cover	inches mm	Lognormal
k_e	Environmental function	N/A	N/A
RH_{real}^*	Relative humidity (from local weather data)	%	Beta
RH_{ref}	Reference relative humidity	%	Constant
g_e	Environmental function exponent	N/A	Constant
f_e	Environmental function exponent	N/A	Constant
k_c	Execution transfer parameter	N/A	N/A
t_c^*	Time of curing	days	Constant
b_c	Exponent of regression	N/A	Normal
$R^{-1}_{ACC,0}$	Inverse effective carbonation resistance of concrete from Accelerated Carbonation Tests (ACC)	(in^2/yr) / (lb/in^3) (mm^2/yr) / (kg/m^3) (m^2/s) / (kg/m^3)	Lognormal
x_c^*	Measured carbonation depth in the ACC compliance test	inches mm m	N/A
T	Time constant for described test conditions	(s/kg/m^3)$^{0.5}$	Constant
$R^{-1}_{NAC,0}$	Inverse effective carbonation resistance of dry concrete (65 percent RH) determined at a certain point of time on specimens with the normal carbonation test	(in^2/yr) / (lb/in^3) (mm^2/yr) / (kg/m^3) (m^2/s) / (kg/m^3)	N/A
k_t	Regression parameter that considers the influence of test method on the ACC test	N/A	Normal
ε_t	Error term considering inaccuracies that occur conditionally when using the ACC test	(mm^2/yr) / (kg/m^3)	Normal
$C_{s,atm}^*$	CO$_2$ concentration of the atmosphere	lb/in^3 kg/m^3	Normal

Table 4: Input Parameters for Full Probabilistic Design Method for Carbonation-Induced Corrosion - Uncracked Concrete.

Parameter	Description	Units	Distribution Function
$C_{s,emi}$*	Additional CO_2 concentration due to emission sources	lb/in^3	Normal
		kg/m^3	
p_{SR}*	Probability of driving rain	N/A	Constant
ToW*	Time of wetness is number of days per year with rainfall exceeding 2.5 mm (0.1 inch)	days	Constant
		years	
b_w	Exponent of regression	N/A	Normal
t_o	Reference point of time (28 days = 0.0767 years)	years	Constant
t_{SL}	Design service life	years	N/A
β	Target Reliability	N/A	N/A

Notes:
* To be determined specifically for the project
$(s/kg/m^3)^{0.5}$ = square root of seconds per kilogram per cubic meter
ACC = Accelerated Carbonation Test
in^2/yr = square inch per year
kg/m^3 = kilogram per cubic meter
lb/in^3 = pound per cubic inch
m^2/s = square meter per second
mm^2/yr = square millimeter per year
N/A = Not Applicable
RH = Relative Humidity

3.6.2 Chloride-Induced Corrosion

For chloride-induced corrosion, chlorides from the surrounding environment, such as de-icing salts or chloride ions from sea water, penetrate the concrete towards the reinforcement. This penetration is typically considered and analyzed as a pure diffusion process that can then be described by Fick's 2nd law. Once a critical concentration is reached at the surface of the steel, chloride ions disrupt the passive layer around the steel reinforcement and corrosion is initiated.

The critical chloride concentration (or chloride threshold value) cannot be represented by a single value since it depends on several factors, such as the type and amount of cement, the pH of the concrete pore solution, the moisture content, and the presence of other aggressive ions such as sulphate (SO_4^{2-}). Chloride threshold values reported in literature show a large scatter; values from 0.04 to 8.34 percent by mass of cementitious materials have been reported (Angst et. al., 2009).

The *fib* Model Code suggests a mean value of 0.6 percent by mass of cementitious materials and is based on uncoated steel reinforcement. The variability of this parameter is considered by using a beta distribution with a standard deviation of 0.15, a lower bound of 0.2, and an upper bound of 2.0 (*fib*, 2006).

3.6.2.1 Chloride-Induced Corrosion Modelling

One established service life design methodology that is built on a broad base of experience and that resides in the public domain is the *Model Code for Service Life Design* (*fib*, 2006). This methodology is referenced in the *Model Code for Concrete Structures* (*fib,* 2010) and the ISO 16204 *Service Life Design of Concrete Structures* (ISO, 2012). The methodology was developed through the DuraCrete research project and was validated by a consortium of universities and consulting firms.

Three different design strategies for concrete structures are typically adopted in accordance with the approach of the *Model Code for Service Life Design* (*fib,* 2006):

1. Strategy A: Avoid the potential degradation mechanism.
2. Strategy B: Apply protective measures that are deemed-to-satisfy the durability requirements.
3. Strategy C: Select material composition and structural detailing to resist, for the required period, the potential degradation mechanism based on a full probabilistic approach.

Examples of Strategy A are selection of non-reactive or inert materials, for example, non-reactive aggregates and stainless-steel reinforcement, or the use of protection systems to separate the concrete element from the aggressive media.

Strategy B consists of application of codes or standards that have been developed to provide satisfactory performance. This would include properties such as adequate air entrainment to provide resistance to freezing and thawing, or the use of sulfate-resistant cements and supplementary cementing materials to provide resistance to potentially reactive aggregates or sulfate bearing soil or groundwater.

In contrast to Strategies A and B, Strategy C allows the deterioration; however, only to the defined service-life-limit state. This can be achieved with a performance-based service life approach using deterioration modelling. An outline of this performance-based service life procedure for Strategy C is as follows:

- Define the performance and service life criteria, including the limit states; for example, 100-year service life for non-replaceable components.
- Define the environmental conditions to be expected.

- Apply realistic modelling of the deterioration process; that is, environmental exposure and the material resistance. Provide concrete properties and design details that will result in the limit states not being exceeded within the required service life, such as cover thickness and concrete quality that is adapted to the local or micro-environment as identified for each structural member of the structure.
- Based on the performance criteria, perform compliance tests for prequalification and quality control purposes; for example, chloride diffusion/migration testing in the case of chloride-induced reinforcement corrosion.

3.6.2.2 Definition of Exposure Zones and Degradation Mechanisms

The environmental exposure of the different structural members and their locations should be identified and quantified before service life design. The portion of a member subjected to a given exposure condition is referred to as an exposure zone. The following exposure zones are usually assumed for bridge members:

- Splash
- De-icing Salts Spray
- Buried
- Submerged
- Atmospheric

Splash refers typically to areas at water level (river, seawater, etc.). De-icing salts spray refers to areas exposed to de-icing salts. Buried and submerged are for areas permanently buried in soil or submerged in water. Atmospheric refers to areas not included previously that are typically exposed to air. A color code can be used to display the exposure zones. An example is presented in Figure 12.

