Appendix D
Design Examples and Calculation Books

A total of four Service Life Design Report examples are provide in this appendix as described in Table 1.

The first example includes a worked service life design example from a conventional multiple span composite-deck highway overpass bridge located in the Northeast region of the US. The bridge is subjected to heavy de-icing salt use and freeze-thaw cycles. The second example provides a concrete durability assessment for substructure elements including reinforced concrete foundations, drilled shafts, pile caps, towers and anchor piers. The bridge is located in the Mideast of the United States. The third example elaborates a calculation booklet for the service life design of a conventional multi-span prestressed concrete girder bridge located in Southeast United States over coastal sea or brackish waterway.

The fourth example uses the first three examples extended by performing multiple full probabilistic calculations with varying chloride loading and concrete material durability resistance parameters to develop a theoretical partial factor design approach for chloride-induced reinforcement corrosion. This example includes a technical report on the development of a potential partial factor methodology for chloride-induced reinforcement corrosion, which may be used in the future when sufficient data has been collected to substantiate load and resistance partial safety factors (PSFs).

Table 1 summarizes details on the examples including the type of bridge, basic details on location and exposure conditions applicable in the example, and details on the examples provided.

Table 1 Overview of Design Examples

Example/ Section No.	Bridge type	Location and brief exposure conditions description	Example of:	Pages
D1	Multi-span composite-deck highway overpass bridge	Urban environment in New York Periods of snow, freeze-thaw cycles Heavy use of de-icing salts	Detailed Service Life Design report Monte Carlo simulations used to assess cover and chloride migration coefficient requirements 75-year service life for non-replaceable components	D-4 to D-35
D2	A two-span cable-stayed bridge	Mideast United States over a river Periods of snow, freeze-thaw cycles and de-icing salts	Detailed Service Life Design report Probabilistic modeling software used to assess cover and chloride migration coefficient requirements 100-year service life for non-replaceable components	D-36 to D-70
D3	Conventional multi-span prestressed concrete girder bridge	Southeast United States over coastal bayou Splash, spray, and tidal exposure	Service Life Design calculation booklet Probabilistic modeling software used to assess cover and chloride migration coefficient requirements Consideration of prestressing Varying service life requirements for concrete elements	D-71 to D-82
D4	A report on the development of a theoretical partial factor methodology for verification of service life regarding chloride-induced reinforcement corrosion, based on and compared to Examples 1-3.			D-83 to D-115

List of Symbols

Throughout this appendix, the following symbols are used:

- a age factor (-)
- A lower bound of beta distribution
- B upper bound of beta distribution
- b_e regression variable (K)
- c cover depth (millimeters [mm])
- C_0 initial chloride concentration (mass-% of total cementitious materials)
- C_{crit} chloride threshold of the reinforcement (mass-% of total cementitious materials)
- $C_{s,\Delta x}$ surface chloride concentration at a depth Δx from the surface (mass-% of total cementitious materials)
- $D_{app,C}$ apparent coefficient of chloride diffusion through concrete (square millimeters per year [mm²/year])
- $D_{RCM,0}$ chloride migration coefficient (mm²/year)
- erf error function
- k_e environmental transfer variable (-)
- k_t transfer parameter (-)
- t_0 reference point of time (years)
- t_{SL} target service life (years)
- T_{real} temperature of the structural element or the ambient air (K)
- T_{ref} standard test temperature (K)
- w_{cm} weight of total cementitious materials
- Δx depth of the convection zone (transfer function) (mm)
- α representative α-value (for determining PSFs) (-)
- β reliability index (-)
- μ mean value
- σ standard deviation

D1 Example 1 – Service Life Design Report for a Multi-span composite-deck highway overpass bridge

Contents

D1.1 Introduction

This document provides a worked service life design example of a conventional multiple span composite-deck highway overpass bridge located in the Northeast region of the US, which is subjected to heavy de-icing salt use and freeze-thaw cycles.

Figure 1-1 shows the general arrangement of the Bridge:

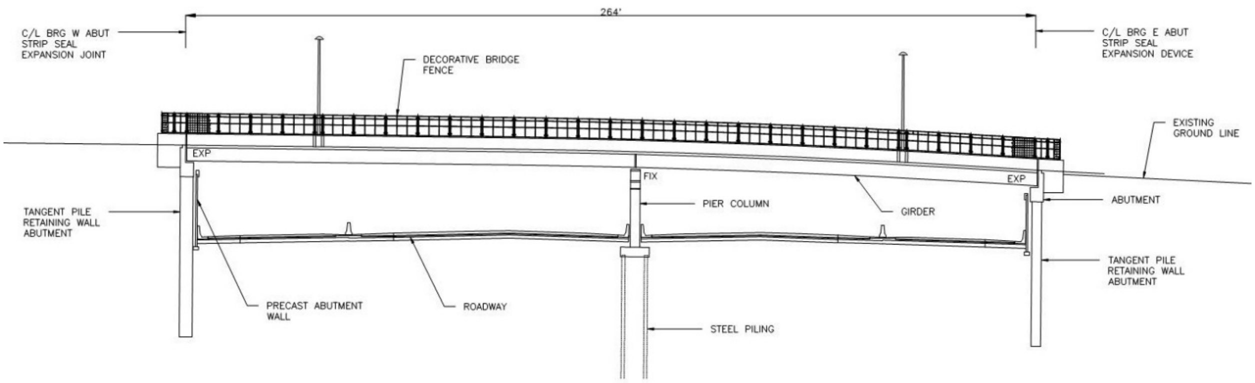


Figure 1-1: General arrangement of the Bridge.

D1.1.1 Location

- New York City.
- Highway under the bridge.
- Urban environment with periods of snow and freeze-thaw cycles.
- Annual mean temperature of 11.5 degrees Celsius (°C) (52.7 degrees Fahrenheit [°F]) [1].
- Heavy use of de-icing salts.
- Some sulfate present in soil: 0.14% by mass of water soluble sulfate was measured.

D1.1.2 General Bridge Superstructure Characteristics

- 264-foot (ft.) span steel-girder bridge with 2 spans (139 ft. and 125 ft.).
- Deck system is comprised of a composite cast-in-place concrete deck and steel girders.
- Over the abutments, the girders are supported on elastomeric bearings and at the piers, the girders are supported on fixed bearings.
- Deck carries two traffic lanes, each with a width of 12 ft., and a sidewalk with a width of 6 ft. on the side. Figure 1-2 shows a typical section of the superstructure.
- Wearing surface is high-performance concrete, no asphalt or waterproofing membrane.

- Deck and girders are continuous over the pier.
- Uncoated reinforcement (black steel) is used everywhere.

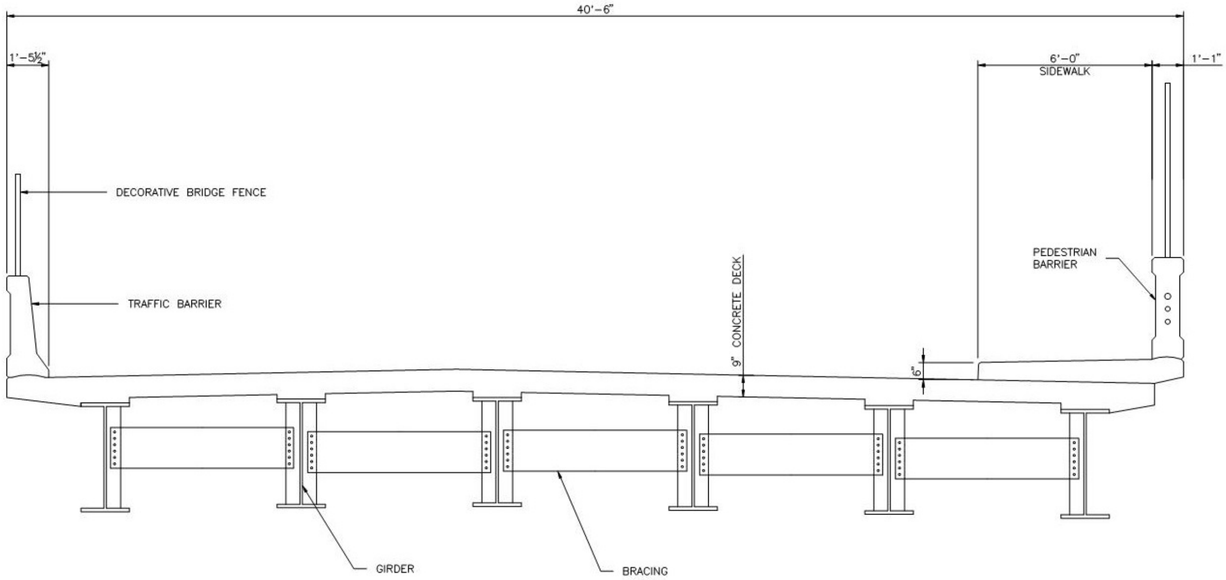


Figure 1-2: Typical section of superstructure of the Bridge.

D1.1.3 General Bridge Substructure Characteristics

- The central pier has three columns each supported by a pile cap and steel H-piles driven into bedrock. Figure 1-3 shows a typical section of the pier.
- Abutments are supported by reinforced concrete tangent piles, see Figure 1-4.
- Full height precast abutment wall in front of the abutments protect them.
- Expansion joints are located between abutments and concrete deck.
- Uncoated reinforcement (black steel) is used everywhere.
- No mass concrete.

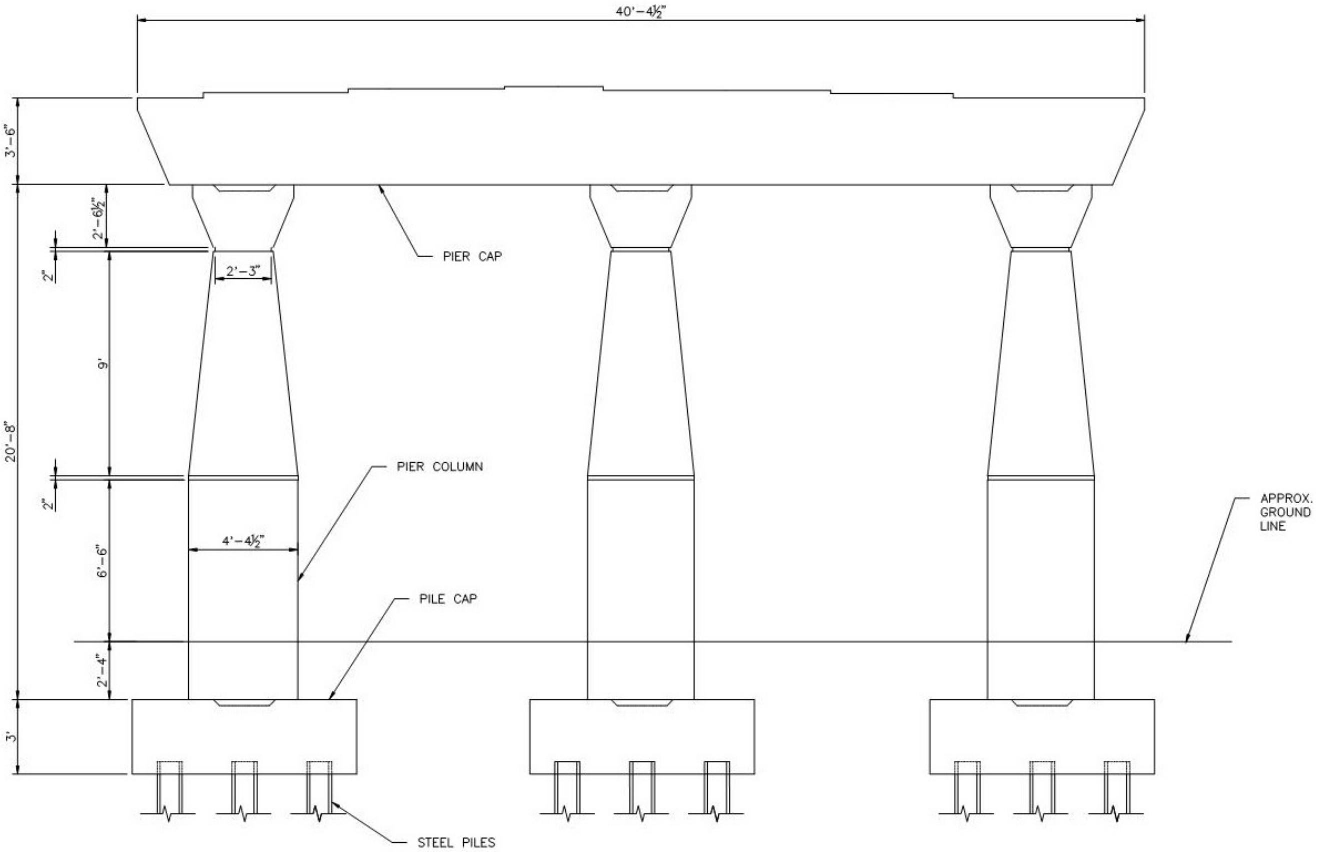


Figure 1-3: Pier elevation of the Bridge.

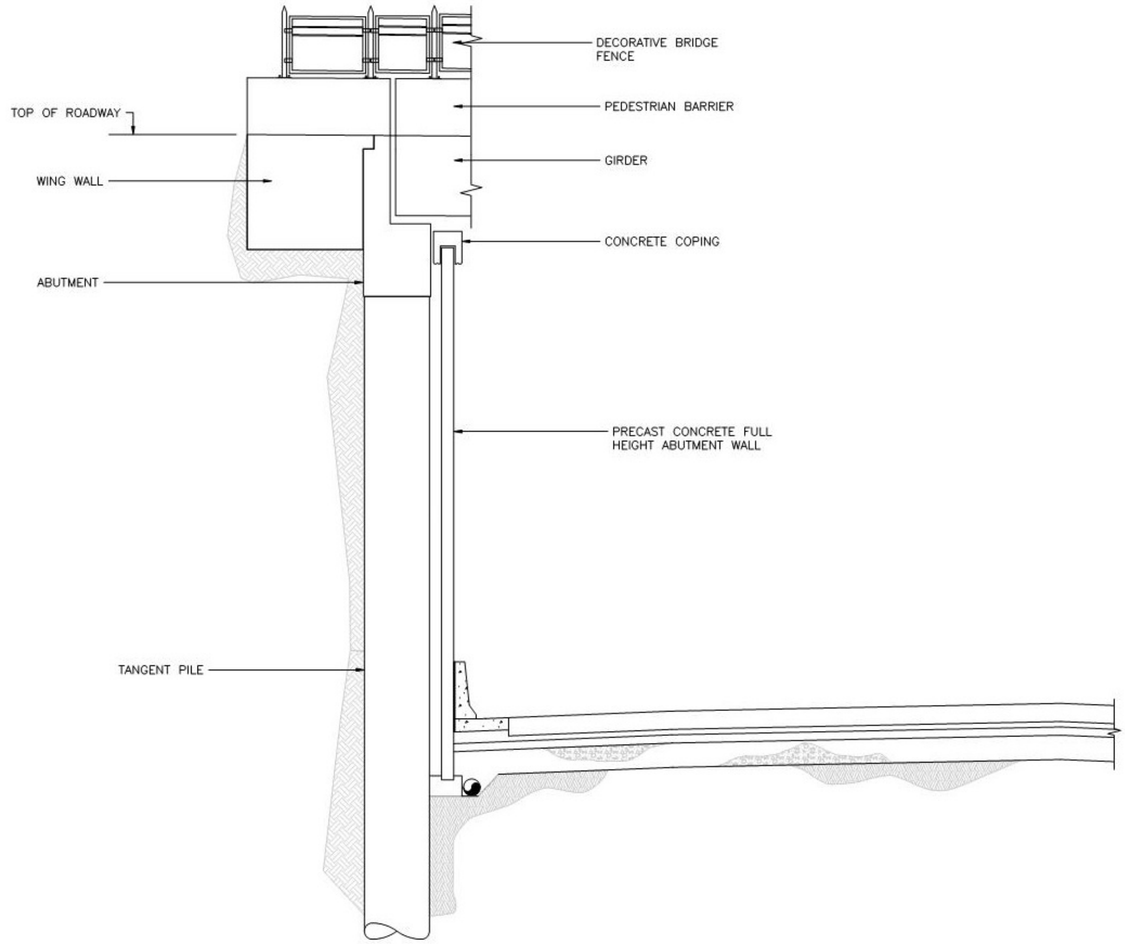


Figure 1-4: Typical section at abutment of the Bridge.

D1.1.4 Service Life Requirements

- Non-replaceable components must meet a minimum service life of 75 years. This service life shall be achieved with preservation activities (i.e., cyclical and condition-based maintenance) and without the need for replacement, rehabilitation, or preventative maintenance.
- For chloride-induced corrosion in concrete structures, the limit state is to achieve the specified service life with a target confidence level of 90% (approximately equivalent to a reliability index of 1.3) based on guidance provided by International Federation for Structural Concrete (*fib*) Bulletin 34 [4].
- The replaceable components must meet a minimum service life as shown in Table 1-1.