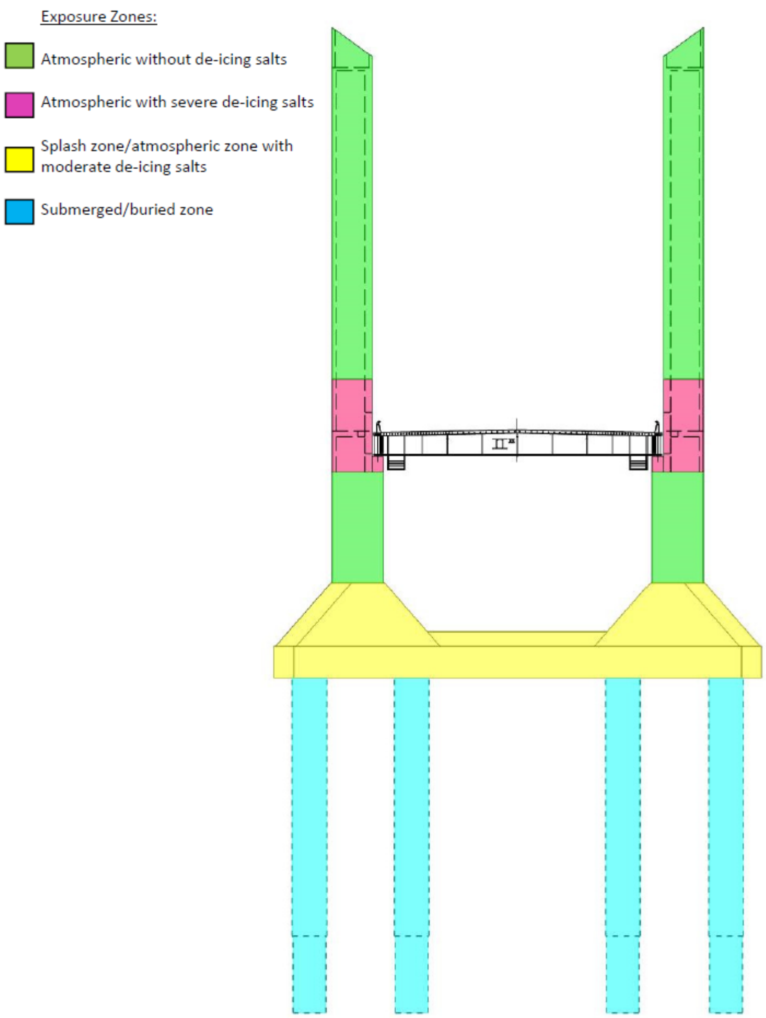


Figure 12: Example of exposure zones for a tower located in water.

3.6.2.3 Selection of the Limit State

The limit states vary based on the project requirements. For example, a limit state can be corrosion initiation with a confidence level of 90 percent that corrosion will not be initiated within the targeted service life. This corresponds to a reliability index of 1.3 and is consistent with guidance provided in the *Model Code for Service Life Design* (*fib*, 2006).

3.6.2.4 Determination of the Design Parameters Required through the Mathematical Modelling

The time- to- corrosion modeling can follow the methodology described in the *Model Code for Service Life Design* (*fib*, 2006). This model is based on the diffusion equation (Fick's 2nd law). The concrete is assumed to be a homogenous semi-infinite material with a constant diffusion coefficient and surface concentration. The solution to the diffusion equation is written as follows:

$$C(x,t)=C_0+\left(C_{S,\Delta x}-C_0\right)\left(1\text{-erf}\left[\frac{x-\Delta x}{2\sqrt{D_{app,C}\cdot t}}\right]\right) \qquad \text{Equation 25}$$

where:

$$D_{app,C}=k_e\cdot D_{RCM,0}\cdot k_t\cdot A(t) \qquad \text{Equation 26}$$

$$k_e=\exp\left(b_e\left(\frac{1}{T_{ref}}-\frac{1}{T_{real}}\right)\right) \qquad \text{Equation 27}$$

$$A(t)=\left(\frac{t_0}{t}\right)^a \qquad \text{Equation 28}$$

where:

- $C(x,t)$ denotes the chloride concentration at the time t and at the distance x from the surface;
- $C_{S,\Delta x}$ denotes the chloride surface concentration;
- C_0 is the initial chloride concentration;
- $D_{app,C}$ is the apparent coefficient of chloride diffusion through concrete;
- Δx is the depth of the convection zone (transfer function);
- erf is the error function;
- $D_{RCM,0}$ is the chloride migration coefficient;
- k_t is a transfer parameter;
- b_e is a regression variable;
- T_{ref} is the standard test temperature;
- T_{real} is the temperature of the structural element or the ambient air; and,
- t_0 is the reference point of time and a is the age factor.

The chloride migration coefficient depends on several factors such as cement type and concrete water/cement (w/c) ratio. The lower the w/c ratio, the lower the chloride migration coefficient. If no test data are available, the *Model Code for Service Life Design* (*fib*, 2006) recommends using a 28-day mean value varying from 8.9×10^{-12} m^2/s (for w/c=0.40) to 25.00×10^{-12} m^2/s (for w/c=0.60) for Portland cement Type I with a 28-day strength of a minimum of 6,000 pounds per square inch.

Corrosion is initiated when the chloride concentration at the surface of the reinforcement exceeds the critical chloride concentration. The limit state function, which is defined as less than or equal to zero if corrosion initiation occurs, can be written as:

$$g(z,t)=C_{crit}-C(x,t) \qquad \text{Equation 29}$$

where x denotes the cover thickness, C_{crit} is the chloride threshold of the reinforcement, and z is the vector of stochastic variables, such as, the concrete cover thickness, surface concentration, diffusion coefficient or other variables The *Model Code for Service Life Design* (*fib*, 2006) contains guidance regarding the choice of input parameters. More information about input parameters can also be found on the SHRP2 *Service Life Design* (R19A) website at:

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

The spreadsheet, *Service Life Design-Full Probabilistic* Tools, is provided on the SHRP2 webpage and is a tool developed to solve the equations following the full-probabilistic design method detailed in the *Model Code for Service Life Design* (*fib,* 2006), for chloride-induced corrosion.

The method uses a Monte Carlo approach to solve Equation 25 repeatedly, by varying each of the variables in the equation according to a probabilistic distribution of values. Each variable is defined by a mean value, a standard deviation, and a distribution type (such as normal, lognormal, and beta). Each individual calculation solution either passes or fails the comparison of chloride concentrations. The total number of times that the critical chloride content is exceeded is compared to the number of trials, and results in a probability of failure and reliability index. The reliability index recommended by the *Model Code for Service Life Design* (*fib*, 2006) is 1.3. Other reliability indices can be considered.

3.6.2.5 Chloride profiles

A more visual understanding of the error function solution presented in Section 3.6.2.4 can be found by studying so-called chloride profiles. A chloride profile shows the chloride content in concrete as a function of the distance from the surface exposed to chlorides. To create a chloride profile, a concrete core sample is cut into several thin slices and the chloride content of each slice is measured. The total length of the core should cover the distance from the surface of the concrete until the point where the original chloride content (typically close to 0) is found. A typical chloride profile is shown in Figure 13. This chloride profile shows actual chloride measurements of the Danish Farø Bridge at a concrete age of t=9 years (based on Stoltzner and Sørensen, 1994).