D1.1.5 Recommended Service Life Design Procedure

1. Define exposure zones for all bridge components.

2. Define deterioration mechanisms for each exposure zone.
3. Define mitigation methods for deterioration mechanisms for concrete components.
4. Define mitigation methods for deterioration mechanisms for steel components.

Table 1-1. Requirements to Minimum Service Life of the Bridge.

Non-Replaceable Components	Minimum Service Life (years)
Foundations, abutments, piers, structural steel, and deck	75
Replaceable Components	**Minimum Service Life (years)**
Bridge bearings	50
Expansion joints	30
Painting	25
Barriers	50

D1.2 Exposure Zones

D1.2.1 Defined Exposure Conditions

- Buried: zone permanently buried in soil. Abutment and tangent pile surfaces exposed to soil, pile cap, steel piles.
- Indirect de-icing salts: zone subject to runoff water or spray containing de-icing salts, typically areas under and within 10 ft. of expansion joints or between 6 ft. and 20 ft. vertically from a roadway. Girder, bracing, pier column, pier cap, abutment wall.
- Direct de-icing salts: zone directly exposed to the use of de-icing salts. Top surface of deck, traffic barrier, pedestrian barrier, piers directly next to roadway up to 6 ft. vertically of the roadway, fencing.
- Atmospheric: zone not exposed to soil or de-icing salts. Bottom surface of deck, wing wall surfaces and tangent pile surfaces exposed to atmospheric air.

D1.2.2 Color Code to Identify Exposure Zones

Typical exposure conditions are presented on Figure 1-5 to Figure 1-8.

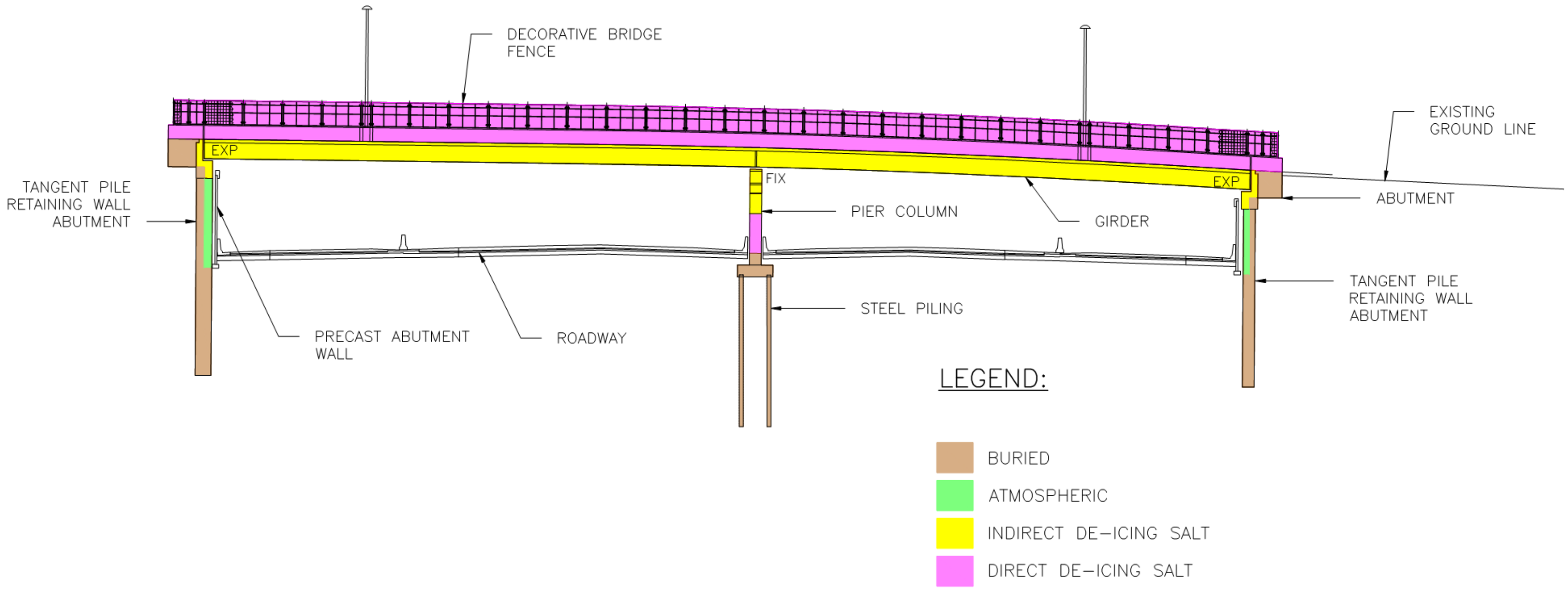


Figure 1-5: Exposure zones for the Bridge, longitudinal section.

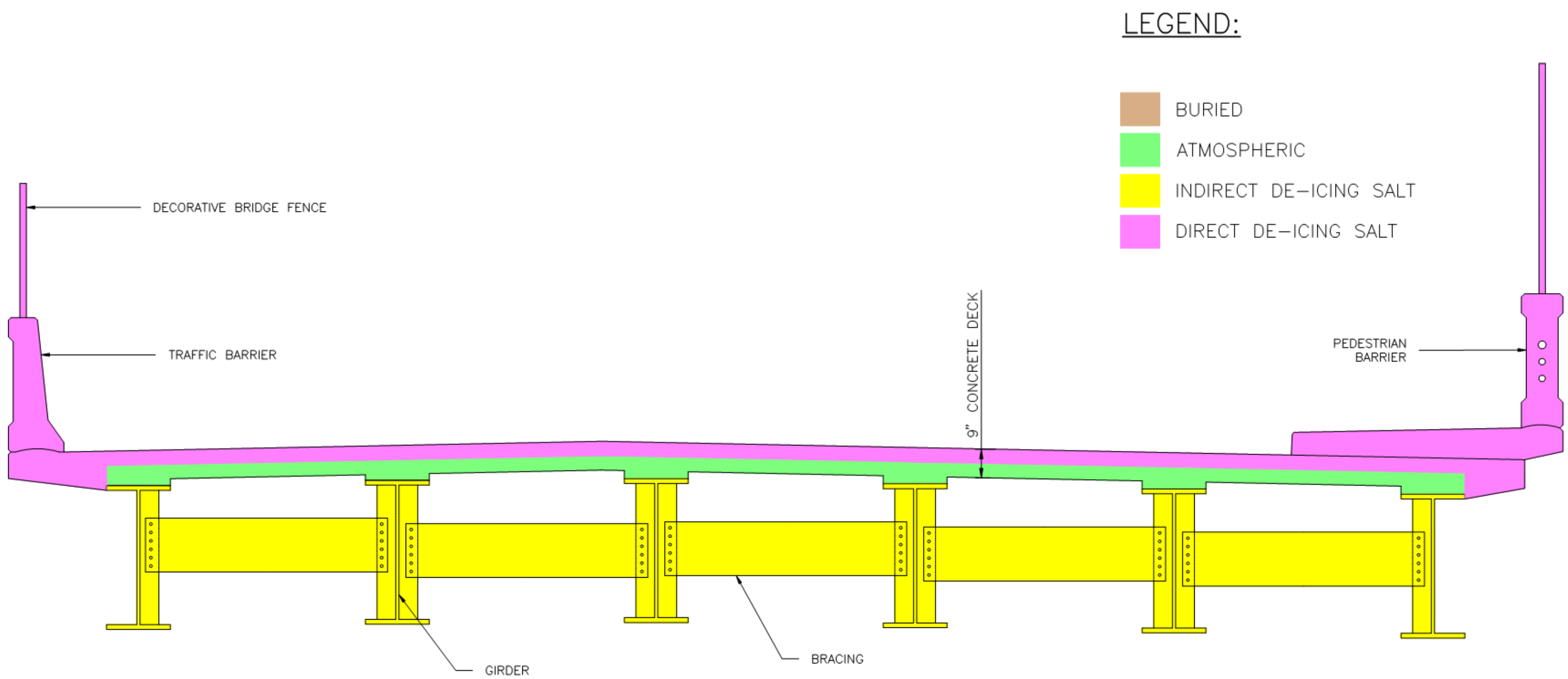


Figure 1-6: Exposure zones for superstructure.

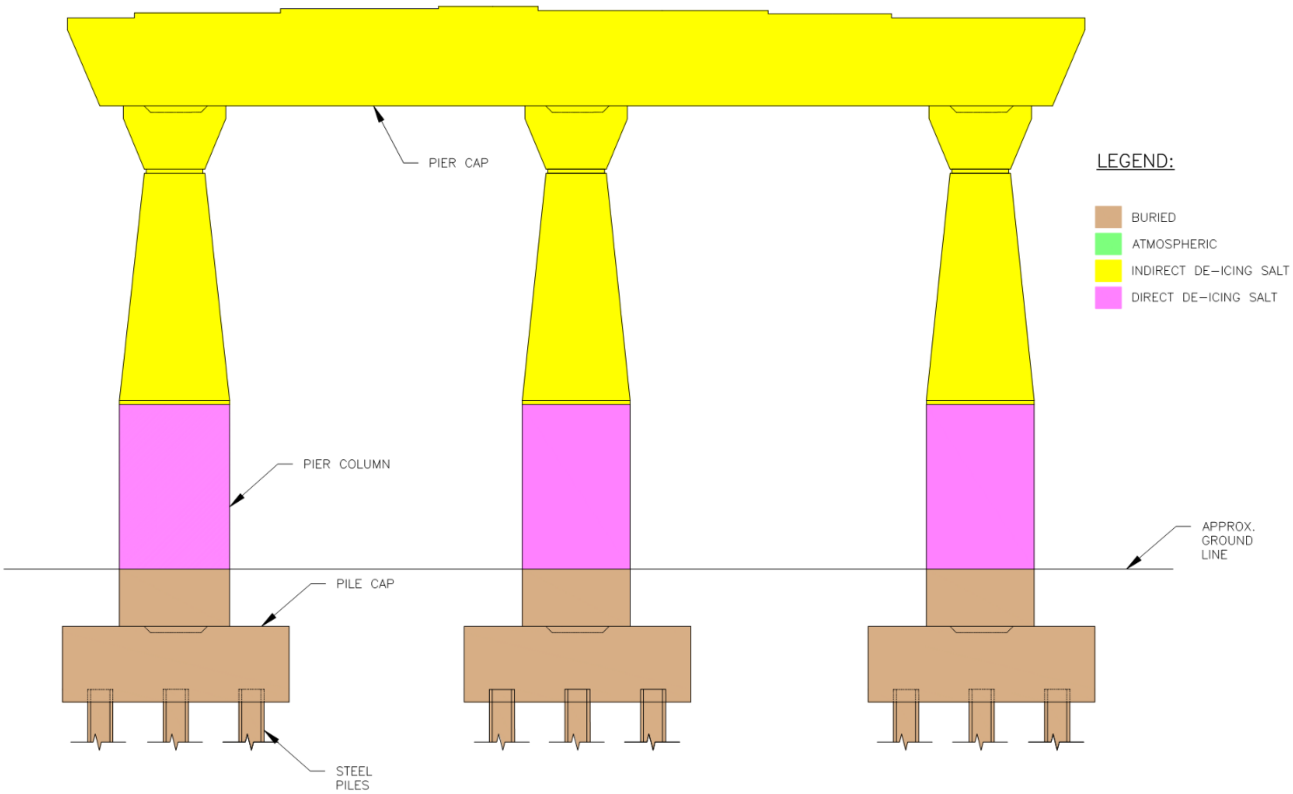


Figure 1-7: Exposure zones for substructure.

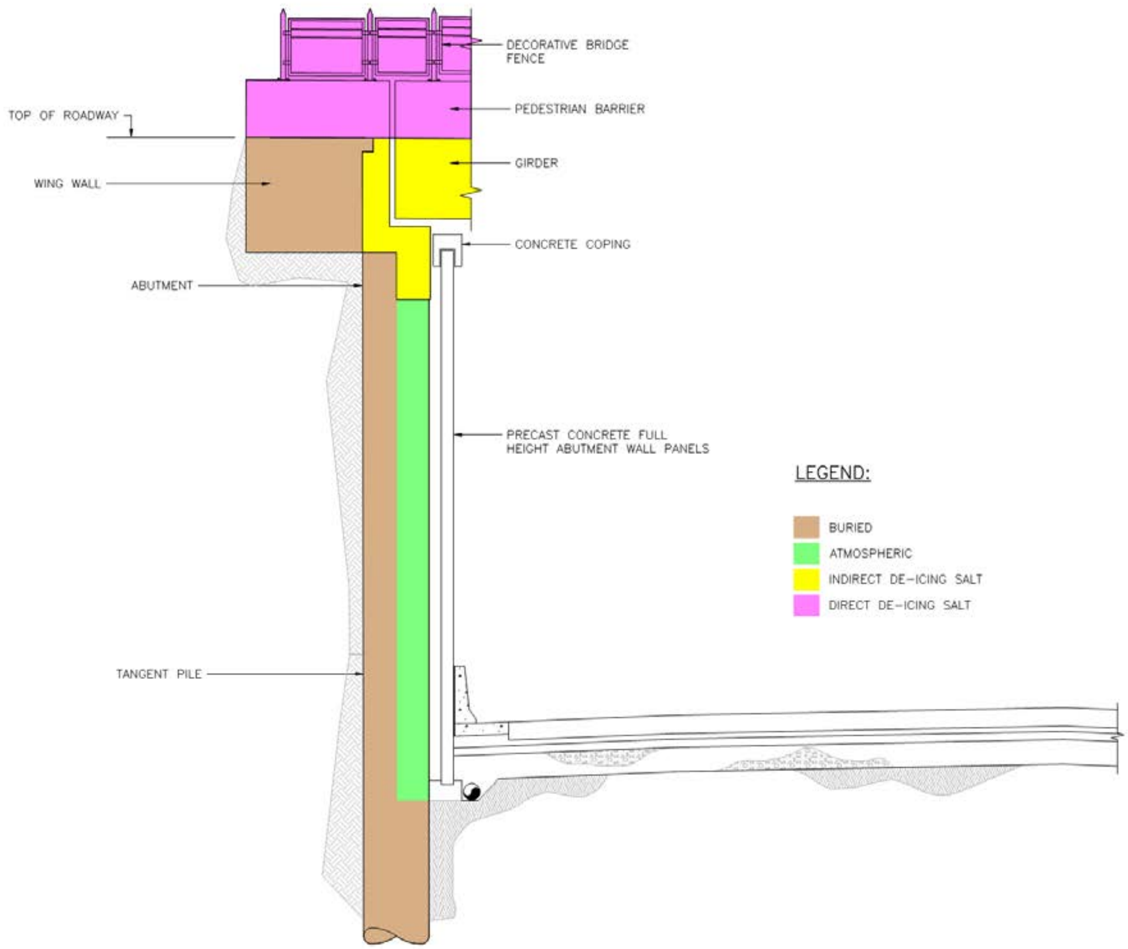


Figure 1-8: Exposure zones for abutments supported by tangent piles.

D1.3 Deterioration Mechanisms

D1.3.1 Considered Deterioration Mechanisms

D1.3.1.1 Considered Deterioration Mechanisms for Concrete

- **Alkali-Aggregate Reactions (AAR):** Aggregates containing reactive minerals react with alkalies from the cement and/or from external sources, such as de-icing salts, under the presence of water and high pH-value to form an expansive gel.

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

- **Freeze-thaw:** Freeze-thaw cycles can cause deterioration (cracking) when the pore structure of the cement paste is not designed with a sufficiently fine entrained air system, the concrete is critically saturated, and the water in the pores freezes to ice and expands.
- **Scaling:** The expansion of water because of freezing and thawing cycles combined with the use of de-icing chemicals can lead to scaling, which is a general loss of surface mortar.
- **Carbonation-induced corrosion:** Carbon dioxide from the surrounding air reacts with calcium hydroxide in the cement paste, which decreases the pH-value of the concrete pore solution. The alkaline protective reinforcement environment breaks down, which can initiate reinforcement corrosion.
- **Chloride-induced corrosion:** Chloride ions from seawater or de-icing salts can penetrate the concrete through the pore solution. A concentration of chloride ions in excess of the critical chloride threshold can initiate depassivation of the reinforcement, and eventually, corrosion.
- **Delayed Ettringite Formation (DEF):** Form of internal sulfate attack that can occur in concrete cured at elevated temperatures such as in precast units or mass concrete placements.
- **Ice abrasion (not applicable for this example):** Ice floes can impact and rub against concrete components abrading the surface, which causes loss of concrete over time. This is mainly affecting pier columns and piles located in major rivers at water level and therefore is not relevant for this example.

D1.3.1.2 Considered Deterioration Mechanisms for Steel

- The main deterioration mechanism for steel is corrosion.