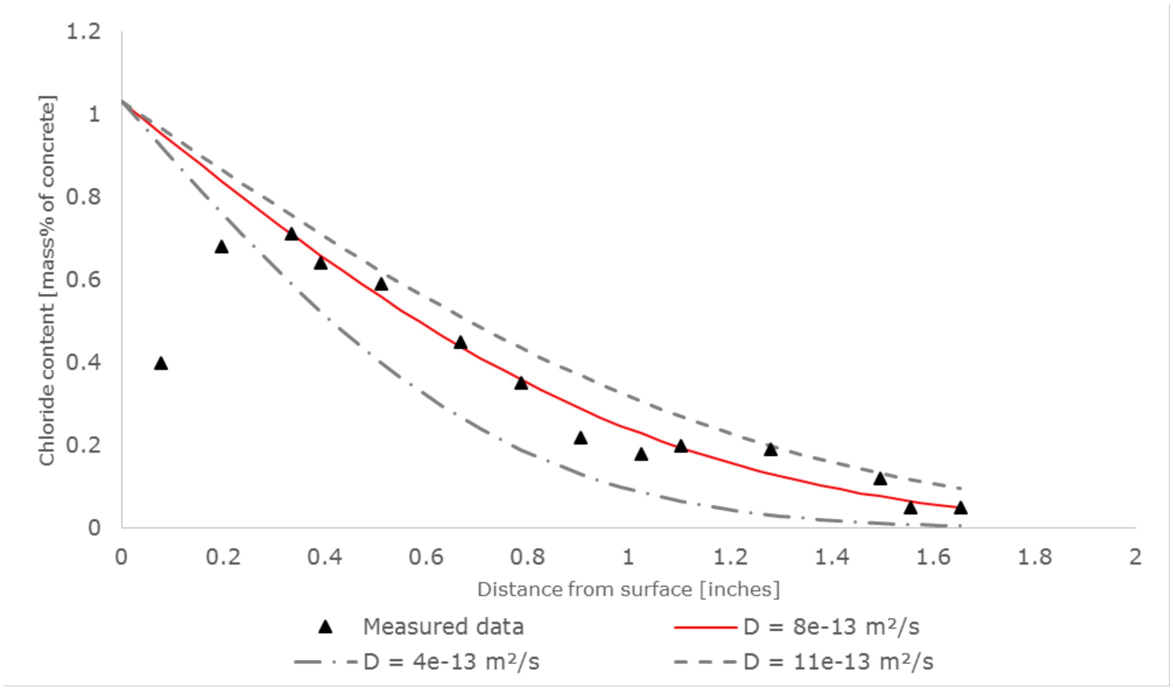


Figure 13: Example of chloride profile from the Danish Farø Bridge at t=9 years.

Overall, the chloride content decreases with the distance from the chloride-exposed surface and the chloride concentration approaches the initial chloride content ($C(x,t=0) = C_0$) asymptotically. Typically, chloride profiles have a local maximum only a small distance into the concrete (as is also seen in Figure 13), which is due to several factors such as leaching of chlorides at the concrete surface, carbonation, or other means. This local dip is typically not considered in the analysis of the profile and the chloride content $C_{s,\Delta x}$ in Equation 25 is the chloride surface content at the depth Δx which corresponds to the depth of up to which the chloride diffusion deviates from Fick's 2nd law.

The error function solution presented in Equation 25 is the solution to Fick's 2nd law, which can be used to describe chloride diffusion into concrete. Therefore, the error function solution can be fitted to the chloride profile measurements by adjusting the two unknown variables in the equation: the chloride content at the concrete surface ($C_{s,\Delta x}$), and the apparent coefficient of chloride diffusion ($D_{app,C}$). The chloride concentration at the surface of the concrete can typically be approximated from the chloride profile. It is important to note that it does not necessarily equal the chloride content of the surrounding substance such as sea water, for example, but typically takes on a higher value because chlorides accumulate on the surface.

For the chloride profile in Figure 13, an initial chloride content of C_0=0 and a surface chloride content of C_s=1.03 mass% of concrete have been assumed. Using these constants, a chloride diffusion coefficient of $D_{app,C}$ = 8.0 x 10^{-13} m^2/s makes the error function solution fit the measurements in Figure 13 as best as possible. For comparison, curves obtained by using $D_{app,C}$ = 4.0 x 10^{-13} m^2/s and $D_{app,C}$ = 11.0 x 10^{-13} m^2/s have also been shown, but it is obviously $D_{app,C}$ = 8.0 x 10^{-13} m^2/s that makes the best fit to the measurement points. An Excel spreadsheet tool that performs a regression analysis to determine the best fit can also be found on the SHRP2 *Service Life Design* (R19A) website at:

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

3.6.3 Propagation of corrosion

The propagation phase follows the initiation phase where de-passivation of the reinforcement takes place (typically either due to carbonation [Section 3.6] or ingress of chlorides [Section 3.6.2]). During the propagation phase, corrosion of reinforcement takes place and a corrosion product is formed. In general, less research has been carried out on the propagation phase compared with the initiation phase because the service life of reinforced concrete structures is often entirely linked to the initiation phase. Some researchers claim, however, that this definition strongly underestimates the actual service life of the concrete structure and that the propagation phase also needs to be included when defining service life. Regardless, to completely understand corrosion of concrete structures, it is necessary to also understand and study the propagation phase, which is often described from the following limit states: crack formation, spalling of concrete cover, and collapse of structure.

The propagation phase can be described from electrochemistry. Corrosion is an electrochemical reaction occurring at anode and cathode. At the anode, oxidation of the reinforcing steel takes place ($Fe \rightarrow Fe^{++} + 2e^-$) and the electrons left from this reaction wander through the reinforcing steel to the cathode where they are used for the reduction reaction. During reduction, the electrons react with water and oxygen to form hydroxide ($O_2 + H_2O + 4e^- \rightarrow 4OH^-$). The result of the oxidation and reduction reactions is the formation of ferrous iron and hydroxide ions that react to form the corrosion product. Corrosion can only occur if water and oxygen are present. All corrosion products are iron oxides or hydroxides but the exact chemical constitution of the corrosion product depends on exactly how much water and oxygen are present (Matthews, 2015).

The volume of the corrosion product depends on its chemical composition; however, some of the most typical corrosion products (such as ferric hydroxide) have a volume more than four times higher than the steel consumed. Therefore, corrosion is often associated with cracking and spalling of the concrete because the concrete cover cannot resist the build-up of internal stress caused by this volume expansion such as seen in Figure 14.

Figure 14: Corrosion of the underside of an existing bridge deck (FHWA, 2014).

An important deviation from this occurs in environments with a low oxygen content (typically under water [Figure 12]). Here, the volume of the corrosion product is considerably less and the corrosion product is almost fluid. This makes it possible for the corrosion product to sometimes expand into the pore system of the concrete without causing cracking and thereby leaving any signs of corrosion. This should be kept in mind when assessing if a structure is attacked by corrosion (Matthews, 2015).

If depassivation of the reinforcement is due to chloride ingress, the corrosion is typically localized and can be rather deep “pitting”. If the corrosion is due to carbonation of the concrete, it may be more generalized and uniform. This is because pitting is caused by chloride ions reaching the surface of the reinforcement, which can be very local. During carbonation, an entire carbonation front typically reaches the reinforcement surface. When corrosion is localized, the anode and cathode are separated, and, in principle, it is therefore possible to measure a current through the reinforcing steel, which gives an indication of the corrosion rate. For generalized corrosion, however, the sites of anode and cathode reaction are closely adjacent and therefore no current can be measured.