D1.3.2 Deterioration Mechanisms for Different Components

- Table 1-2 specifies which components are exposed to different types of deterioration along with the severity categories for each exposure condition. Exposure zones from International Standards Organization (ISO) 12944 [2] for steel elements and American Concrete Institute (ACI) 318-14 [3] for concrete elements and are included for reference.
- Soil data specific to the site has shown a level of sulfates corresponding to S1 exposure according to ACI 318-14.
- DEF must also be included if any of the concrete components are precast.

Table 1-2: Exposure Zones and Deterioration Mechanisms for the Bridge.

Exposure Zone	Examples of Elements for Piers	Exposure Conditions	Steel Corrosivity Category ISO 12944-2 [2]	Exposure Zones ACI 318-14 [3]	Potential Concrete Deterioration Mechanisms					
					Materials		Environmental			
					AAR	Sulfate	Freeze-thaw	Scaling	Carbonation-induced Corrosion	Chloride-induced Corrosion
Buried	Pile cap, wing wall, abutment wall, tangent piles.	Limited chloride exposure in soil. Limited O_2. Freeze-thaw above frost line. Sulfates.		S1, C1, F1	x	x	x			x
	Steel piles at the pier.		Im3: soil							
Atmospheric	Cast-in-place deck bottom surface, wing wall, face of tangent piles facing the precast concrete full height wall.	Atmospheric O_2 and CO_2. Some airborne chlorides. Temperature and humidity variations, including freeze-thaw.		F2	x		x		x	x
Indirect De-icing Salts	Areas under or within 10 ft. horizontally of expansion joints, zone within 6-20 ft. vertically of a roadway: upper part of pier columns, pier cap, abutment wall.	Alternating wetting and drying. Atmospheric O_2 and CO_2. Freeze/thaw with indirect exposure to de-icing salts, leakage from deck joints, temperature and humidity variations.		C2, F3	x		x		x	x
	Girders.		C4: Temperate zone, atmosphere with moderate salinity							
Direct De-icing Salts	Top surface of decks, barriers, pier columns within 6 ft. vertically of a roadway.	Alternating wetting and drying. Atmospheric O_2 and CO_2. Freeze/thaw with direct exposure to de-icing salts applications, temperature and humidity variations.		C2, F3	x		x	x	x	x
	Decorative fence.		C5-M: Temperate zone, aggressive atmosphere							

D1.4 Mitigation Methods for Concrete Components

- Table 1-3 shows the mitigation methods identified for the different concrete deterioration mechanisms.
- Guidance from ACI 318-14 was used and modified as necessary.

D1.4.1 Full Probabilistic Modeling of Chloride-Induced Corrosion

D1.4.1.1 *fib* Bulletin 34 Chloride-Induced Corrosion Model

- For non-replaceable components, the limit state is to achieve 75-year service life (50 years for barriers) with a target confidence level of 90% (reliability index of 1.3). The confidence level is based on guidance from *fib*.
- Service life is considered equal to corrosion initiation time.
- Parameters are modelled in accordance with guidance provided by *fib* Bulletin 34 by using the following equation for the chloride content C at depth x and time t:

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

where:

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

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

$$A(t)=\left(\frac{t_0}{t}\right)^a$$

- Table 1-4 gives an evaluation of the input parameters used in the above equations.
- The calculations are performed in metrics. US units are shown as applicable.
- Two types of mix designs, both containing a minimum of 590 pounds per cubic yard (lbs/yd^3) (350 kilogram per cubic meters [kg/m^3]) of cementitious materials, are assumed based on availabilities of local materials:
  - OPC: Portland Cement Type I or Type II only.
  - OPC+20-35%FA: Portland Cement Type I or Type II with 20%-35% Type F fly ash (FA) by mass of total cementitious materials.

- Table 1-4 shows the parameters chosen for the modeling of concrete mix design ‘OPC+20-35%FA’ for the deck directly exposed to de-icing salts.
- Table 1-5 summarizes the input parameters for the chloride-induced corrosion model for all structural elements and all exposure zones for both types of concrete-mix design (OPC and OPC+20-35% FA).

Table 1-3: Mitigation Methods for the Identified Concrete Deterioration Mechanisms.

Deterioration Mechanism	AAR	DEF	Sulfate Attack	Freeze-Thaw and Scaling	Carbonation-induced Corrosion	Chloride-induced Corrosion
Design strategy	Avoidance of deterioration				Deemed-to-satisfy	Full probabilistic approach following *fib* Bulletin 34 [4].
Considerations	Local non-reactive aggregates may not be available or long-term test data may not be available.	Only applicable if there are precast components.	Geotechnical measurements indicate that the soil surrounding the abutments is contaminated and has a sulfate content of 0.14%. ACI 318-14 states that sulfate attack is not applicable when the sulfate content is below 0.1% in soil and therefore sulfate mitigation methods must be identified.	All parts of the concrete structure will be exposed to freeze-thaw cycles. In addition, concrete exposed to freeze-thaw cycles and de-icing salts is subject to scaling.	Mitigation methods for chloride-induced corrosion also prevent carbonation-induced corrosion and will govern.	The probabilistic model in *fib* Bulletin 34 is based on Fick's second law of diffusion and contains improvements to yield a good approximation of chloride distribution in concrete.
General Mitigation Methods	Mitigation methods include: - Limiting the alkali contribution by the Portland Cement to the concrete; and - Using a sufficient amount of effective supplementary cementitious materials.	Mitigation methods include: - Application of a maximum temperature of 160°F during initial curing (~7 days). - Use of FA or ground granulated blast furnace slag (GGBS).	Mitigation methods include: - Using Portland Cement with a low alkali content and C_3A-content (sulfate resistant cement, Type II or V); - Providing a concrete with low permeability and a low water-cement ratio (w/c ratio); and - The use of supplementary cementitious materials.	Mitigation methods include: - Using freeze-thaw resistant aggregates; and - Providing air-entrainment in the concrete. - The supplementary cementitious materials content should be limited for concrete with a risk of scaling. For decks and barriers, a limit of 25% FA by total mass of cementitious is typically used.	Mitigation methods for carbonation-induced corrosion include low concrete permeability and adequate concrete cover.	Mitigation methods include: - Use of low permeability concrete; - Adequate concrete cover; - Use of corrosion-resistant reinforcing (not used in this example); and - Proper control of cracking per applicable structural design code and construction specifications.

Deterioration Mechanism	AAR	DEF	Sulfate Attack	Freeze-Thaw and Scaling	Carbonation-induced Corrosion	Chloride-induced Corrosion
Requirements in U.S. Codes and Standards	Guidance from American Association of State Highway and Transportation Officials (AASHTO) R80-17 [5] can be used.	Not applicable (NA)	Requirements according to ACI 318-14 for concrete classified as S1: - Maximum w/c ratio of 0.50 and a minimum compressive strength of 4000 pounds per square inch (psi) (28 megapascal [MPa]). - American Society for Testing and Materials (ASTM) C150 Type II cement is allowed. Types I and III are also allowed if the C_3A-content is less than 8%.	Requirements according to ACI 318-14: - For concrete classified as F1: a maximum w/c ratio of 0.55 and a minimum compressive strength of 3500 psi (24 MPa). Plastic air content of 4.5% for maximum aggregate size of 1 inch (in.). - For concrete classified as F2: a maximum w/c ratio of 0.45 and a minimum compressive strength of 4500 psi (31 MPa). Plastic air content of 6% for maximum aggregate size of 1 in. - For concrete classified as F3: a maximum w/c ratio of 0.40 and a minimum compressive strength of 5000 psi (35 MPa). Plastic air content of 6% for maximum aggregate size of 1 in.	NA	Requirements according to ACI 318-14 for concrete classified as C2: - Maximum w/c ratio of 0.40 and a minimum compressive strength of 5000 psi (35 MPa). - Maximum water-soluble chloride content in concrete of 0.15 mass-% of cement (this limit is reduced to 0.1 mass-% of total cementitious materials for acid-soluble chloride or 0.8 mass-% for water-soluble chloride).
Required Testing	The following testing is required based on AASHTO R80-17: - Expansion in accordance with ASTM C1260 [6] or ASTM C1293 [7] in order to determine aggregate-reactivity class. - Petrographic analysis per ASTM C295 [8]. If aggregates are shown to be reactive, additional mitigation measures as per AASHTO R80-17 can be implemented.	If precast concrete is used: - Limit curing temperatures to 160°F. - To be measured using temperature sensors.	No testing required. Implement limits on cementitious materials as per ACI 318-14.	The following testing is required (includes more than required by ACI 318-14 to demonstrate that the concrete has sufficient resistance): - Plastic air content of freshly mixed concrete tested. ACI requirement: see section "Requirements according to ACI 318-14." - Air-void system of hardened concrete in accordance with ASTM C457 [9]. ACI guideline: maximum spacing factor of 0.008 inches. - Freeze-thaw resistance in accordance with ASTM C666 Procedure A [10]. Recommendation: minimum durability factor of 90 after 300 cycles. - Resistance to scaling for deck and barrier concrete in accordance with ASTM C672 [11]. Requirement: a visual rating equal or less than 3 after 50 cycles, this means that moderate scaling (visible coarse aggregate) is allowed at the end of the test. Alternatively: test CSA A23.2-22C can be used, a maximum mass loss of 0.16 per square feet (0.8 kg/m^2) can be used as a passing criterion.	NA	The following testing is required: - The chloride migration coefficient must be determined from migration tests in accordance with NT Build 492 [12] at 28 days. - Water-soluble chloride (ASTM C1218 [13]) or acid-soluble chloride (ASTM C1152 [14]) Test criteria will be determined by the modeling.

Table 1-4: Input Parameters for *fib* Bulletin 34 Modeling of Chloride-induced Corrosion of Concrete Deck Exposed Directly to De-icing Salt Using the Concrete Mix 'OPC+20-35% FA'.

Variable	Symbol	Short Description	*fib* Bulletin 34 Recommendations	Used in Example for Direct De-icing Salt Exposure Zone			
				Distribution	Unit	Mean	Standard Deviation and Function Parameters
Cover	c	Concrete thickness measured from concrete surface to the surface of the outermost steel reinforcement.	Recommends that the distribution function for large cover depths be typically chosen as a normal distribution, whereas for small cover depths, distributions excluding negative values should be chosen, such as the lognormal function. For this example, covers from AASHTO LRFD [16] are used as starting point. It is assumed that 90% of the cover is within the construction tolerance of ± 0.5 in. For a normal distribution, this means that the standard deviation is found by dividing the tolerance by a z-value of 1.645.	Normal	mm (in.)	70 (2.75)	7.6 (0.3)
Temperature	T_{real}	Temperature of the structural element or the ambient air.	Recommends that T_{real} can be determined by using available data from a weather station nearby the structure. The data used for this example is based on public data for monthly averages for New York City [1]. A mean value of 11.5°C is determined as the annual average temperature. The standard deviation is estimated from the expected value over a period of 100 years. A value of 2°C is assumed. Can be calculated if sufficient data are available.	Normal	°C (°F)	11.5 (52.7)	2.0 (3.6)
Initial Chloride Concentration	C_o	Initial chloride content in concrete at time t = 0.	States that the initial chloride content in the concrete is not only caused by chloride ingress from the surface, but can also be due to chloride contaminated aggregates, cements, or water used for the concrete production. The total amount of chlorides present in the concrete mix will be determined during the construction phase and will be specified to be less than the assumed value.	Deterministic	Mass-% of total cementitious materials	0.1	-
Surface Concentration	$C_{s,\Delta x}$	Chloride content at the depth Δx.	States that it depends on material properties and on geometrical and environmental conditions. Ideally, data are gathered from similar structures. In this example, the surface concentration is based on interpretation of measured in situ chloride surface concentration of bridge decks from the literature.	Lognormal	Mass-% of total cementitious materials	4.0	2.0
Chloride Migration Coefficient	$D_{RCM,0}$	Chloride migration coefficient measured from NT Build 492 at t = 28 days.	Recommends the standard deviation of the chloride migration coefficient to be 0.2 times the mean value. The mean value is assumed in the model such that the desired reliability index is obtained.	Normal	x 10^{-12} square meters per second (m^2/s)	7.0	1.4
Aging Factor	a	The age factor describes the time-dependent change of the migration coefficient as concrete matures.	*fib* Bulletin 34 and *fib* Bulletin 76 [15] recommend the following aging factors for concrete with an equivalent w/c ratio between 0.40-0.60:	Beta	-	0.6	σ=0.15 A=0; B=1

<table>
<tr><th rowspan="2">Variable</th><th rowspan="2">Symbol</th><th rowspan="2">Short Description</th><th rowspan="2">fib Bulletin 34 Recommendations</th><th colspan="4">Used in Example for Direct De-icing Salt Exposure Zone</th></tr>
<tr><th>Distribution</th><th>Unit</th><th>Mean</th><th>Standard Deviation and Function Parameters</th></tr>
<tr><td></td><td></td><td></td><td><table>
<tr><th rowspan="2">Concrete mixes</th><th rowspan="2">Distr.</th><th colspan="2">Submerged/buried, water level, de-icing salts zones</th><th colspan="2">Atmospheric zone</th></tr>
<tr><th>Parameters</th><th>Mean (μ)</th><th>Parameters</th><th>Mean (μ)</th></tr>
<tr><td>Portland Cement + 20-35% FA</td><td>Beta</td><td>σ=0.15,
A=0; B=1</td><td>0.60</td><td>σ=0.15,
A=0; B=1</td><td>0.65</td></tr>
<tr><td>Portland Cement</td><td>Beta</td><td>σ =0.12,
A=0; B=1</td><td>0.30</td><td>σ=0.15,
A=0; B=1</td><td>0.65</td></tr>
</table>μ = mean value; σ = standard deviation; A and B are the upper and lower bounds.</td><td></td><td></td><td></td><td></td></tr>
<tr><td>Transfer Function</td><td>Δx</td><td>Capillary action leads to a rapid transport of chlorides into the concrete up to a depth Δx from the surface. Beyond this depth, chloride ingress is controlled by diffusion.</td><td>Recommends the following values for the transfer function:
- For water level, direct and indirect de-icing salts zones: beta distribution with a mean value of 8.9 mm, standard deviation of 5.6 mm with parameter A = 0.0 and B = 50.0.
- For buried, submerged, and atmospheric zones: deterministic value of 0.</td><td>Beta</td><td>mm
(in.)</td><td>8.9
(0.35)</td><td>σ=5.6
(0.22)
A=0; B=50</td></tr>
<tr><td>Critical Chloride Concentration</td><td>C_{crit}</td><td>Concentration required to break down the passive layer protecting the steel reinforcement.</td><td>Recommends using a beta distribution with a mean value of 0.6% by mass of cementitious materials (based on uncoated carbon steel reinforcement), a standard deviation of 0.15, a lower bound of 0.2, and an upper bound of 2.0.</td><td>Beta</td><td>Mass-% of total cementitious materials</td><td>0.6</td><td>σ=0.15
A=0.2; B=2.0</td></tr>
<tr><td>Transfer Parameter</td><td>k_t</td><td>-</td><td>Assumes k_t as a constant value equal to 1.</td><td>Deterministic</td><td>-</td><td>1</td><td>-</td></tr>
<tr><td>Regression Variable</td><td>b_e</td><td>-</td><td>Recommends using a normal distribution with a mean value of 4800K and a standard deviation of 700K.</td><td>Normal</td><td>K</td><td>4800</td><td>700</td></tr>
<tr><td>Reference Time</td><td>t_0</td><td>-</td><td>Assumes t_0 as a constant value equal to 28 days = 0.0767 years.</td><td>Deterministic</td><td>years</td><td>0.0767</td><td>-</td></tr>
<tr><td>Standard Test Temperature</td><td>T_{ref}</td><td>-</td><td>Defines T_{ref} to be constant with a value of 293K (= 20^{o}C).</td><td>Deterministic</td><td>^{o}C
(^{o}F)</td><td>20
(68)</td><td>-</td></tr>
</table>

Table 1-5: Summary of Input Parameters for the Modeling of Chloride-induced Corrosion. The Temperature, T_{real}, the Initial Chloride Content, C_0, and the Critical Chloride Content, C_{crit}, are not Shown Because these Follow the Distributions in Table 1-4 for all Structural Elements.

Structural Element	Description	Exposure zone	Cover, c			Surface Concentration, $C_{S,\Delta x}$ [mass-% of cem. matl]			Chloride Migration Coefficient, $D_{RCM,0}$ [x 10^{-12} m^2/s]					Aging Factor, a [-]					Transfer Function, Δx [mm]		
									Distr.	OPC		OPC+20-35%FA		Distr.	OPC		OPC+20-35%FA				
			Distr.	Mean	Std. dev.	Distr.	Mean	Std. dev.		Mean	Std. dev.	Mean	Std. dev.		Mean	Std. dev. and function parameters	Mean	Std. dev. and function parameters	Distr.	Mean	Std. dev. and function parameters
Tangent piles	Tangent piles	Buried	Normal	64 mm (2.5 in.)	15.2 mm (0.6 in)	Lognormal	0.5	0.25	Normal	To be calculated		To be calculated		Beta	0.3	0.12 A=0; B=1.0	0.6	0.15 A=0; B=1.0	Deterministic	0	-
Piers	Pile cap	Buried	Normal	76 mm (3.0 in.)	15.2 mm (0.6 in.)	Lognormal	0.5	0.25	Normal	To be calculated		To be calculated		Beta	0.3	0.12 A=0; B=1.0	0.6	0.15 A=0; B=1.0	Deterministic	0	-
	Bottom part of column	Direct de-icing salts	Normal	76 mm (3.0 in.)	15.2 mm (0.6 in.)	Lognormal	4.0	2.0	Normal	To be calculated		To be calculated		Beta	0.3	0.12 A=0; B=1.0	0.6	0.15 A=0; B=1.0	Beta	8.9	5.6 A=0; B=50
	Column and pier cap	Indirect de-icing salts	Normal	76 mm (3.0 in.)	15.2 mm (0.6 in.)	Lognormal	2.0	1.0	Normal	To be calculated		To be calculated		Beta	0.3	0.12 A=0; B=1.0	0.6	0.15 A=0; B=1.0	Beta	8.9	5.6 A=0; B=50
Abutments	Wing wall	Buried	Normal	64 mm (2.5 in.)	15.2 mm (0.6 in.)	Lognormal	0.5	0.25	Normal	To be calculated		To be calculated		Beta	0.3	0.12 A=0; B=1.0	0.6	0.15 A=0; B=1.0	Deterministic	0	-
	Abutment wall	Indirect de-icing salts	Normal	76 mm (3.0 in.)	15.2 mm (0.6 in.)	Lognormal	2.0	1.0	Normal	To be calculated		To be calculated		Beta	0.3	0.12 A=0; B=1.0	0.6	0.15 A=0; B=1.0	Beta	8.9	5.6 A=0; B=50
Cast-In-Place Deck	Top of the deck	Direct de-icing salts	Normal	70 mm (2.75 in.)	7.6 mm (0.3 in.)	Lognormal	4.0	2.0	Normal	To be calculated		To be calculated		Beta	0.3	0.12 A=0; B=1.0	0.6	0.15 A=0; B=1.0	Beta	8.9	5.6 A=0; B=50
	Underside of the deck	Atmospheric	Normal	44 mm (1.75 in.)	7.6 mm (0.3 in.)	Lognormal	1.5	0.75	Normal	To be calculated		To be calculated		Beta	0.65	0.15 A=0; B=1.0	0.65	0.15 A=0; B=1.0	Deterministic	0	-
Barriers (50 years)	Barriers	Direct de-icing salts	Normal	63.5 mm (2.5 in.)	7.6 mm (0.3 in.)	Lognormal	4.0	2.0	Normal	To be calculated		To be calculated		Beta	0.3	0.12 A=0; B=1.0	0.6	0.15 A=0; B=1.0	Beta	8.9	5.6 A=0; B=50

Additional notes on chloride-surface concentrations ($C_{s,\Delta x}$):

With a full probabilistic methodology, all input variables, such as surface chloride concentration, are expressed as probability functions. The appropriateness of this approach can be appreciated by observing the wide variation in chloride concentrations that are frequently determined from coring a particular bridge deck; a single value could not realistically represent such variation of observations. Any testing program for determining the appropriate probabilistic input for chloride exposure on the new bridge would therefore require numerous samples. In addition, the samples would need to be taken on a structure that has been exposed to similar conditions to which the new bridge will be exposed. This is sometimes difficult to establish.

The choice for the surface chloride concentrations can be based on published data from multiple field testing programs undertaken by others as well as data obtained from the Owner for other similar structures in a similar environment.

For this case study, it is assumed that chloride surface concentration data for bridge decks were obtained from four nearby structures as shown in Table 1-6.

Table 1-6: Chloride Surface Concentration Data Obtained from Nearby Structures.

Author	Mean $C_{s,\Delta x}$ (kg/m^3)	Mean $C_{s,\Delta x}$ (% w/w_{cm})	Comments
Hooton et al. [20]	10.1	2.2	NYSDOT Bridge (455 kg/m^3)
	18.2	4.8	NYSDOT Bridge (380 kg/m^3)
	15.9	4.0	NYSDOT Bridge (400 kg/m^3)
	15.7	4.1	NYSDOT Bridge (380 kg/m^3)

Calculating the average of the values presented in Table 1-6 provides an average chloride surface concentration of 3.8% w/w_{cm}. Conservatively, a mean value of 4.0% w/w_{cm} is chosen for the service life design of the topside of the bridge deck. Due to the small sample size, the large scatter observed among the sample, and the known variability related to chloride levels in bridge decks, a coefficient of variation (COV) equal to 50% is assumed. With the chosen average value, this provides a standard deviation of 2.0% w/w_{cm} for the topside of the bridge deck.

D1.4.1.2 Chloride-induced Modeling for Concrete in Deck

- This example considers the concrete used for the deck exposed directly to de-icing salts.
- Two combinations of cementitious materials are considered: 'OPC' and 'OPC+20-35%FA' as defined in Section 4.1.1.
- A Monte Carlo simulation with 50,000 runs is performed to determine the required chloride mitigation coefficient for both mix designs to obtain a reliability index of 1.3.

- A spreadsheet for the performance of such full probabilistic modeling with 5,000 runs can be downloaded from the SHRP2 website (additional runs can be added by the user):

https://www.fhwa.dot.gov/goshrp2/Solutions/Renewal/R19A/Service_Life_Design_for_Bridges

D1.4.1.3 Concrete mix OPC+20-35%FA used in deck exposed to direct de-icing salts

- Input to spreadsheet based on values in Table 1-4:

	INPUT PARAMETERS			**Normal Distr Coefficients**			**Log-Normal Distr Coeffs**		**Beta Distr Coeffs**			
Parameter	**Description**	**Units**	**Distribution Function**	**Mean, μ**	**Std Dev, σ**	**Coeff of Variation, σ/μ**	$\ln\mu - \ln((\sigma/\mu)^2+1)/2$	$\sqrt{\ln((\sigma/\mu)^2+1)}$	**Lower Bound, a**	**Upper Bound, b**	**α**	**β**
		in^2/yr		0.340	0.068	0.20						
	Chloride Migration Coefficient (from Nordtest NT	mm^2/yr		219.4	43.9							
$D_{RCM,0}$	Build 492 - results are given in m^2/sec)	m^2/sec	Normal	6.95E-12	1.39E-12							
b_e	Regression variable, (limited to 3500 °K to 5500 °K)	°K	Normal	4800	700							
		°F		52.7	3.60							
		°C		11.5	2.00							
T_{real}	Temperature (from Local Weather Data)	°K	Normal	284.65	2.00							
		°F		67.6								
		°C		19.8								
T_{ref}	Standard test temperature	°F	Constant	292.9								
k_e	Environmental transfer variable	n/a	n/a									
k_t	Transfer parameter	n/a	Constant	1.0								
α	Ageing exponent - PCC w/ ≥ 20% Flyash	n/a	Beta	0.6	0.15				0	1	5.80	3.87
t_o	Reference point of time (28 days = 0.0767 yrs)	yrs	Constant	0.0767								
A(t)	Ageing function	n/a	n/a									
C_o	Initial Chloride Content of Concrete	mass% of binder	Normal	0.10	0.00	0.001						
C_s or $C_{s,\Delta x}$	Chloride Concentration at surface, or at substitute surface Δx	mass% of binder	Log-Normal	4.00	2.00	0.50	1.3	0.47				
		in		0.35	0.22	0.629			0	1.97		
Δx	Transfer function - splash/spray zone	mm	Beta	8.90	5.60				0	50	1.90	8.77
		in		2.75	0.30							
cover, a	Concrete cover	mm	Normal	69.85	7.62							
C_{crit}	Critical chloride content (plain reinforcing)	mass% of binder	Beta	0.60	0.15	0.25			0.2	2	5.31	18.58
t_{SL}	Design service life	yrs	n/a	75								
β	Target Reliability	n/a	n/a	1.3								

- Output from spreadsheet showing the last six simulations and the results, based on the input parameters listed above (note that the spreadsheet downloaded from the SHRP2 website has been expanded to include 50,000 simulations):

SHRP2 SOLUTIONS
STRATEGIC HIGHWAY RESEARCH PROGRAM

Trial Results of Randomly Generated Values of Input Parameters to Fick's 2nd Law

	$D_{RCM,0}$ (mm²/yr)		b_e (°K)		T_{real} (°K)		k_e	α		$A(t_{SL})$	$D_{app,C}$ (mm²/yr)	C_o (mass% of binder)		$C_{s,\Delta x}$ (mass% of binder)		Δx (mm)		cover (mm)		C_{crit} (mass% of binder)		$C(x=cov,t_{SL})$	Pass (1)
Trial	rand 0-1	RESULT	rand 0-1	RESULT	rand 0-1	RESULT		rand 0-1	RESULT			rand 0-1	RESULT	rand 0-1	RESULT	rand 0-1	RESULT	rand 0-1	RESULT	rand 0-1	RESULT	RESULT	/Fail (0)
49995	0.882	271.33	0.368	4564	0.195	282.9	0.6	0.164	0.446	0.0463	7.25	0.25	0.10	0.850	5.836	0.394	6.437	0.835	77.3	0.530	0.598	0.28	1
49996	0.059	150.90	0.449	4710	0.241	283.2	0.6	0.300	0.521	0.0277	2.41	0.07	0.10	0.047	1.626	0.009	0.647	0.547	70.8	0.263	0.495	0.10	1
49997	0.767	251.27	0.036	3540	0.348	283.9	0.7	0.600	0.648	0.0116	1.98	0.71	0.10	0.590	3.983	0.395	6.456	0.037	56.2	0.746	0.693	0.12	1
49998	0.501	219.47	0.440	4694	0.406	284.2	0.6	0.087	0.385	0.0705	9.45	0.04	0.10	0.909	6.721	0.768	12.575	0.620	72.2	0.951	0.870	0.85	1
49999	0.504	219.82	0.466	4740	0.084	281.9	0.5	0.012	0.255	0.1722	20.10	0.97	0.10	0.859	5.947	0.285	5.046	0.513	70.1	0.754	0.697	1.48	0
50000	0.461	215.11	0.318	4468	0.573	285.0	0.7	0.311	0.526	0.0267	3.76	0.69	0.10	0.803	5.356	0.220	4.205	0.946	82.1	0.742	0.690	0.11	1
SUMMARY																							
Computed Mean		219.31		4749		284.6	0.6		0.60	0.03	3.83		0.10		4.01		8.87		69.83		0.60		
Input Mean		219.35		4800		284.7			0.60				0.10		4.00		8.90		69.85		0.60		
Max		405.53		5500		293.18	1.01		0.99	0.67	109.29		0.10		23.19		36.94		101.73		1.44		
Min		12.32		3500		276.10	0.34		0.06	0.00	0.03		0.10		0.37		0.03		41.14		0.24		
																		Total Passing			45872		
																		Total # of Trials			50000		
																		Reliability			0.92		
																		P_f, Probability of failure			0.08		
																		β, Reliability Index (calculated)			1.388	Passes	
																		β, Target Reliability Index			1.3		

- The reliability index is greater than 1.3 for a maximum allowable chloride migration coefficient of 7 x 10^{-12} m^2/s.

D1.4.1.4 Concrete mix OPC used in deck exposed to direct de-icing salts

- Input to spreadsheet based on values in Table:

INPUT PARAMETERS

				Normal Distr Coefficients			Log-Normal Distr Coeffs		Beta Distr Coeffs			
Parameter	Description	Units	Distribution Function	Mean, μ	Std Dev, σ	Coeff of Variation, σ/μ	$\ln\mu - \ln((\sigma/\mu)^2+1)/2$	$\sqrt{\ln((\sigma/\mu)^2+1)}$	Lower Bound, a	Upper Bound, b	α	β
$D_{RCM,0}$	Chloride Migration Coefficient (from Nordtest NT Build 492 - results are given in m²/sec)	in²/yr		0.064	0.013	0.20						
		mm²/yr		41.0	8.2							
		m²/sec	Normal	1.30E-12	2.60E-13							
b_e	Regression variable, (limited to 3500 °K to 5500 °K)	°K	Normal	4800	700							
T_{real}	Temperature (from Local Weather Data)	°F		52.7	3.60							
		°C		11.5	2.00							
		°K	Normal	284.65	2.00							
T_{ref}	Standard test temperature	°F		67.6								
		°C		19.8								
		°F	Constant	292.9								
k_e	Environmental transfer variable	n/a	n/a									
k_t	Transfer parameter	n/a	Constant	1.0								
α	Ageing exponent - Type I Portland Cement (PCC)	n/a	Beta	0.3	0.12				0	1	4.08	9.51
t_o	Reference point of time (28 days = 0.0767 yrs)	yrs	Constant	0.0767								
A(t)	Ageing function	n/a	n/a									
C_o	Initial Chloride Content of Concrete	mass% of binder	Normal	0.10	0.00	0.001						
C_s or $C_{s,\Delta x}$	Chloride Concentration at surface, or at substitute surface Δx	mass% of binder	Log-Normal	4.00	2.00	0.50	1.3	0.47				
Δx	Transfer function - splash/spray zone	in		0.35	0.22	0.629			0	1.97		
		mm	Beta	8.90	5.60				0	50	1.90	8.77
cover, a	Concrete cover	in		2.75	0.30							
		mm	Normal	69.85	7.62							
C_{crit}	Critical chloride content (plain reinforcing)	mass% of binder	Beta	0.60	0.15	0.25			0.2	2	5.31	18.58
t_{SL}	Design service life	yrs	n/a	75								
β	Target Reliability	n/a	n/a	1.3								

- Output from spreadsheet showing the last six simulations and the results, based on the input parameters listed above:

SHRP2 SOLUTIONS
STRATEGIC HIGHWAY RESEARCH PROGRAM

Trial Results of Randomly Generated Values of Input Parameters to Fick's 2nd Law

	$D_{RCM,0}$ (mm^2/yr)		b_e (°K)		T_{real} (°K)		k_e	α		$A(t_{SL})$	$D_{app,C}$ (mm^2/yr)	C_0 (mass% of binder)		$C_{s,\Delta x}$ (mass% of binder)		Δx (mm)		cover (mm)		C_{crit} (mass% of binder)		$C(x{=}cov,t_{SL})$	Pass (1)
Trial	rand 0-1	RESULT	rand 0-1	RESULT	rand 0-1	RESULT		rand 0-1	RESULT			rand 0-1	RESULT	rand 0-1	RESULT	rand 0-1	RESULT	rand 0-1	RESULT	rand 0-1	RESULT	RESULT	/Fail (0)
49995	0.813	48.24	0.984	5500	0.939	287.7	0.7	0.768	0.386	0.0699	2.40	0.05	0.10	0.980	9.396	0.852	14.885	0.019	54.0	0.925	0.832	0.46	1
49996	0.891	51.05	0.080	3815	0.037	281.1	0.6	0.059	0.127	0.4169	12.28	0.54	0.10	0.251	2.606	0.207	4.040	0.057	57.8	0.249	0.490	0.63	0
49997	0.749	46.46	0.364	4557	0.059	281.5	0.5	0.012	0.079	0.5796	14.35	0.01	0.10	0.019	1.347	0.822	13.953	0.871	78.5	0.649	0.646	0.30	1
49998	0.990	59.95	0.472	4750	0.314	283.7	0.6	0.244	0.209	0.2371	8.38	0.09	0.10	0.556	3.822	0.553	8.650	0.187	63.1	0.006	0.306	0.56	0
49999	0.621	43.50	0.835	5483	0.448	284.4	0.6	0.776	0.390	0.0681	1.69	0.41	0.10	0.874	6.153	0.759	12.373	0.646	72.7	0.975	0.929	0.10	1
50000	0.378	38.41	0.131	4015	0.011	280.1	0.5	0.177	0.185	0.2806	5.74	0.72	0.10	0.327	2.894	0.402	6.550	0.345	66.8	0.764	0.703	0.21	1
SUMMARY																							
Computed Mean		40.99		4749		284.6	0.6		0.30	0.17	4.43		0.10		3.98		8.93		69.87		0.60		
Input Mean		40.97		4800		284.7			0.30				0.10		4.00		8.90		69.85		0.60		
Max		73.13		5500		293.34	1.03		0.79	0.88	32.43		0.10		26.44		37.47		102.79		1.41		
Min		10.19		3500		276.67	0.34		0.02	0.00	0.07		0.10		0.44		0.05		33.48		0.23		
																		Total Passing			45271		
																		Total # of Trials			50000		
																		Reliability			0.91		
																		P_f, Probability of failure			0.09		
																		β, Reliability Index (calculated)			1.313	Passes	
																		β, Target Reliability Index			1.3		

- The reliability index is greater than 1.3 for a maximum allowable chloride migration coefficient of 1.3×10^{-12} m^2/s. It is, however, not possible to design an OPC concrete mix with such low chloride migration coefficient and therefore this concrete mix design will not be allowed for deck concrete.
- *fib* Bulletin 34 provides a summary of normally anticipated values for the chloride migration coefficient, $D_{RCM,0}$, for different types of cement, presented in Table 1-7.