The corrosion rate is typically measured in inches/year and describes how fast the corrosion process occurs and thus determines the length of the propagation period. Researchers disagree if cracks influence the corrosion rate. Although some researchers claim that cracks accelerate both the initiation phase and the propagation phase, others claim that the propagation phase is not influenced (Matthews, 2015). There is, however, agreement that several other factors relating to the concrete and its environment, such as cover depth, concrete quality, oxygen level, and moisture content, have an impact on the corrosion rate. The corrosion rate is reduced when the cover depth is great, the concrete quality is good (low w/c ratio), the amount of oxygen is low, and the corrosion environment is either very dry (high resistivity) or very wet (prevents diffusion of oxygen) (Matthews, 2015). Furthermore, the corrosion rate is also influenced by electrochemical processes, and factors such as electrochemical potential, conductivity, and pH.

Several electrochemical and non-destructive techniques are available for monitoring and measuring corrosion and corrosion rates of reinforcing steel in concrete; however, no agreement exists on which method is most accurate. To obtain maximum information about the corrosion state of reinforcement, a combination of measuring techniques is recommended (Song and Saraswathy, 2007). The most commonly used method is to establish a potential map of a structure, as described in ASTM C876 *Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete* (ASTM, 2015). The method is typically known as open circuit potential (OCP) measurement. This method measures the difference in electrochemical potential between a sensor placed on a non-corroded part of the reinforcement bar and a reference electrode placed at the surface of the concrete above the same rebar. When reinforcement is protected by a passive film (Section 3.6.2), the electrochemical potential of the steel is relatively high, whereas a difference in the electrochemical potential takes place when de-passivation occurs. OCP measures this difference to define areas with corrosion, but the methodology cannot be used to determine the corrosion rate. Other methods, such as Linear Polarization Resistance Measurement, must be used for this purpose. This procedure uses measurements of polarization resistance to determine the corrosion rate. See references below, including Song and Saraswathy, for more information about the details behind the measurements.

Many different assumptions on the total propagation period have been postulated. Life-365^{TM} assumes a total propagation period of six years, which is increased to 20 years if epoxy-coated reinforcements are used (Life-365^{TM} , 2014). On the other hand, the United States Navy assumes a typical propagation period of 15 years (UFGS, 2012). It is rather oversimplified to assume that a single value can describe the propagation period for various concrete structures because the many factors influencing the propagation of corrosion differ substantially from concrete structure to concrete structure. As an example, concrete structures submerged into water have a much longer propagation period that structures in splash or tidal zones.

3.7 Project Specifications Documents

Using the output of the chloride-induced corrosion modelling and mitigation methods used for other deterioration mechanisms, the designer summarizes the materials' properties that are required for each component, considering their governing exposure zone. This step also includes producing the concrete specifications where requirements for quality control are detailed. Requirements for the type of tests, test limits, and test frequency should be detailed. It is recommended that a specialist in concrete materials prepare the concrete specifications to determine the final requirements for the concrete properties and quality.

3.8 Construction and Quality Control

The quality control and quality assurance for concrete structures will consist of two phases:

Prequalification Phase

Properties of the concrete mix constituents are reviewed (aggregates, cementitious materials, admixtures, mix designs) to verify that they meet the requirements of the project. A series of laboratory mixtures (trial batches) is completed and appropriate testing is done to demonstrate that all requirements are met.

Production and Construction Phase

During construction, monitoring of the key properties (such as strength, plastic air content, and chloride migration coefficient) is conducted by testing samples obtained from the production concrete. The tests and their frequency are specified in the project specifications.

3.9 Exercises

Example 3.9.1

Calculate the probability of failure (time to chloride-induced corrosion initiation) for t = 25, 50, 75, 100 years using a full probabilistic approach with the input value, according to Table 5.

Table 5: Input Parameters for the Chloride Ingress Mathematical Tool.

Parameter	Value		Unit	Type of Statistical Distribution
	Mean	Standard deviation		
Cover	60	12	mm	Normal
$C_{S,\Delta x}$	2	1	wt%/c	Lognormal
C_0	0.1		wt%/c	Deterministic

Table 5: Input Parameters for the Chloride Ingress Mathematical Tool.

Parameter	Value		Unit	Type of Statistical Distribution
	Mean	Standard deviation		
C_{crit}	0.6	0.15	wt%/c	Beta (lower limit=0.2 and upper limit=2)
$D_{RCM,0}$	378	75.6	mm²/year	Normal
a	0.6	0.15	-	Beta (lower limit=0 and upper limit=1)
Δx	8.9	5.6	mm	Beta (lower limit=0 and upper limit=50)
b_e	4800	700	Kelvin	Normal
T_{real}	283.65	5.2	Kelvin	Normal
T_{ref}	293		Kelvin	Deterministic
t_0	0.0767		Years	Deterministic
t	25, 50, 75, 100		years	Deterministic

Answer: A second order reliability method was used to develop the following graph (Figure 15) showing the probability of failure over time.

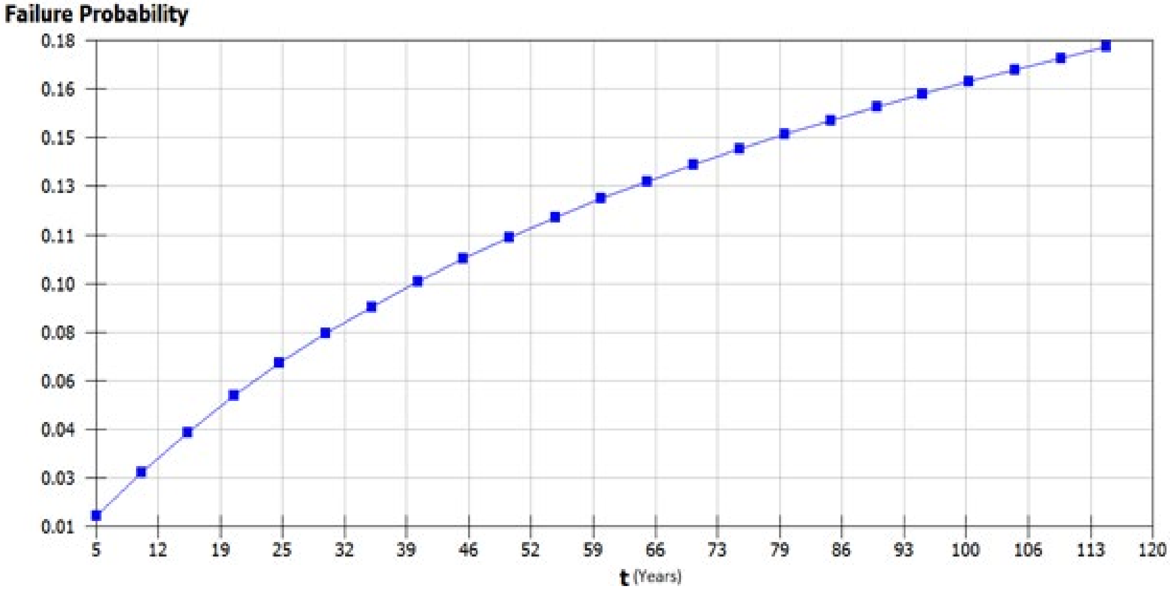


Figure 15: Probability of failure versus time.

Example 3.9.2

Calculate the probability of failure (time to chloride-induced corrosion initiation) for t = 25, 50, 75, 100 years using a full probabilistic approach with the input value, according to Table 6.

Table 6: Input Parameters for the Chloride Ingress Mathematical Tool for Example 3.9.2.

Parameter	Value		Unit	Type of Statistical Distribution
	Mean	Standard deviation		
Cover	55	8.3	mm	Lognormal
$C_{S,\Delta x}$	2.64	0.83	wt%/c	Lognormal
C_0	0.12		wt%/c	Deterministic
C_{crit}	0.65	0.15	wt%/c	Lognormal
$D_{RCM,0}$	6	0.38	10^{-12} m^2/s	Lognormal
a	0.47	0.2	-	Beta (lower limit=0 and upper limit=1)
Δx	8.9	5.6	mm	Beta (lower limit=0 and upper limit=50)
b_e	4800	700	Kelvin	Normal
T_{real}	286.5	4.2	Kelvin	Normal
T_{ref}	293		Kelvin	Deterministic
t_0	28		days	Deterministic
t	25, 50, 75, 100		years	Deterministic

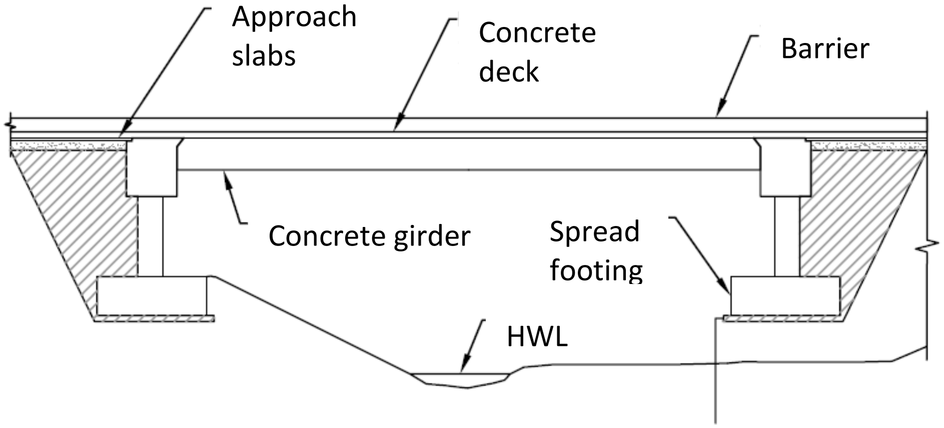
Answer:

A second order reliability method was used to calculate the following probability of failure:

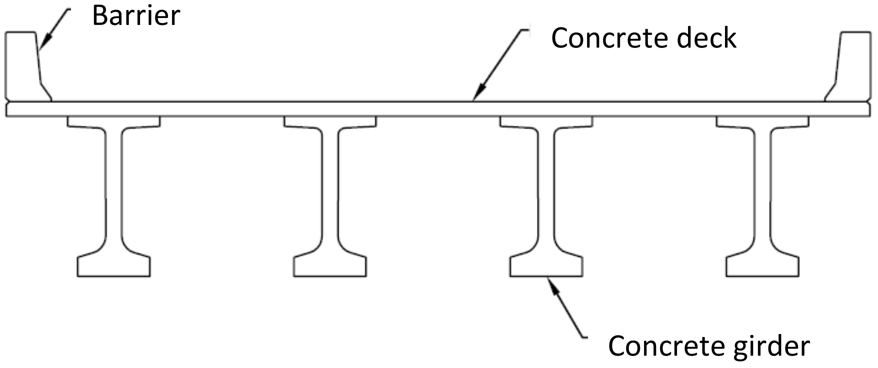
Service Life (years)	P_f (%)
25	20
50	31
75	37
100	40

Example 3.9.3

Figure 16 presents a pre-stressed bulb-tee concrete girder bridge with a concrete deck. Define the exposure zones that could be applicable to this structure assuming that the river below is fresh water and the bridge will be salted in the winter.



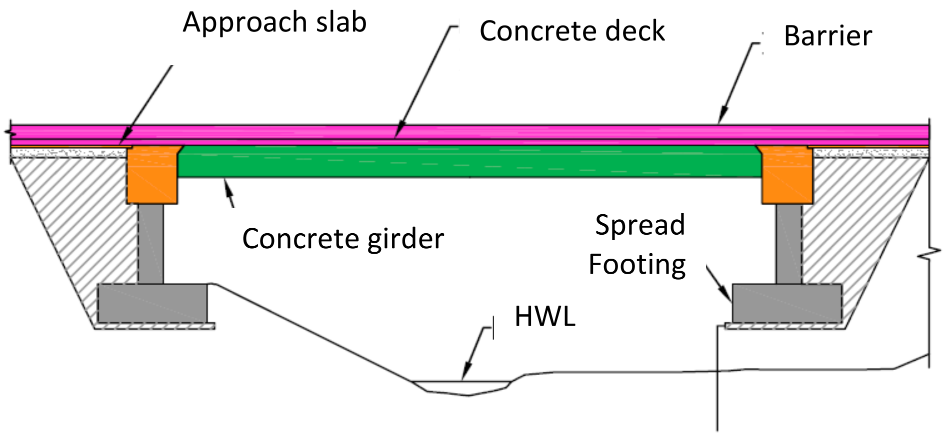
Elevation



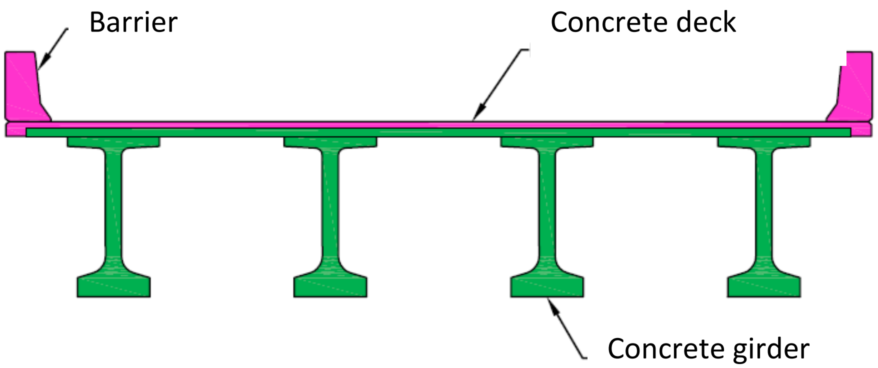
Section

Figure 16: Pre-stressed bulb-tee concrete girder bridge with a concrete deck. The river below is fresh water and the bridge will be salted in the winter.

Proposed Answer: Figure 17 shows a proposed answer. A color code similar to what is presented in Section 3.6.2.2 is used. Typically, covers for buried components are greater than for exposed faces in an atmospheric environment, so, for simplicity, the abutments are considered completely buried.



Elevation



Section

Exposure zones

🟩 Atmospheric without de-icing salts
🟪 Atmospheric with severe de-icing salts
🟧 Atmospheric zone with moderate de-icing salts
⬛ Buried zone

Figure 17: A color code is used to define the different exposure zones of the pre-stressed bulb-tee concrete girder bridge with a concrete deck.

3.10 List of References

American Association of State Highway and Transportation Officials (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.

American Association of State Highway and Transportation Officials (AASHTO). 2011. *Standard Practice for Determining Aggregate Source Shape Values From Digital Image Analysis Shape Properties.* AASHTO PP64.

American Concrete Institute (ACI). 2014. *Building Code Requirements for Structural Concrete and Commentary.* ACI 318.