Table 1-7: Normally Anticipated Values for the Chloride Migration Coefficient, $D_{RCM,0}$, for Different Types of Cement. Source: *fib* Bulletin 34 [4].

$D_{RCM,0}$ [x 10^{-12} m^2/s]	Equivalent Water-Cement Ratio*					
Cement type	0.35	0.40	0.45	0.50	0.55	0.60
OPC	NA	8.9	10.0	15.8	19.7	25.0
OPC + FA (k = 0.5)	NA	5.6	6.9	9.0	10.9	14.9
OPC + SF (k = 2.0)	4.4	4.8	NA	NA	5.3	NA
OPC + 66-80% GGBS	NA	1.4	1.9	2.8	3.0	3.4

* Equivalent w/c ratio, hereby considering FA or silica fume (SF) with the respective k-value (efficiency factor). The considered contents were: FA: 22 wt.-%/cement (wt.-%/c); SF: 5 wt.-%/c.

GGBS = ground granulated blast-furnace slag.

D1.4.2 Requirements for Concrete Mixes

- Table 1-8 shows a summary of the requirements to the different concrete mixed, based on the full probabilistic service life design.
- When a component is exposed to multiple exposure zones and deterioration mechanisms, the most severe exposure zones and deterioration mechanisms govern that component.
- All concrete mix designs will have a maximum allowed water-cementitious material ratio of 0.40 to achieve the service life.

Table 1-8: Summary of Requirements for Concrete Mixes.

Structural Element	Description	Cover, c		Governing Exposure Zones	Min. Compressive Strength (psi)	Cement (ASTM C150)	Type of Concrete and Max. Allowable Chloride Migration Coefficient NT BUILD492 at 28 days (x 10^{-12} m^2/s)		Plastic Air Content (%)	Freeze-thaw Tests		
		Specified (in.)	Construction Tolerance (in.)				OPC	OPC+20-35%FA		Spacing Factor (ASTM C457)	Durability Factor (ASTM C666)	Resistance to Scaling (ASTM C672)
Tangent piles	Tangent piles	2.5	1.0	Buried / Atmospheric	4500	Type II	15.0	10.0	6.0	≤0.008 in.	≥90	-
Piers	Pile cap	3.0	1.0	Buried	3500	Type II	15.0	10.0	4.5	≤0.008 in.	≥90	-
	Bottom part of column	3.0	1.0	Direct de-icing salts	5000	Type I-II	Not allowed	7.0	6.0	≤0.008 in.	≥90	-
	Upper part of column and pier cap	3.0	1.0	Indirect de-icing salts	5000	Type I-II	Not allowed	10.0	6.0	≤0.008 in.	≥90	-
Abutments	Wing wall	2.5	1.0	Buried / Atmospheric	4500	Type II	15.0	10.0	6.0	≤0.008 in.	≥90	-
	Abutment wall	3.0	1.0	Buried / Indirect de-icing salts	5000	Type I-II	Not allowed	10.0	6.0	≤0.008 in.	≥90	-
Cast-In-Place Deck	Top of the deck	2.75	0.5	Direct de-icing salts	5000	Type I-II	Not allowed	7.0	6.0	≤0.008 in.	≥90	≤3
	Underside of the deck	1.75	0.5	Atmospheric								
Barriers (50 years)	Barriers	2.5	0.5	Direct de-icing salts	5000	Type I-II	Not allowed	7.6	6.0	≤0.008 in.	≥90	≤3

D1.5 Mitigation Methods for Steel Components

- The main deterioration mechanism for buried steel and steel exposed to de-icing salts is corrosion. Mitigation methods may include:
    - Protective coatings (painting)
    - Concrete encasement
    - Cathodic protection
    - Use of special steel alloys
    - Increased steel area (corrosion allowance)
- Table 1-9 shows the mitigation methods identified for the steel components in this example.

Table 1-9: Mitigation Methods for Steel Components.

Steel component	Exposure zone	Corrosivity category ISO 12944-2 [2]	Mitigation method
Steel H-piles	Buried	Im3	Corrosion allowance
Girder	Indirect de-icing salts	C4	Painting
Decorative fence	Direct de-icing salts	C5-M*	Painting

* C5-M is the most severe environment and therefore conservatively assumed in this case due to the presence of de-icing salts.

D1.5.1 Corrosion Allowance

- AASHTO LRFD provides guidance to determine whether a site or soil should be considered corrosive; however, it does not provide guidance to estimate the level of corrosivity, the rate of corrosion or associated section loss.
- Table 1-10 summarizes corrosion allowances using different references.

Table 1-10: Total Corrosion Allowance for Buried Steel According to Different References.

References	Total Corrosion Allowance for 75 Years for Fully Buried H-Piles (2-Sided Exposure) in Different Exposure Zones		
	Slightly Aggressive	Moderately Aggressive	Extremely Aggressive
FHWA Design and Construction of Driven Piles Foundations, V1 – Section 6.12.1 [17]	-	-	0.45 in. for fill or disturbed natural soils*
FDOT Structures Design Guidelines – Section 3.1 [18]	0.075 in.	0.15 in.	0.225 in.
EN 1993-5, Eurocode 3: Design of Steel Structures, Part 5: Piling – Section 4.4 [19]	0.07 in. for undisturbed natural soils	0.18 in. for polluted natural soils and industrial sites	0.35 in. for non-compacted and aggressive fills

* A corrosion rate of 0.003 in. per year is stated in the reference and it is unclear if this a corrosion loss for one exposed face or two exposed faces. The corrosion allowance was conservatively doubled to consider both sides of the H piles.

D1.5.2 Paint Systems

- All steel located in direct and indirect de-icing salt zones will be painted to prevent corrosion as described in Table 1-9.
- For painted steel, the corrosion of the steelwork will be prevented as long as the paint is properly maintained. Therefore, the service life verification of painted elements is driven by the service life of the paint system.
- The primary reference used for estimating paint life is “Expected Service Life and Cost Considerations for Maintenance and New Construction Protective Coating Work”, National Association of Corrosion Engineers (NACE) paper 7422 [21].
- The NACE paper 7422 provides a long list of estimated practical lives for different coating systems. The practical life is defined as the time until touch-up painting is required. The actual end of service life is assumed to occur later when a full paint replacement is required.
- Table 1-11 shows a typical painting sequence as recommended by the NACE paper 7422.

Table 1-11: Typical Painting Sequence According to the NACE paper 7422.

Operation	Painting Occurs in Year
Initial painting	0
Touch-up	Practical life (P)
Maintenance repaint*	M = P x 133%
Full repaint	F = P x 183%

* Maintenance repaint is understood to mean touch-up paint plus a full overcoat.

- Estimated practical lives are provided for two different coating systems for corrosivity categories C3 and C5-M in Table 1-12 based on the information in the NACE Paper 7422. For category C4, the time of the practical life has been assumed to be the average of the times given for categories C3 and C5-M.

Table 1-12: Estimated Practical Life, P (years), of the Considered Paint Systems.

Paint System*	Surface Preparation**	No. of Coats	Min. DFT (mils)	Corrosivity Category ISO 12944-2 [2]		
				C3	C4	C5M
Inorganic Zinc/ Epoxy/ Polyurethane	Blast	3	11	21	18	15
Organic Zinc/ Epoxy/ Polyurethane	Blast	3	6	18	15	12

* Description is for the base system. Touch-up systems to be compatible with the base system.
** Blast requires SSPC-SP 6 "Commercial Blast" or SP 10 "Near White Blast".
DFT = Dry Film Thickness

- Based on the expressions for M and F in Table 1-11, the time until maintenance replacement and full repaint is determined for the different coated steel components as shown in Table 1-13. Optimization of the cycles for maintenance paint and full replacement could be based on a life-cycle analysis. Guidance is provided in NACE Paper 7422.

Table 1-13: Estimated Service Life to Maintenance Repaint, M, and Full Repaint. C4 and C5-M refer to the Steel Corrosivity Categories Defined in ISO 12944-2 [2].

Component	Paint system	Touch-up (years)		Maintenance Repaint (years)		Full Repaint (years)	
		C4	C5-M	C4	C5-M	C4	C5-M
Girder	Inorganic Zinc/ Epoxy/ Polyurethane	18	-	24	-	33	-
Decorative Fence		-	15	-	20	-	27
Girder	Organic Zinc/ Epoxy/ Polyurethane	15	-	20	-	27	-
Decorative Fence		-	12	-	16	-	22

- For the girders, regardless which paint system is chosen, the time until a full replacement of the paint system is greater than 25 years as required in Table 1-1.
- For the decorative fence, the inorganic zinc paint system meets the service life criteria of 25 years. However, when the organic zinc paint system is used, the time until full repaint is less than the minimum required paint service life and therefore this paint system is not allowed for the decorative fence. Alternatively, the service life of the organic system could be expended by increasing the number of touch-up and maintenance repaint cycles.

D1.6 Construction

Output and recommended design features and materials properties from the service life assessment are included in the Project Specifications.

Quality control (QC) and quality assurance (QA) during construction are essential to achieving the service life requirements. For concrete structures, this process typically 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. The Designer will review data sheets, mill reports, and aggregates source reports, to verify that the materials comply with the Project Specifications. Testing of constituent material properties will be performed if test data are missing.

  When the constituents are deemed to satisfy the requirements, a series of laboratory mixtures (trial batches) are completed using one or more of the proposed cementitious material combinations, and appropriate testing is done to demonstrate that all requirements are met. Note that several weeks of lead time are required for this process.

- Production and construction phase:

  During construction, the key properties such as compressive strength, plastic air content, and chloride migration coefficient should be monitored by testing samples obtained from production concrete. As-built concrete covers may also be measured. Measured values from the construction phase can be compared with design values to assess if the service life criteria will be met.

Other factors influencing the service life are subject to a rigorous QC: placement, finishing, and curing procedures for concrete structures; for coatings, these are surface preparation, application procedures, and monitoring procedures. The QC and QA of these operations should be described in the Project Specifications.

D1.7 References

[1] National Centers for Environmental Information, National Oceanic and Atmospheric Administration (NOAA). Accessed on April 11, 2018. https://www.ncdc.noaa.gov/cdo-web/

[2] ISO 12944-2. Paints and varnishes – Corrosion protection of steel structures by protective paint systems – Part 2: Classification of environments. Geneva, Switzerland: International Organization for Standardization (ISO); 2000.

[3] ACI 318-14. Building Code Requirements for Structural Concrete and Commentary. Michigan: American Concrete Institute (ACI); 2014.

[4] *fib* Bulletin 34. Model Code for Service Life Design. Lausanne, Switzerland: International Federation for Structural Concrete (*fib*); 1st edition. 2006. 126 pp.

[5] AASHTO R80. Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction. Washington DC: American Association of State Highway and Transportation Officials (AASHTO); 2017.

[6] ASTM C1260. Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method). West Conshohocken, PA: ASTM International; 2014.

[7] ASTM C1293. Standard Test Method for Determination of Length Change of Concrete Due to Alkali-Silica Reaction. West Conshohocken, PA: ASTM International; 2015.

[8] ASTM C295/C295M. Standard Guide for Petrographic Examination of Aggregates for Concrete. West Conshohocken, PA: ASTM International; 2012.

[9] ASTM C457-06. Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete. West Conshohocken, PA: ASTM International; 2011.

[10] ASTM C666/C666M-03. Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing. West Conshohocken, PA: ASTM International; 2008.

[11] ASTM C672/C672M-12. Standard Test Method for Scaling Resistance of Concrete Surface Exposed to De-icing Chemicals. West Conshohocken, PA: ASTM International; 2012.

[12] NT BUILD 492. Concrete, mortar and cement-based repair materials: Chloride migration coefficient from non-steady-state migration experiments. Finland: Nordtest; 1999, ISSN 0283–7153.

[13] ASTM C1218/C1218M. Standard Test Method for Water-Soluble Chloride in Mortar and Concrete. West Conshohocken, PA: ASTM International; 2017.

[14] ASTM C1152. Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete. West Conshohocken, PA: ASTM International; 2012.

[15] *fib* Bulletin 76. Benchmarking of deemed-to-satisfy provisions in standards. International Federation for Structural Concrete (*fib*), Lausanne, Switzerland, 2015. 189 pp.

[16] AASHTO LRFD Bridge Design Specification. Seventh Edition, 2014.

[17] FHWA Design and Construction of Driven Piles Foundations – Volume 1, Publication No. FHWA-NHI-16-009, July 2016.

[18] Florida Department of Transportation (FDOT). Structures Design Guidelines. FDOT Structures Manual, Volume 1, January 2017.

[19] EN 1993-5. Design of Steel Structures – Part 5: Piling. Brussels: European Standards; 2007.

[20] Hooton, R.D., Bentz, E.C., and Jojundic, T. "Long-term Chloride Penetration of Silica Fume Concretes Based on Field Exposure". Proceedings, Service Life Design for Infrastructure RILEM PRO 70, Vol.1, pp.503-512; 2010.

[21] NACE International paper 7422. Expected Service Life and Cost Considerations for Maintenance and New Construction Protective Coating Work. Houston, Texas: NACE (National Association of Corrosion Engineers); 2016.

D2 Example 2 – Service Life Design Report for a Bridge Substructure

Contents

D2.1 Introduction

Demonstrating the concepts of the durability assessment for concrete structures, a bridge (referred to as "the Bridge") located Mideast of the United States is used as an example. The Bridge consists of a cable-stayed structure with three towers and two anchor piers for a total length of 2,106 feet (ft), is located over a river, and is exposed to de-icing salts.

This document summarizes the durability assessment of the reinforced concrete foundations and substructure: drilled shafts, pile caps, towers and anchor piers. The durability assessment's objective is to determine the required concrete type, concrete quality and concrete cover in order to meet the required service life. This document is not meant to teach the reader how to do a durability assessment.

This durability assessment uses *fib* Bulletin 34, *Model Code for Service Life Design* [1]. The input parameters used in the *fib* model are expressed in International System of Units (SI units), and therefore, this report includes both English units and SI units, where applicable.

D2.2 Methodology

In accordance with the project requirements, the non-replaceable concrete substructure of the Bridge is designed for a 100-year service life.

The service life is defined as the time before rehabilitation is required and preservation activities (i.e., cyclical and condition-based maintenance) are expected during the service life. Preservation activities are typically planned and described in the Operations and Maintenance Manual.

For concrete structures, a two-phase service life model is generally used to represent the development over time of most types of deterioration mechanisms. There are a limited number of other potential deterioration mechanisms that arise from specific environmental events and are not characterized by the two-phase model. These other mechanisms are described in Section 2.4 of this report. The following describes the two-phase service life model:

- The Initiation Phase

  During this phase no noticeable weakening of the material or the function of the structures occurs. Aggressive substances from the surrounding environment penetrate into the concrete and either accumulate with time in the outer concrete layer (such as, sulfates) or alternatively, diffuse further inward, towards the reinforcement (such as, chlorides). Carbonation, chloride penetration, and sulfate accumulation accelerated by cyclic wetting and drying, are examples of such mechanisms.

- The Propagation Phase

  At the start of this phase, protective barrier(s) are broken down and/or critical levels of detrimental substances are reached, such that during the propagation phase an active deterioration develops, and loss of function commences. In many cases, deterioration mechanisms develop at an increasing rate with time.

The two-phase model of chloride-induced reinforcement corrosion is illustrated in Figure 2-1. The nominal service life is equal to the corrosion initiation time which is at the end of the initiation phase. This definition of the limit state is consistent with the Owner's objective of having concrete structures with minimal maintenance requirements over the service life.

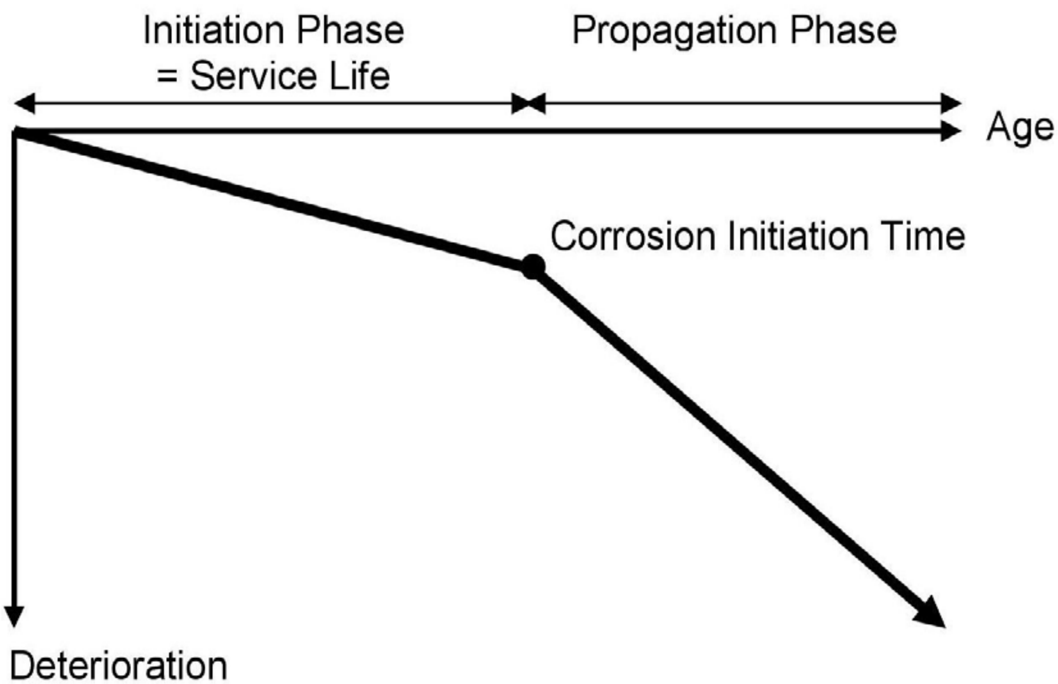


Figure 2-1: Two-Phase Modelling Approach of Deterioration Specific to Chloride-Induced Reinforcement Corrosion.

Three different design strategies for concrete structures have been adopted for the Bridge, in accordance with the approach of *fib* Bulletin 34 [1]:

- **Strategy A—**Avoid the potential degradation mechanism.
- **Strategy B—**Apply the deemed-to-satisfy method.
- **Strategy C—**Select material composition and structural detailing to resist, for the required period of time, the potential degradation mechanism.

Examples of Strategy A are selection of nonreactive or inert materials, such as nonreactive aggregates, sulfate resistant cements, low alkali cements, and stainless steel reinforcement.

Strategy B consists of application of codes or standards that have been developed to provide satisfactory performance based on statistical evaluation of data or long-term performance experience.

In contrast to Strategies A and B, Strategy C allows the deterioration but only to the extent that the service life limit state will not be reached within the design service life. Strategy C 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.
- Define the environmental conditions or loadings to be expected.
- Apply realistic modelling of the deterioration process to each structural member considering the local environment and material resistance to determine sufficient concrete cover thicknesses and performance criteria for concrete quality (that is, chloride migration coefficient).
- Based upon the performance criteria, perform compliance tests for QC purposes during preproduction and production (for example, chloride migration coefficient testing in the case of chloride-induced reinforcement corrosion).

The project requirements specify that the design methodology for service life following Strategy C uses a probabilistic approach based on *fib* Bulletin 34, *Model Code for Service Life Design*, with a target confidence level of 90 %. Strategy C will be implemented for reinforced concrete to limit the risk of initiation of chloride-induced reinforcement corrosion. Based on the deterioration modeling, the minimum concrete cover and the maximum chloride migration coefficient will be determined to ensure that the required service life can be obtained without major maintenance.

The deemed-to-satisfy method (Strategy B) will be implemented on the Bridge for freeze-thaw deterioration.

For other possible types of concrete deterioration such as sulfate attack, AAR, DEF, and leaching, the avoidance of deterioration approach (Strategy A) will be implemented.

Carbonation-induced reinforcement corrosion is not specifically considered as requirements for chloride-induced corrosion will prevail and protect the structure against carbonation.

D2.3 Exposure Conditions

The Bridge will be subject to multiple concrete deterioration processes. The severity of the various processes is influenced by the exposure conditions within the structure. Therefore, the structure is divided into exposure zones defined as follows:

- Atmospheric zone without de-icing salts
- Atmospheric zone with severe de-icing salts

- Splash zone/atmospheric zone with moderate de-icing salts
- Submerged/buried zone

The exposure zones and deterioration mechanisms are listed in Table 2-1 and have been defined for the different parts of the bridge structure. To facilitate the identification of the different zones, a color code is provided. Figure 2-2 presents the exposure zones for the drilled shafts, pile caps and towers. Figure 2-3 presents the exposure zones for the anchor piers.

All concrete, except for the drilled shafts, will be exposed to atmospheric oxygen (O_2) and carbon dioxide (CO_2), some temperature and humidity variations, and freeze-thaw cycles.

The drilled shaft concrete above the rock surface will be permanently protected by a steel casing that is 1 in. (25 mm) thick, which will be submerged in fresh water or embedded in soil. Because of the construction process, the steel reinforcement has a nominal concrete cover of 6 in. (152 mm) within the steel casing and 3 in. (76 mm) below the steel casing (that is, in the rock socket). It is reasonable to assume that the concrete within the steel casing will be sufficiently protected from deterioration mechanisms such as sulfate attack, freeze-thaw damage, leaching, carbonation and chloride-induced reinforcement corrosion for the entire 100 years of service life. The portion of the drilled shaft within the rock will be similarly protected.

The pile caps and pier columns at the anchor piers are in a splash zone exposed to fresh water, where exposure to sulfate and risk of leaching may occur.

In addition, for the pile caps and tower pedestals, there is a possibility of some chlorides originating from the bridge deck runoff. Because there are full height solid barriers at the roadway edges and there are no deck expansion joints at the towers, there will be no bridge deck drains in close proximity to the towers. Therefore, the risk of deck runoff contacting the pile caps and tower pedestals is low. However, because of the height of the deck above the pile caps, some runoff from the open deck drains could occasionally be blown onto the foundation elements. To ensure the desired 100-year service life of the pile caps and tower pedestals, this possibility has been taken into account by assuming a moderate exposure of these elements to de-icing salts.

The chloride loading for the tower concrete located above the pedestals, below the deck level, as well as more than 35 ft above the deck level, is assumed to be very low with only some airborne chlorides. Deck drains will be located sufficiently far enough away from the towers that drainage will not be blown onto the tower legs. Should rain wash chlorides from deck spray down the tower legs, the chloride content will be diluted and only minor chloride concentrations will result at the concrete surface. This is consistent with typical bridge towers where usually no corrosion is observed in these areas.

The area of the towers near the deck will be exposed to spray containing chlorides from de-icing salts. These surfaces will be subject to alternate cycles of wetting and drying. This exposure zone is starts where the towers become hollow, approximately 15 ft below the deck surface. This exposure zone is defined to extend up to 35 ft above the top of the deck surface.

The interior surfaces of the towers are located in the atmospheric zone without de-icing salts and will be protected from rain and airborne chlorides.

Expansion joints located above the anchor piers are designed to prevent deck runoff from contacting the piers; however, it is recognized that the joints may at times leak and result in chlorides being transported from the deck to the pier caps. Because of this possibility, the entire pier is assumed to be exposed to a moderate chloride environment from occasional exposure to de-icing salts. This is consistent with observations of corrosion on typical bridge piers located near deck joints.

Table 2-1: Classification of Exposure Conditions and Deterioration Mechanisms for the Different Reinforced Concrete Elements.

Exposure Zone	Elements	Exposure Conditions	Potential Deterioration Mechanisms						
			Materials	Environmental					
			AAR	DEF	Sulfate	Freeze-Thaw	Leaching	Carbonation-induced	Chloride-induced
Atmospheric zone without de-icing salts	Towers (exterior surfaces) more than 35 ft above or more than 15 ft below bridge deck level.	Atmospheric O_2 and CO_2. Some limited airborne chlorides. Some temperature and humidity variations, including freeze-thaw.	x	x		x		x	x
	Towers (interior surfaces)	Atmospheric O_2 and CO_2. Some temperature and humidity variations.	x	x				x	
Atmospheric zone with severe de-icing salts	Towers (exterior face) at bridge deck level up to 35 ft above deck and 15 ft 4 inches below deck.	Chlorides from de-icing salts with alternating wetting and drying. Atmospheric O_2 and CO_2. Freeze-thaw.	x	x		x		x	x
Splash zone / atmospheric zone with moderate de-icing salts	Pile caps Tower pedestals Piers	Alternating wetting and drying with river water. Limited chlorides from de-icing salts and river water. Sulfates in the river water. Atmospheric O_2 and CO_2. Freeze-thaw.	x	x	x	x	x	x	x
Submerged/ buried zone	Reinforced concrete in steel casings and rock socket								

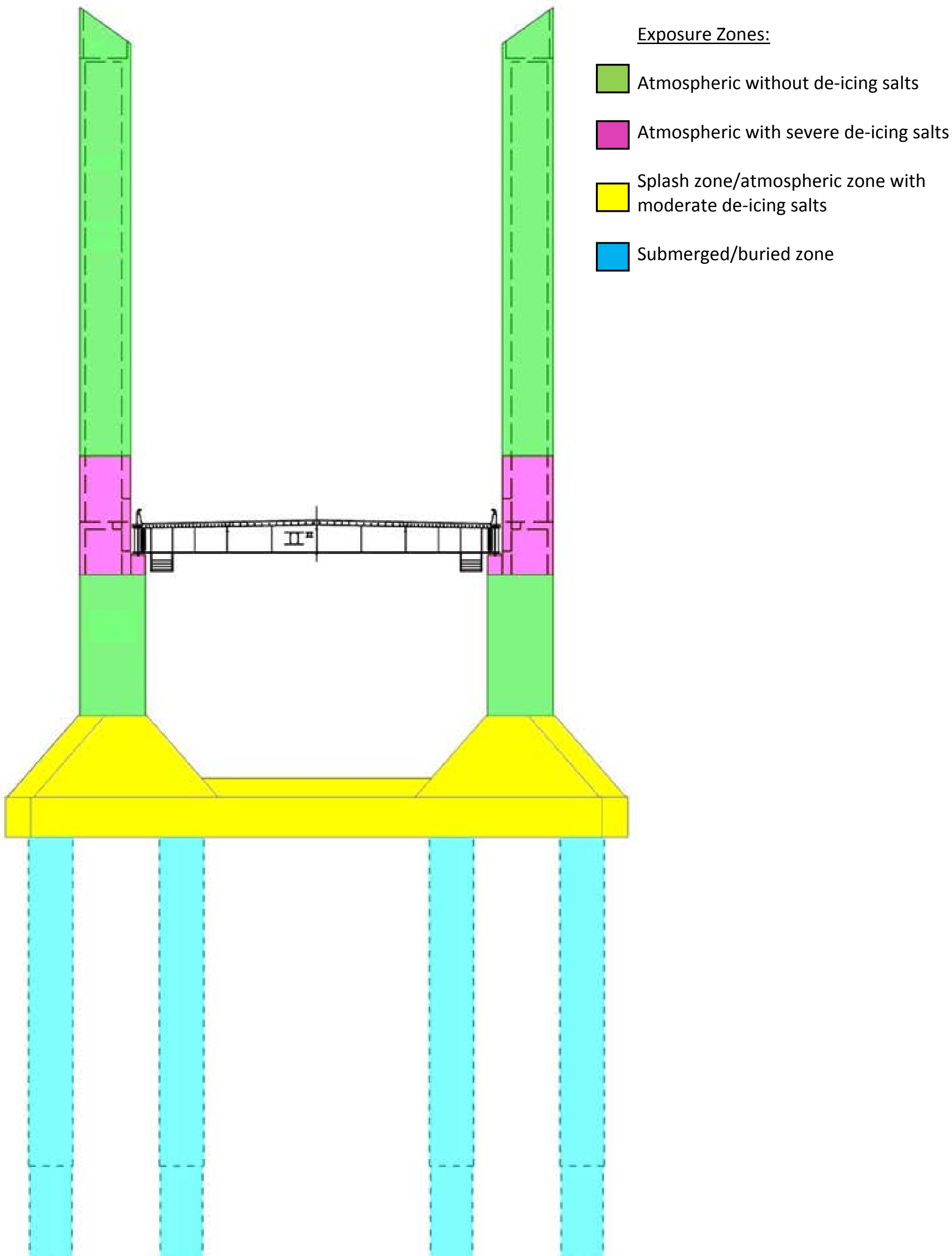


Figure 2-2: Typical Exposure Zones for the Towers.

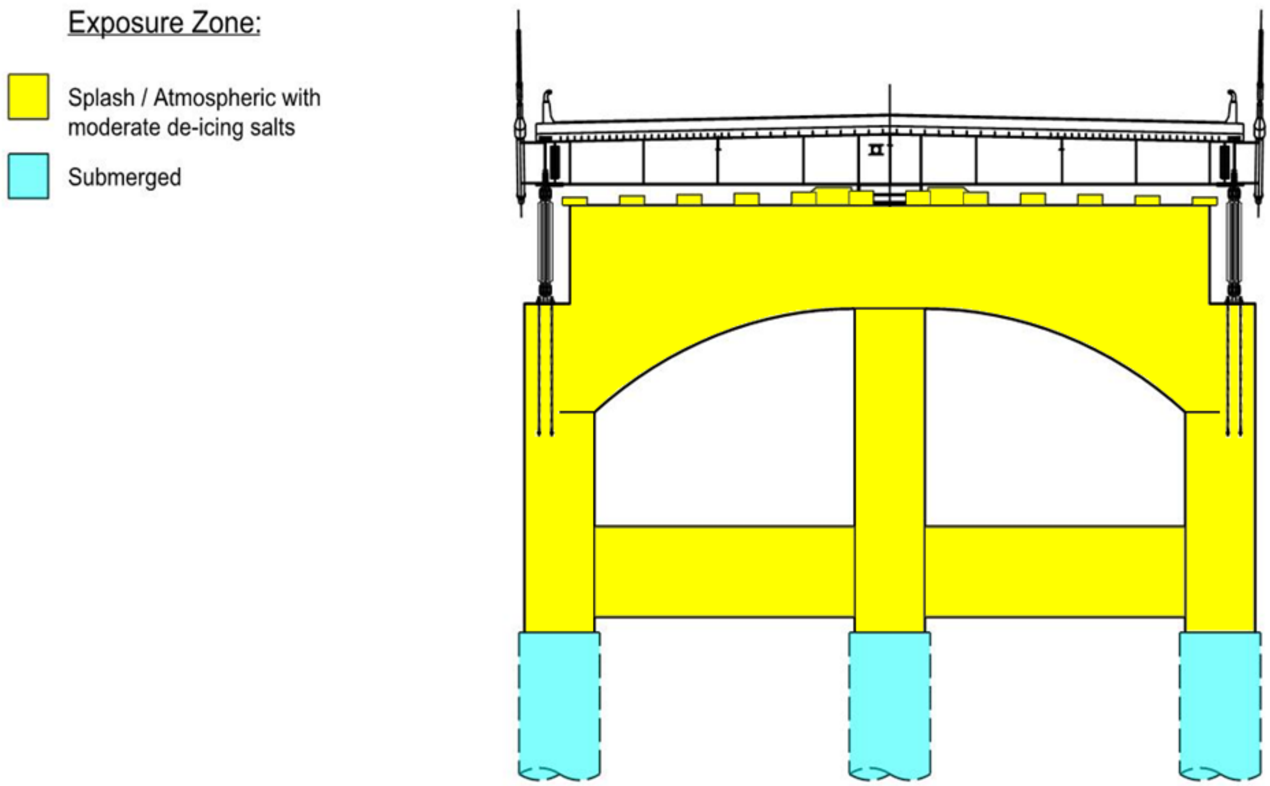


Figure 2-3: Typical Exposure Zones for the Anchor Piers.

D2.4 Deterioration Assessment and Mitigation Measures

The following deterioration mechanisms were identified for the various exposure conditions of the reinforced concrete: AAR, sulfate attack, DEF, freeze-thaw, leaching, carbonation induced-corrosion and chloride-induced reinforcement corrosion.

The avoidance of deterioration approach (Strategy A) is implemented for the following concrete deterioration mechanisms: sulfate attack, AAR, DEF, and leaching. Freeze-thaw is addressed by the deemed-to-satisfy method (Strategy B). Assessment of chloride-induced corrosion is based on a probabilistic approach (Strategy C) and is addressed in Section 2.5. Carbonation-induced corrosion is not specifically addressed as requirements for chloride-induced corrosion will prevail.

Deterioration mechanisms applicable to each structural element and exposure zone are identified in Table 2-1. Because of the lack of a comprehensive document for durability requirements, durability requirements are reviewed based on ACI 318, and AASHTO R80-17, when applicable. The local Standard Specifications are reviewed to ensure they adequately address each deterioration mechanism. Additional requirements are specified to supplement the local Standard Specifications in some instances. A summary of the different documents consulted for each deterioration mechanisms is as follows:

- AAR: AASHTO R80-17
- Sulfate attack: ACI 318
- DEF: project specifications
- Freeze-thaw: ACI 318

D2.4.1 Alkali-Aggregate Reaction

AAR are reactions within hardened concrete where active components found in certain types of susceptible aggregates and alkali hydroxides (found mainly in cement) react to form an expansive gel, which may lead to cracking of the concrete. Moisture must be available for AAR to proceed and must be below about 80 % internal relative humidity or the reaction will cease [23].