Angst, U., Elsener, B., Larsen, C.K., and Vennesland, Ø. 2009. *Critical chloride content in reinforced concrete — A review*. Cement and Concrete Research, (39), 1122-1138.

ASTM International (ASTM). 2012a. *Standard Guide for Petrographic Examination of Aggregates for Concrete*. ASTM C295. West Conshohocken, PA.

ASTM International (ASTM). 2012b. *Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals*. ASTM C672. West Conshohocken, PA.

ASTM International (ASTM). 2014. *Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)*. ASTM C1260. West Conshohocken, PA.

ASTM International (ASTM). 2015. *Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete*. ASTM C876. West Conshohocken, PA.

ASTM International (ASTM). 2017. *Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete*. ASTM C457. West Conshohocken, PA.

ASTM International (ASTM). 2018. *Standard Test Methods for pH of Water*. ASTM C1293. West Conshohocken, PA.

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.*

International Organization for Standardization (ISO). 2012. *Service Life Design of Concrete Structures*. ISO 16204.

Federal Highway Administration (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

Federal Highway Authority (FHWA). 2015. *Materials-related Distress: Hardened Cement Paste. Best Practices for Jointed Concrete Pavements.* Technical Brief. FHWA-HIF-15-018. June. https://www.fhwa.dot.gov/pavement/concrete/pubs/hif15018.pdf.

Federal Highway Authority (FHWA). https://www.fhwa.dot.gov/multimedia/research/infrastructure/bridges/ltbp/images/09152014_memorialbridge_fig09.jpg.

International Federation for Structural Concrete (*fib*). 2006. *Model Code for Service Life Design. fib* Bulletin 34, Lausanne, Switzerland, 1st edition, 110 pp.

International Federation for Structural Concrete (*fib*). 2010. *Model Code for Concrete Structures*, Lausanne, Switzerland, 402 pp.

Life-365$^{\text{TM}}$. 2014. Life-365$^{\text{TM}}$ Service Life Prediction Model $^{\text{TM}}$ and Computer Program for Predicting the Service Life and Life-Cycle Cost of Reinforced Concrete Exposed to Chlorides. Version 2.2.1.

Matthews, S. 2015. *Design of Durable Concrete Structures*, IHS BRE Press, 312 pp.

Song, H-W and Saraswathy, V. 2007. *Corrosion Monitoring of Reinforced Concrete Structures – A Review*, Int. J. Electrochem. Sci., 2, 1- 28. Available from: http://www.mkckorea.com/catalog/qseeman/hc-200_ref8.pdf

Stoltzner, E and Sørensen, B. 1994. *Chloride investigations of the Farø bridges* (in Danish).

Tuutti K. 1982. *Corrosion of steel in concrete*. Report No. CBI Research FO 4:82. Stockholm, Sweden, Swedish Cement and Concrete Research Institute.

Uniform Facilities Guide Specification (UFGS). 2012. Section 03 31 29, Marine Concrete.

4.0 Service Life Design of Steel Materials

Structural steel is a common material used for bridge components. It provides high compressive and tensile strengths with considerable ductility, which makes it particularly suitable for long-span bridges.

4.1 Deterioration Mechanisms and Mitigation

The primary deterioration mechanism for steel is corrosion. The steel corrosion reaction is an electro-chemical process requiring the presence of moisture and oxygen to convert the iron in steel into one of its oxides. This process is accelerated in two ways. One method is by stimulation of either the anodic or cathodic portion of the corrosion reaction by aggressive ions such as chlorides (for example, from road de-icing salts) or sulfur dioxide (for example, from atmospheric pollution). The other method involves the establishment of well-defined local galvanic cells that can occur when rust, dirt, or crevices cause differential access to air or from the placement of dissimilar metals in contact. The three types of corrosion most relevant to the use of bare and coated steel are discussed below.

Pitting Corrosion

Pitting corrosion results from the local breakdown of a protective coating or patina. This commences with small pits and can progress rapidly. Prevention will be through proper coating selection and application, or, in the case of uncoated weathering steel, by supporting the formation of a well-adhered, effective patina. Both protection methods are enhanced by regular washing of the surface to remove debris and salts. This is most commonly done using a low-pressure water wash in the spring after the end of the de-icing salt season. Rain water also contributes to the washing process for unsheltered steelwork.

Crevice Corrosion

Crevice corrosion affects bridge elements where layers of steel are connected with bolts spaced greater than sealing pitch, or at welded joints where sealing is faulty, allowing moisture ingress to a limited oxygen environment that sets up a differential aeration cell. "Rust jacking" then forces the layers apart, distorting the metal, increasing stress on the bolts or welds, and allowing additional moisture to penetrate between the mated surfaces. Effective prevention will include satisfying the maximum sealing pitch for bolt connections, welding in accordance with project specifications, well-draining design details, and penetrating sealers and sealants during the coating process.

Galvanic Corrosion

Galvanic corrosion can occur when metals with differing electrical potential are placed in contact, usually in the presence of an electrolyte (water, salts) that allows another path for ion exchange. Prevention will be through material choice, material positioning, and coating to avoid setting up galvanic cells.

Figure 18 shows the structural steel elements susceptible to corrosion.

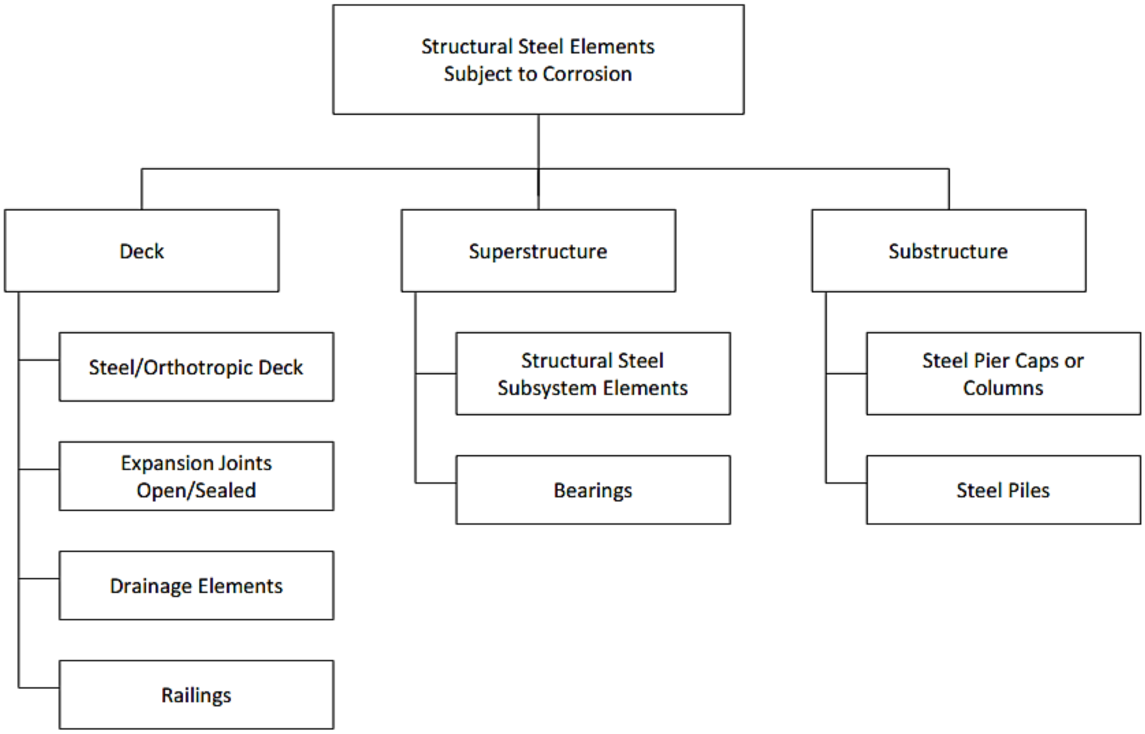


Figure 18: Structural steel elements subject to corrosion (Azizinamini A. et al., 2013).