Aggregates shall be from approved material sources by the local authorities. Potential reactive aggregates will be addressed through the provisions of AASHTO R80-17 [24]. Risk of AAR is minimized by selecting a nonreactive aggregate as defined in AASHTO R80-17 using ASTM C 1260 or selection of adequate preventative measures in accordance with AASHTO R80-17.

D2.4.2 Sulfate Attack

Sulfate attacks occur when an external sulfate source (such as water surrounding the structure, sulfate bearing soils, or improper sulfate-containing aggregates), causes expansive reactions that result in cracking and ultimately disintegration of the concrete structure. Expansive sulfate reactions are seen for concrete containing cement with a moderate to high C_3A-content in case of high sulfate content in the soil and groundwater.

Concrete exposed to the river water and soil is potentially subject to sulfate attack (pile caps, piers, and scour area of the drilled shafts). Investigations show that the water soluble sulfate content in the local soil and water are not sufficient to cause sulfate attack to the concrete [3]. The exposure conditions remain low with exposure category S1 in accordance with ACI 318 [3].

For exposure category S1, the w/c ratio should be below 0.50 and a minimum concrete compressive resistance of 4,000 psi (28 MPa) should be provided (ACI 318). Both of these parameters will be met.

D2.4.3 Delayed Ettringite Formation

DEF is a form of internal sulfate attack, which can be affected by concrete composition, curing conditions, and exposure conditions. Mineral ettringite, which is not harmful to concrete, is commonly formed at an early age when concrete is cured at ambient temperature. If temperatures are high during curing, the formation of ettringite is delayed, and its gradual formation in a cooled, hardened concrete can lead to expansion and cracking. Risk of DEF is reduced through proper temperature control during concrete placement and curing. This can be assisted by the use of FA and/or GGBS cement.

DEF is relevant for foundations and substructure. DEF can be avoided by limiting the internal temperature of the concrete during the hardening phase to 160 ^{o}F (71 ^{o}C). This requirement has been written into the project Standard Specifications for Structural Mass Concrete.

For the Bridge, mass concrete is considered any concrete placement, excluding drilled shafts, with a plan dimension at least 7 ft or greater. Project-specific Thermal Control Plans are required for all mass concrete and these plans will include provisions to limit the maximum temperature of curing concrete to 160^{o}F.

Temperature requirements stated in the local Standard Specifications are applicable to all other concrete placements. In addition, measures will be taken to ensure that the maximum internal temperature of all concrete during the hardening phase will be limited to 160^{o}F (71^{o}C).

D2.4.4 Freeze-Thaw

All parts of the concrete structure above the water level will be exposed to freeze-thaw cycles. This includes all concrete mixes except the drilled shafts. Freeze-thaw cycles 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. The frost resistance of concrete depends on the mix design and concrete permeability: concrete with high water content and high w/c ratio is less resistant. The presence of de-icing salts can lower the freeze-thaw resistance of the concrete. Damage from freezing and thawing can be avoided by using freeze-thaw resistant aggregates and providing air-entrainment in the concrete.

Concrete exposed to freeze-thaw cycles and in continuous contact with moisture is classified as exposure category F2 by ACI 318. If chlorides are present, the exposure category is F3. A maximum w/c ratio of 0.45 and a minimum compressive strength of 4,500 psi (31 MPa) are recommended by ACI 318 for both categories of exposure. The recommended air content of fresh concrete varies based on the nominal maximum aggregate size; 6 % is recommended for aggregate sizes of 0.75 in. to 1.0 in. (19 to 25 mm) [3]. This is consistent with the local Standard Specifications where a fresh concrete air content of 6 % ± 2 % is specified for all concrete classes and which should be sufficient to mitigate the effects of freeze-thaw action.

Requirements related to corrosion are more stringent than for freeze-thaw and a maximum w/c ratio of 0.40 is recommended as explained in Section 5.

Only Class F FA will be used. The local Standard Specification limits the Class F FA content to 20 % by mass of total cementitious materials. AASHTO LRFD [4] and ACI 318 [3] would allow this limit to be raised to 25 % by mass of total cementitious material. It is recommended not to go over 35 %, as higher amounts of FA can have a negative effect on the freeze-thaw resistance of concrete exposed to de-icing salts.

The use of GGBS is not planned; however, if its use is later found desirable, the GGBS will meet ASTM C989 requirements as stated by the local Standard Specifications. The local Standard Specification limits the GGBS content to 30 % by mass of total cementitious materials. ACI 318 allows this limit to be raised to 50 % by mass of total cementitious material [3].

In addition to meeting the local Standard Specifications, the air-void system will be tested in accordance with ASTM C457 Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete, using a magnification factor between 100 and 125. The air-void system of the concrete mix will be considered satisfactory when the average of all tests shows a spacing factor not exceeding 0.008 inches, with no single test greater than 0.010 inches, and air content greater than or equal to 3.0 % in the hardened concrete. For concrete with a water-to-cementitious materials ratio of 0.36 or less, the average spacing factor will not exceed 0.0098 inches, with no single value greater than 0.0118 inches.

D2.4.5 Leaching

Leaching is not normally a problem for good quality concrete. However, if water penetrates the concrete through cracks, it may dissolve various minerals present in the cement paste (such as calcium hydroxide) or in the aggregates. The dissolved ions are leached out and transported to other locations where different conditions may cause the precipitation of these minerals. This phenomenon can create deposits or efflorescence inside cracks and at the concrete outer surface. Leaching increases the porosity of the concrete and hence reduces strength and stiffness.

Leaching should not be an issue as a low permeability concrete in conjunction with good workmanship as specified in the local Standard Specifications will be provided to mitigate other deterioration mechanisms, such as freeze-thaw cycles and chloride-induced reinforcement corrosion.

D2.4.6 Carbonation-Induced Reinforcement Corrosion

Carbonation is caused by CO_2 from air penetrating the concrete and reacting with calcium hydroxide to form calcium carbonate. This is a slow and continuous process that lowers the alkalinity of the concrete, which is essential for corrosion protection. When the pH decreases, the steel passivation layer is dissolved and corrosion can occur if moisture and oxygen are present. Carbonation-induced reinforcement corrosion leads to uniform corrosion around the steel reinforcement and usually develops later and at slower rates than chloride-induced corrosion.

Mitigation methods for carbonation-induced reinforcement corrosion include low concrete permeability and adequate concrete cover. Hence, for structural elements exposed to chlorides, mitigation methods for chloride-induced reinforcement corrosion also prevents carbonation-induced corrosion.

For bridge structures, this deterioration mechanism is not critical for most components since chloride-induced reinforcement corrosion typically prevails in most locations.

D2.5 Service Life Modeling: Chloride-induced Reinforcement Corrosion

The Bridge will be subject to the use of de-icing chemicals and the main deterioration mechanism for the reinforced concrete substructure components will be chloride-induced reinforcement corrosion.

Chloride ions (Cl^-) can penetrate the concrete, eventually disrupt the passive layer around the steel reinforcement, and cause initiation of reinforcement corrosion. Chloride-induced reinforcement corrosion can occur within a relatively short period depending on the concrete properties, concrete cover thickness, and exposure conditions. From the resulting reinforcement corrosion from Cl^- exposure, pitting corrosion is considered the most aggressive and dangerous form of corrosion.

Cracks with excessive widths can also affect the protection offered by concrete. Design and detailing will be in accordance with the applicable design codes to limit crack widths to prevent excess chloride contamination through cracks. A performance-based service life approach, *fib* Strategy C, is used to address chloride-induced reinforcement corrosion for the substructure, including towers. The *fib* Model Code has been selected as the service life design model. Key input parameters are quantified as probabilistic distributions. Based on this methodology, the probabilistic nature of the input parameters (both the material resistances and the environmental stresses) and intrinsic model uncertainties are taken into account. The material resistance parameters (for example, chloride migration coefficient) considered in the modeling are chosen from literature data for comparable projects. Achievement of the required 28-day chloride migration coefficients selected for the project will be verified through laboratory testing of the concrete as stated in Section 2.6.

The limit state is corrosion initiation with a confidence level of 90 % that corrosion will not be initiated within the targeted service life. This corresponds to a reliability index of 1.3.

The following sections explain the exposure conditions, input parameters, and results of the probabilistic analyses for the reinforced concrete elements subject to chloride-induced reinforcement corrosion.

D2.5.1 Quantification of Input Parameters

Data used as input parameters for the *fib* Model Code modelling have been adopted based on results of the DuraCrete/Durable and Reliable Tunnel Structures investigations [2],[7], extensive literature research, other data from existing structures and engineering judgment accounting for details of the specific structure, materials, and the prevailing environmental conditions.

As a starting point, it is assumed that the concrete mixes will consist of Portland Cement with 20 % to 25 % Class F FA by mass of total cementitious materials for all structural elements and exposure conditions other than drilled shafts.

Although GGBS use is not planned in the concrete, consideration of Portland Cement with 30 % to 45 % GGBS by mass of total cementitious materials has been included in the discussion for the splash zone (pile caps, piers, tower pedestals) to illustrate the difference in requirements that would apply if GGBS was used. The local Standard Specifications state that a maximum of 30 % GGBS grade 120 or grade 100 may be used in the concrete mix. As a reference, ACI 318 limits the content of GGBS to 50 % by mass of cementitious materials. In the event that GGBS is used on this project, it will conform to ASTM C989 Grade 100 or 120. GGBS content in excess of 30 % is subject to acceptance by the local authorities.

The concrete mix for the drilled shafts contains 45 % Class F FA by mass of total cementitious materials. The local Standard Specifications state that a maximum of 20 % Class F FA or 30 % Class C FA by mass of cementitious materials may be used in the concrete mix. As a reference, ACI 318 limits the content of FA to 25 % by mass of cementitious materials. These limits are primarily based on considerations that do not apply to the drilled shafts environment; therefore, in agreement with the local authorities, the limits have been waived. The high FA content aids in limiting the temperature rise in the fresh concrete. Other cements/cementitious material combinations could be considered. However, Portland Cement alone will not provide a sufficient chloride diffusion resistance for chloride-dominating exposure and 100 years of service life. A triple-blend mix of ordinary Portland Cement, FA, and SF is not preferred because of potential problems related to handling, compaction and early age cracking related to SF. However, if difficulties are encountered in achieving the required chloride migration coefficient, then the addition of SF in a limited quantity (estimated at 5 to 7 % by mass of cementitious materials) could be investigated.

D2.5.1.1 Concrete Properties

D2.5.1.1.1 Chloride Migration Coefficient

The chloride migration coefficient is a measure of the resistance of concrete to chloride penetration and is a direct input parameter in *fib* Bulletin 34 modeling. Low values indicate that the concrete has a high resistance to chloride penetration.

Service life analysis has been performed for concrete having chloride migration coefficients varying from 3.10 x 10^{-9} square inches per second (in^2/s) to 15.50 x 10^{-9} in^2/s (2.00 x 10^{-12} m^2/s to 10.00 x 10^{-12} m^2/s). A proposed value within this range is then selected for each exposure zone. It is not recommended to use concrete with a chloride migration coefficient greater than 15.5 x 10^{-9} in^2/s (10.0 x 10^{-12} m^2/s) for the 20 % to 25 % Class F FA mix designs or 11.0 x 10^{-9} in^2/s (7.0 x 10^{-12} m^2/s) for the potential GGBS mix designs, as the concrete permeability should be limited to ensure a satisfactory concrete quality. The chloride migration coefficient will be determined based on the test NT Build 492 [8], referenced in the *fib* Model Code.

D2.5.1.1.2 Age Factor

The age factor describes the time-dependent change of the migration coefficient. This input parameter depends on the following factors:

- Prevailing exposure conditions.
- Hydration of the cementitious material including a correction because of convection that causes a more rapid chloride ingress into young concrete when compared to pure diffusion, which is the primary mechanism in older concrete.

The age factor choice is based on documentation available in *fib* [1], Gehlen [6], and presented in Table 2-2. The age factor is modeled as a beta distribution with a lower bound (A) equal to 0 and a higher bound (B) equal to 1.

Table 2-2: Age Factor used for the Bridge.

Cement/Cementitious Material Combination	Exposure Zone	
	Splash / Atmospheric with De-icing Salts (Beta Distribution Parameters)	Atmospheric without De-icing Salts (Beta Distribution Parameters)
Portland Cement + FA Class F (20%-35%)	μ=0.60;σ=0.15;A=0;B=1.0	μ=0.65;σ=0.15;A=0;B=1.0
Portland Cement + GGBS (30%-45%)	μ=0.40;σ=0.15;A=0;B=1.0	μ=0.65;σ=0.15;A=0;B=1.0

D2.5.1.1.3 Initial Chloride Content

The maximum total initial chloride concentration is assumed to be deterministic and equal to 0.1 % by mass of cementitious materials. This is consistent with ACI 318 Commentary, which recommends that for concrete exposed to a wet environment, the chloride limit in fresh concrete shall be no more than 0.10 % by mass of cement for acid soluble chlorides (ASTM C1152) or 0.08 % by mass of cement for water soluble chlorides (ASTM C1218M). The limit on acid soluble chloride is used here.

D2.5.1.1.4 Threshold Chloride Concentration

The threshold chloride concentration (or critical chloride concentration) is the concentration required to break down the passive layer protecting the steel reinforcement, which may lead to corrosion initiation. The threshold concentration cannot be represented by a single value as it depends on the concrete pH, amount of cement, concrete humidity, the type of cement/binder, and the interface properties between steel and concrete [9]. Chloride threshold values reported in the literature show a large scatter, values from 0.04 % to 8.34 % by mass of cementitious materials have been reported [9]. The *fib* Model Code suggests a mean value of 0.6 % 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 [1] as suggested by *fib.*

D2.5.1.1.5 Concrete Cover

Concrete cover is defined as the concrete thickness measured from the concrete surface to the outermost steel reinforcement. All concrete covers are modeled using a normal distribution with a standard deviation of 0.24 inches (6 mm) to account for the variability of as-constructed cover. The standard deviation is based on guidance provided by *fib*. It should be noted that the standard deviation suggested by *fib* is based on typically observed accuracy of reinforcement placement and is distinct from specified placement tolerances. The local Standard Specifications require that reinforcement be placed to provide a tolerance of ±1/4 in. for cover. Therefore, the analysis with a standard deviation of 0.24 in. is conservative since it assumes that approximately 16 % of the bars (the proportion outside of one standard deviation) do not actually meet the specified construction tolerance of 1/4 in.

Table 2-3 presents a summary of the concrete covers. “Mean” cover corresponds to the cover given in the construction specifications and “Construction Tolerance” corresponds to the maximum shortfall in cover (1/4 in.) that is permitted in the as-constructed work. The true minimum cover that is permitted in the as-constructed work is therefore the mean (or specified) cover minus the permitted construction tolerance.

Because the durability analysis was performed before completion of the design, the cover specified in the design is in some cases greater than the cover used in the analysis. This results in the provided protection being greater than the minimum required protection.

Table 2-3: Summary of Nominal Concrete Covers.

Structural Elements	Specified Construction Tolerance (in.)	Mean Concrete Cover Used in Durability Analysis (in.)	Standard Deviation Used in Analysis (in.)
Pile Caps	±0.25	2.0, 3.0	0.24
Tower Pedestals	±0.25	2.0, 3.0	0.24
Towers (below deck level - solid section)	±0.25	2.0, 3.0	0.24
Tower exterior (at deck level - hollow section)	±0.25	3.0	0.24
Tower exterior (>35 in. above deck level - hollow section)	±0.25	2.0	0.24
Piers (columns and cap)	±0.25	2.0, 3.0	0.24

D2.5.1.2 Reinforcing Steel

Uncoated carbon steel reinforcement will be used on all elements of the substructure.

D2.5.1.3 Exposure Conditions

D2.5.1.3.1 Temperature

The local average annual temperature is assumed to follow a normal distribution with a mean of 58.3°F (14.6°C) and standard deviation of 15.7 °F (8.7°C) based on public data.

D2.5.1.3.2 Transfer Function

Concrete elements exposed to chlorides and moisture with interruptions by dry periods is subject to capillary suction: the solution in the concrete pores close to the surface will evaporate during the dry periods and any rewetting will provoke a capillary action. This effect leads to a rapid transport of chlorides into the concrete up to a depth Δx, where the chlorides can accumulate with time until they reach a concentration equal to the surface concentration [1]. Beyond this depth chloride ingress is controlled by diffusion. The use of the transfer function effectively neglects any benefit from a thickness of Δx of the provided cover. By neglecting this amount of cover, the analysis is conservative compared to using a transfer function of 0, which would allow all of the provided cover to be used in the analysis.