As previously mentioned, crevice corrosion and galvanic corrosion will be effectively prevented by use of proper material selection and detailing. Three basic methods to prevent corrosion are:

- Use of coating systems
- Use of corrosion-resistant steel or non-corrosive steel
- Avoidance of corrosive environments or corrosion-prone details

4.2 Coating

Coating is a barrier to structural steel, protecting the member from getting wet. Figure 19 shows general considerations that should be made for coating the steel elements of bridges.

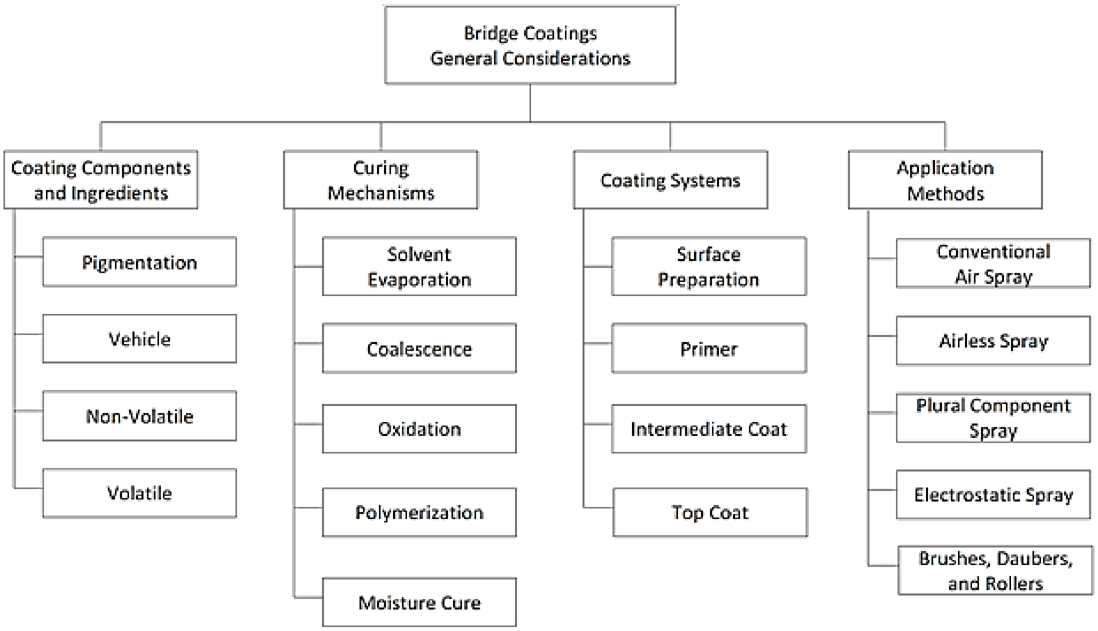


Figure 19: General consideration of bridge coatings (Azizinamini A. et al., 2013).

For painted steel, the corrosion of the steelwork will be eliminated if the paint is properly maintained. Therefore, the service life verification of painted elements is driven by the service life of the paint system. A good reference used for estimating paint life is found in *Expected Service Life and Cost Considerations for Maintenance and New Construction Protective Coating Work* (Helsel and Lanterman, 2016). Guidance from ISO 12944 (ISO, 2017-2018) can be used to characterize the exposure conditions including further guidance on durability of coatings. *The Century Challenge*, FHWA's Coatings and Corrosion Laboratory research published in 2014 by Kodmuri et al., can also be used in selecting successful coating types. Improved service life can also be achieved with improved maintenance. For instance, spring bridge washing can extend painting life, particularly when de-icing salts are used on the bridges.

4.3 Galvanized Steel

As an alternative to the painting discussed in the previous section, hot-dip galvanizing is an effective coating system to prevent steel corrosion. The hot-dip galvanizing process involves immersion of the steelwork in molten zinc, allowing for a metallurgical reaction between the steel and zinc, and providing a tightly bonded coating formed of multiple zinc-iron alloy layers. These layers provide a tough, ductile barrier to external elements, act sacrificially to protect the steel, and, once oxidized, provide an additional protective layer that is resistant to abrasion. The corrosion and abrasion resistance provided by hot-dip galvanizing exceeds that of paint coatings.

The NACE 7422 paper (Helsel and Lanterman, 2016) can also been used to estimate service life of the galvanizing process. As noted previously, improved service life can be achieved with improved maintenance.

Estimates of service life based on severity of environmental exposure are provided by the American Galvanizers Association and relevant results are summarized on Figure 20.

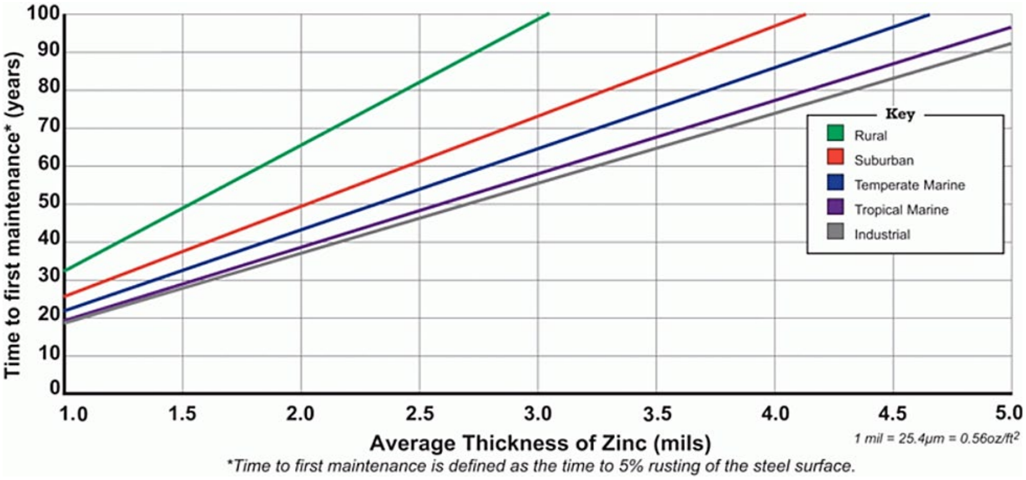


Figure 20: End of service life for various thicknesses of hot-dip galvanizing and environments (Kodumuri et al., 2014).

The data points used for Figure 20 have been collected from several cities and the environmental exposure classes used in the figure relate to which city the specific data point is from.

- 'Rural' relates to Boise, Idaho; Las Cruces, New Mexico; Fargo, North Dakota; Little Rock, Arkansas; Macon, Georgia.
- 'Suburban' relates to Vallejo, California; Tucson, Arizona; Cedar Rapids, Iowa; Jackson, Mississippi; Harrisburg, Pennsylvania; Columbia, South Carolina.
- 'Temperate marine' relates to Seattle, Washington; San Francisco, California; Milwaukee, Wisconsin; Norfolk, Virginia; Atlantic City, New Jersey; Boston, Massachusetts.
- 'Tropical marine' relates to Miami, Florida; Corpus Christi, Texas; San Diego, California; Cancun, Mexico; Mazatlan, Mexico.
- 'Industrial' relates to Pocatello, Idaho; Los Angeles, California; Chicago, Illinois; Dallas, Texas; New York, New York; Knoxville, Tennessee.

4.4 Weathering Steel

The primary method of corrosion protection for uncoated weathering steel is when the properly prepared surface consumes a thin layer of metal to form a dense, well-adhered rust patina that effectively seals the surface beneath, greatly reducing further steel loss to corrosion. The development of an effective weathering steel patina requires initial surface preparation, alternate wet-dry cycles, and limited presence of chlorides. Chlorides are known to pit the surface and prevent the proper formation of the required patina. Preparation of the surface involves the removal of mill scale and cleaning to remove oils, grease, chemicals, and debris that would interfere with patina formation. For all uncoated weathering steel, regardless of exposure zone, the annual spring maintenance washing is an important factor in the long-term durability of the patina.

For uncoated weathering steel, as previously noted, a certain amount of surface metal will be lost because of the formation of the patina as well as corrosion over time. The section loss from corrosion can be estimated using ASTM G101-04 *Standard Guide for Estimating the Atmospheric Corrosion Resistance of Low-Alloy Steels* (ASTM, 2015a). ASTM G101-04 provides projected atmospheric corrosion losses over 50 years for ASTM A588 (ASTM, 2015b) or ASTM A709 (ASTM, 2017) Grade 50W and 70W weathering steels, based on testing in rural, industrial and marine environments in the United States, United Kingdom, Europe, Japan and South Africa.

In the de-icing salts spray zone, the continual presence of chlorides and moisture-holding debris that prevents alternate wet-dry cycles adversely affects the patina bond and effectiveness, wearing it down and causing pitting, while negating much of the weathering steel's ability to self-protect. Significant section loss can result from pitting corrosion under the loosened, ineffective patina. Uncoated weathering steel is therefore not recommended for use in these de-icing salts spray zones or in marine environments. An alternative to weathering steel is, however, to use martensitic stainless steel as described in ASTM A1010 *Higher-Strength Martensitic Stainless-Steel Plate, Sheet, and Strip* (ASTM, 2015c). This type of steel is similar to weathering steel, but with a much higher corrosion resistance, and is therefore suitable for use in marine and de-icing salts spray environments. The use of A1010 steel for bridge structures has been limited; however, initial experience by some states has been positive.

4.5 Exercise

Example 4.5.1

The access components for a new bridge structure are hot-dip galvanized to obtain a service life of 50 years. What is the required minimum thickness of the zinc coating of the access components when the bridge is located in a 1) rural environment, 2) temperate marine environment, and 3) industrial environment? Explain the difference in thickness for the different environments.

Answer:

By using the end-of-service life identified in Figure 20, the average thickness of the zinc coating of the access components as follows:

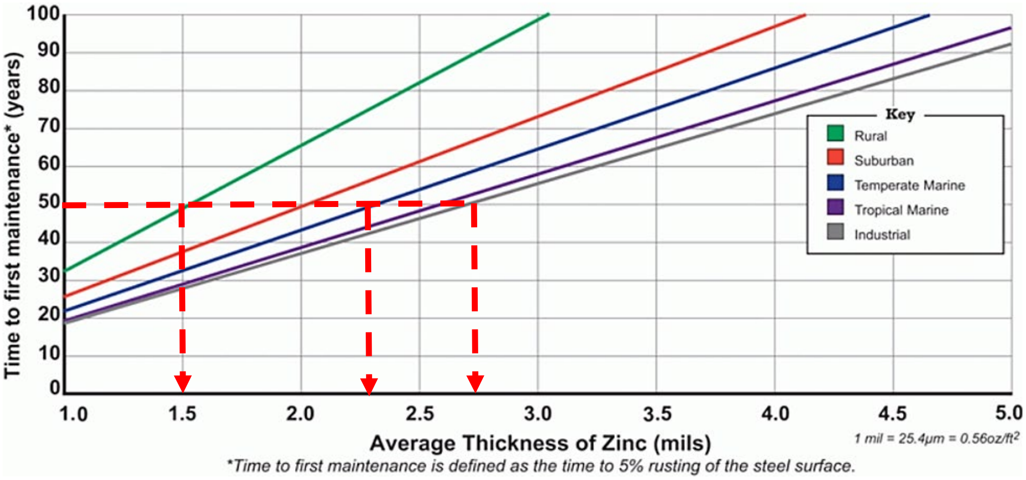


Figure 21. Average thickness of zinc (mils) for access components for a new bridge structure located in different environments.

The time until first maintenance (the service life of the galvanized components) on Figure is defined as the time to 5 percent rusting of the steel surface. For a service life of 50 years, the following required minimum average thicknesses are found in the figure above:

1. Rural: 1.5 mils
2. Temperate marine: 2.3 mils
3. Industrial: 2.7 mils

As seen, the requirement to the minimum average zinc thickness is almost doubled if the bridge is constructed in an industrial area rather than in a rural area. This is as expected because in an industrial area, the access component is exposed to a much harsher environment than in a rural area. Therefore, there is a greater risk of exposure to deteriorating chemicals such as de-icing salts or industrial chemicals. In a temperate marine environment, the access components could also be exposed to salts which can cause corrosion.

4.6 List of References

ASTM International (ASTM). 2015a. *Standard Guide for Estimating the Atmospheric Corrosion Resistance of Low-Alloy Steels* G101-04.

ASTM International (ASTM). 2015b. *Standard Specification for High-Strength Low-Alloy Structural Steel, up to 50 ksi [345 MPa] Minimum Yield Point, with Atmospheric Corrosion Resistance*, A588/A588M-15. West Conshohocken, PA. www.astm.org.

ASTM International (ASTM). 2015c. *Standard Specification for Higher-Strength Martensitic Stainless-Steel Plate, Sheet, and Strip*, A1010/A1010M-13. West Conshohocken, PA. www.astm.org.

ASTM International (ASTM). 2017. *Standard Specification for Structural Steel for Bridges*, A709/A709M-17. ASTM International. West Conshohocken, PA, www.astm.org.

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, 744 pp.

Helsel, J.L. and Lanterman, R. 2016. *Expected Service Life and Cost Considerations for Maintenance and New Construction Protective Coating Work.*, National Association of Corrosion Engineers (NACE) Corrosion 2016 Conference & Expo Paper #7422.

International Organization for Standardization (ISO). 12944. 2017-2018. Paint and Varnishes. Corrosion Protection of Steel Structures by Protective Systems. ISO. 12944-1 to 12944-8.

Kodumuri, P., Lee, S-K., and Virmani, Y.P. 2014. *The Century Challenge*, Federal Highway Administration, FHWA-HRT-14-002, January. The American Galvanizers Association, http://www.galvanizeit.org/hot-dip-galvanizing/how-long-does-hdg-last/in-the-atmosphere/time-to-first-maintenance.