The transfer function Δx has been taken as specified in the *fib* Bulletin 34 for splash zone environments; the mean value is 0.35 in. (8.9 mm) [1]. Besides the splash zone, the transfer function is also applicable to the atmospheric zones with de-icing salts subject to frequent wet-dry cycles.

The transfer function is 0 for atmospheric zone without de-icing salts as specified by the *fib* [1].

D2.5.1.3.3 Chloride Exposure—General

All input variables, such as surface chloride concentration, are expressed as probability functions with the *fib* methodology. The appropriateness of this approach is observable in the wide variation in chloride concentrations that are frequently determined from coring a particular bridge deck. A single value could not realistically represent such variation of observations. Surface chloride concentrations used in this assessment have been based on published data from multiple field testing programs.

D2.5.1.3.3.1 Chloride Exposure—Splash Zone/Atmospheric and Moderate Chloride Exposure

Pile caps and pier columns at anchor piers are exposed to fresh river water. Field data gathered from 1990 to 2007 show that the chloride concentration in the river and its tributaries averaged 30 milligrams per liter (mg/L) in 2007, with a median concentration of 48.9 mg/L. The 90th percentile was measured to be 95 mg/L. The monitoring of the chloride concentrations over that period showed that the concentration of chlorides increased by 2.7 mg/L per year. Hence, based on the median concentrations and the chloride increase rate, the chloride content can be assumed to be 48.9 mg/L in 2014 (opening of the Bridge) and increase linearly up to 318.9 mg/L by 2114 (100 years later); the average exposure over this 100-year period would be 183.9 mg/L. If doing the same exercise assuming the 90th percentile, the chloride exposure would increase linearly from 95.0 mg/L to 365.0 mg/L for an average exposure over 100 years of 230 mg/L. In all cases, the chloride exposure is relatively small and the water would not qualify as brackish water (more than 1 % chloride) even in the worst case scenario. These data are averages of the local river.

The pile caps, tower pedestals, and lower portions of the anchor piers may also be subject to de-icing salts coming from roadway drainage blown from the deck drains above. Since there will be no open drains directly above, it is expected that this will be only a minor issue. In addition, the anchor piers are located directly under expansion joints that may leak at some point. Hence, some chlorides may be transported to the pier caps and shaft.

The actual surface chloride concentration is difficult to accurately predict since it is largely dependent on the future level of maintenance provided for the drainage facilities. For the pile caps, tower pedestals, and piers, a conservative assumption has been made of a mean surface chloride concentration of 1.0 % by mass of binder. This value is modeled as a lognormal distribution with a COV of 0.5. It is expected that this value is more than what the pile caps and pier caps will actually experience.

D2.5.1.3.3.2 Chloride Exposure—Atmospheric Zone with De-icing Salts and Severe Chloride Exposure

Tower sections located near the deck level are in an atmospheric zone exposed to de-icing salts because of spray from the deck. The surface chloride concentration for structures exposed to de-icing salts is highly variable and depends on the type of concrete and environment to which the structure is exposed; moisture, wet-dry cycles, and evaporation rates also influence the surface chlorides concentration.

Table 2-4 presents bridge deck surface chloride concentration as documented in the literature for various locations in North America. Data in the literature is usually reported in kg/m^3 of concrete. The data in Table 2-4 was transformed into percentage by mass of cementitious materials assuming a content of 620 pounds per cubic yard (368 kg/m^3), which is the minimum cement content required by the local Standard Specifications for superstructure concrete. Note that because of this transformation, the actual chloride concentrations for the reported structures may be less than that shown in Table 2-4, depending on the actual cementitious materials content of the concrete. Data show considerable scatter, which is expected as samples were taken in different geographical areas, different locations on the structures, and structures were of different concretes and different ages.

A chloride surface concentration of 1.5 % is recommended by Dutch Guidelines CUR for a splash zone exposed to de-icing chemicals [20],[21]. Based on experience, this value might represent a lower bound. German DAfStb Guidelines recommend surface chloride concentrations of 2.6 % for Type I cement (Portland Cement) and 2.8 % for ground granulated blast furnace slag cement, at a depth of Δx = 8.9 mm for locations with use of de-icing salts [22].

Table 2-4: Measured In-situ Chloride Surface Concentration, C_s for North America from the Literature.

Author	Mean Cs (kg/m^3)	Mean Cs (%cement)	Comments	
Weyers (1998) [10]	5.2	1.41	Delaware	
	3.9	1.06	Minnesota	
	4.8	1.30	Iowa	
	5.1	1.39	West Virginia	
	5.4	1.47	Indiana	
	6.1	1.66	Wisconsin	
	2.2	0.60	Kansas	
	8.8	2.39	New York	

Table 2-4: Measured In-situ Chloride Surface Concentration, C_s for North America from the Literature.

Author	Mean Cs (kg/m^3)	Mean Cs (%cement)	Comments	
	1.9	0.52	California	
	3.6	0.98	Florida	
Cusson (2011) [11]	16.5	4.48	Quebec, Canada	1 structure - barrier
Langlois (2010) [12]	3.1	0.84	Quebec, Canada	20 structures
	(0.5-16)	(0.13-4.4)		
Cady and Weyers (1983) [13]		0.10	Kansas	21 structures
		0.14	Michigan	13 structures
		0.15	California	21 structures
		0.08	Missouri	18 structures
		0.10	Average USA	73 structures
Coggins and French (1990) [14]	7.55	2.05	Minnesota, USA	1 structure
Funashi (1990) [15]	7.52	2.04	Connecticut, USA	1 structure - parking garage
Hoffman and Weyers (1994) [16]	3.5	0.95	USA	321 structures
	(1.2-8.2)	(0.3-2.2)		
Lounis and Amleh (2004) [17]	4.67	1.27	Quebec, Canada	1 structure
Williamson and al. (2008) [18]	(0.62-6.67)	(0.2-1.8)		
LIFE-365 Software		4.4	Urban Highway Bridges (40-years build-up period)	
Hooton, R.D. and al. (2010) [19]		4.15 (3.97-4.32)	Ohio DOT Bridge	1 structure

The value shown in Table 2-4 for "Life - 365 Software" is based on the default setting for Urban Highway Bridges in a location near the Bridge where a chloride surface concentration of 0.68 % by mass of concrete is listed. This includes a linear build-up period of 40 years. Values suggested by LIFE-365 are intended for use in a deterministic assessment using a single value, whereas the assessment methodology used for this project uses a probabilistic approach with expected variation about mean value. A deterministic value would normally be selected to be greater than the mean value.

Data from bridge decks are expected to be conservative for towers not directly subject to the application of de-icing salts. Based on a conservative interpretation of data from the literature, the chloride exposure level of the atmospheric zone exposed to de-icing salts (towers at deck level) is described in this analysis as a lognormal distribution with a mean of 3 % with a COV of 0.5.

D2.5.2 Summary of Input Parameters

D2.5.2.1 Splash Zone/Atmospheric Zone with Moderate De-icing Salts

Table 2-5 presents the input parameters for the splash zone.

Table 2-5: Input Parameters for Splash Zone/Atmospheric Zone with Moderate De-icing Salts and 100-year Service Life.

Variable	Symbol	Distribution	Unit	Mean μ	Standard Deviation and Function Parameters
Chlorides Migration Coefficient	D_{28}	Normal	x 10^{-9} in^2/s (x 10^{-12} m^2/s)	3.10 (2.00) 4.65 (3.00) 6.20 (4.00) 7.75 (5.00) 9.30 (6.00) 10.85 (7.00) 12.40 (8.00) 13.95 (9.00) 15.50 (10.00)	0.2μ
Surface Concentration	C_s	Lognormal	Mass % of binder	1.00	0.50μ
Critical Chloride Concentration	C_{crit}	Beta	Mass % of binder	0.6	0.15 $A^{(1)}=0.2$; $B^{(1)}=2$
Initial Chloride Concentration	C_0	Deterministic	Mass % of binder	0.1	-
Aging Factor	a	Beta	-	Table 2-2	Table 2-2
Temperature	T_{real}	Normal	°F (°C)	58.3 (14.6)	15.7 (8.7)

Variable	Symbol	Distribution	Unit	Mean μ	Standard Deviation and Function Parameters
Cover	c	Normal	Inch (mm)	2.0 (50.8) 3.0 (76.2)	0.24 (6)
Transfer function	Δx	Beta	Inch (mm)	0.35 (8.9)	0.22 (5.6) A=(1)0; B(1)=50

(1) A and B are the beta distribution parameters.

D2.5.2.2 Atmospheric Zone with Severe De-icing Salts

Table 2-6 presents the input parameters for the atmospheric zone with severe de-icing salts and a 100-year service life of the substructure.

Table 2-6: Input Parameters for Atmospheric Zone with Severe De-icing Salts and a 100-year Service Life of the Substructure.

Variable	Symbol	Distribution	Unit	Mean μ	Standard deviation and function parameters
Chlorides Migration Coefficient	D_{28}	Normal	x 10^{-9} in^2/s (x 10^{-12} m^2/s)	3.10 (2.00) 4.65 (3.00) 6.20 (4.00) 7.75 (5.00) 9.30 (6.00) 10.85 (7.00) 12.40 (8.00) 13.95 (9.00) 15.50 (10.00)	0.2μ
Surface Concentration	C_s	Lognormal	Mass % of binder	3.0	0.50μ
Critical Chloride Concentration	C_{crit}	Beta	Mass % of binder	0.6	0.15 A=0.2; B=2

Variable	Symbol	Distribution	Unit	Mean μ	Standard deviation and function parameters
Initial Chloride Concentration	C_0	Deterministic	Mass % of binder	0.1	-
Aging Factor	a	Beta	-	Table 2-2	Table 2-2
Temperature	T_{real}	Normal	°F (°C)	58.3 (14.6)	15.7 (8.7)
Cover	c	Normal	Inch (mm)	3.00 (76.2)	0.24 (6)
Transfer function	Δx	Beta	in. (mm)	0.35 (8.9)	0.22 (5.6) A=(1)0; B(1)=50

(1) a and b are the beta distribution parameters.

D2.5.2.3 Atmospheric Zone without De-icing Salts

Table 2-7 presents the input parameters for the atmospheric zone without de-icing salts and a 100-year service life.

Table 2-7: Input Parameters for Atmospheric Zone without De-icing Salts and 100-year Service Life.

Variable	Symbol	Distribution	Unit	Mean μ	Standard deviation and function parameters
Chlorides Migration Coefficient	D_{28}	Normal	x 10^{-9} in^2/s (x 10^{-12} m^2/s)	15.50 (10.00)	0.2μ
Surface Concentration	C_s	Lognormal	Mass % of binder	Section 2.5.1.3.3	0.50μ
Critical Chloride Concentration	C_{crit}	Beta	Mass % of binder	0.6	0.15 A(1)=0.2; B(1)=2
Initial Chloride Concentration	C_0	Deterministic	Mass % of binder	0.1	-
Ageing Factor	a	Beta	-	Table 2-2	Table 2-2
Temperature	T_{real}	Normal	°F (°C)	58.3 (14.6)	15.7 (8.7)
Cover	c	Normal	Inch (mm)	2.0 (50.8)	0.24 (6)
Transfer function	Δx	Deterministic	Inch (mm)	0	-

(1) A and B are the beta distribution parameters.

D2.5.3 Results

Results for all exposure zones are presented in the following section. The range of material resistance parameters considered for the concrete mixes (that is, the chloride migration coefficients) has been selected based on experience from comparable infrastructure projects with similar concrete mixes. The achieved values for the migration coefficient of concrete mixes developed for use in the project will be confirmed through laboratory testing (NT Build 492 [8]).

D2.5.3.1 Splash Zone/Atmospheric Zone with Moderate De-icing Salts

Figure 2-4 presents the reliability index achieved for concrete with various chloride migration coefficients for the splash zone.

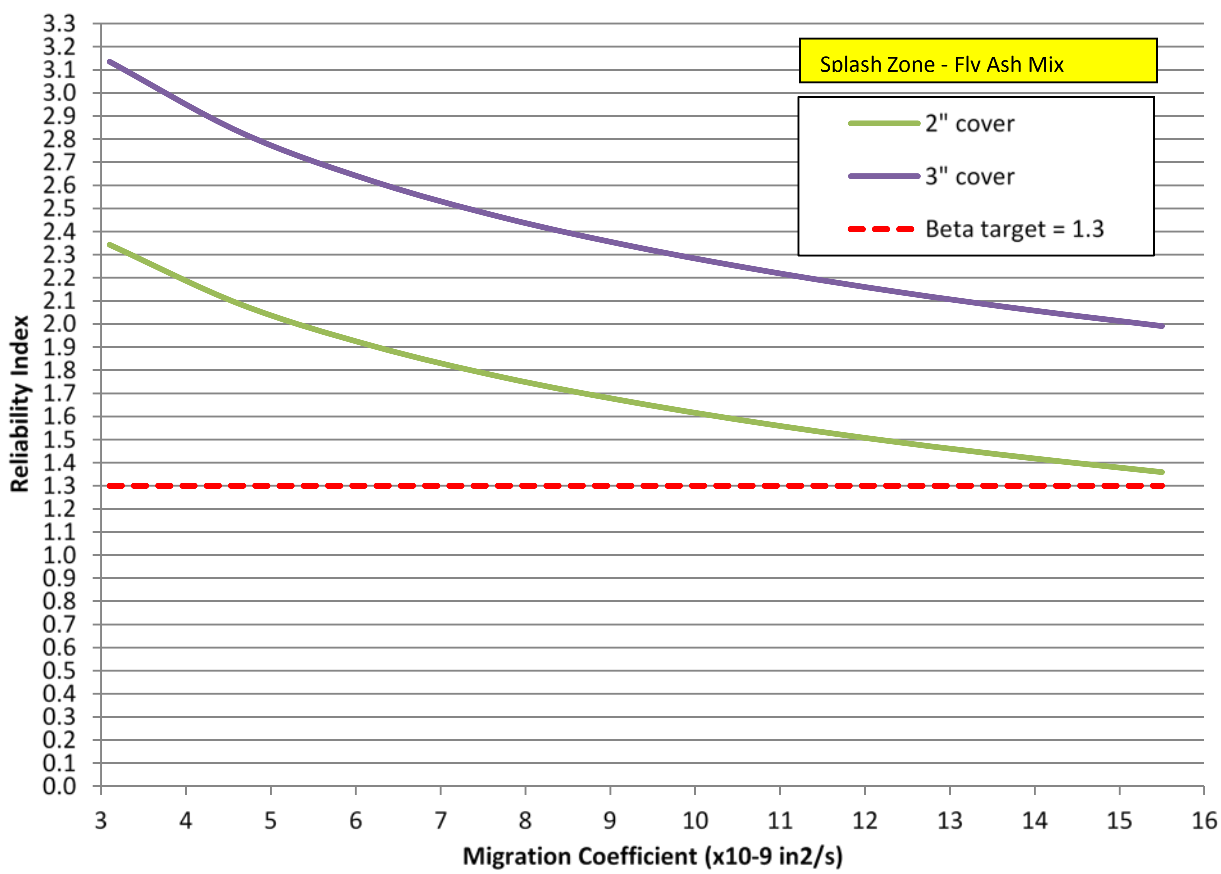


Figure 2-4: Reliability Index for 2- and 3-in. Cover using Different Migration Coefficients, Assuming 1.0 % Surface Chloride Concentration, 100-year Service Life, and Portland Cement with Minimum 20 % Fly Ash.

The target reliability index is 1.3. A reliability index greater than 1.3 means that corrosion has less than 10 % probability of initiation within 100 years. As noted previously, concrete with a chloride migration coefficient greater than 15.5 x 10^{-9} in^2/s (10.0 x 10^{-12} m^2/s) is not recommended.

Based on the results shown on Figure 2-4, it can be seen that the 2-in. cover and a maximum chloride migration coefficient at 28 days of 15.5 x 10^{-9} in^2/s (10.0 x 10^{-12} m^2/s) are sufficient to achieve a 100-year service life for a surface chloride concentration of 1.0 %. This is based on a concrete mix design with a minimum of 20 % FA by mass of total cementitious materials.

Figure 2-5 illustrates the different requirements that apply if GGBS is used. Based on the results shown on Figure 2-5, it can be seen that a 3-in. cover and a maximum chloride migration coefficient at 28 days of 11.3 x 10^{-9} in^2/s (7.3 x 10^{-12} m^2/s) are required to achieve a 100-year service life for a surface chloride concentration of 1.0 % in the splash zone. This is based on a concrete mix design with Portland Cement and 30 to 45 % GGBS by mass of total cementitious materials. A 2-in. cover is not recommended if using GGBS concrete because of the very low migration coefficient that would be required. It is not currently planned to use GGBS.

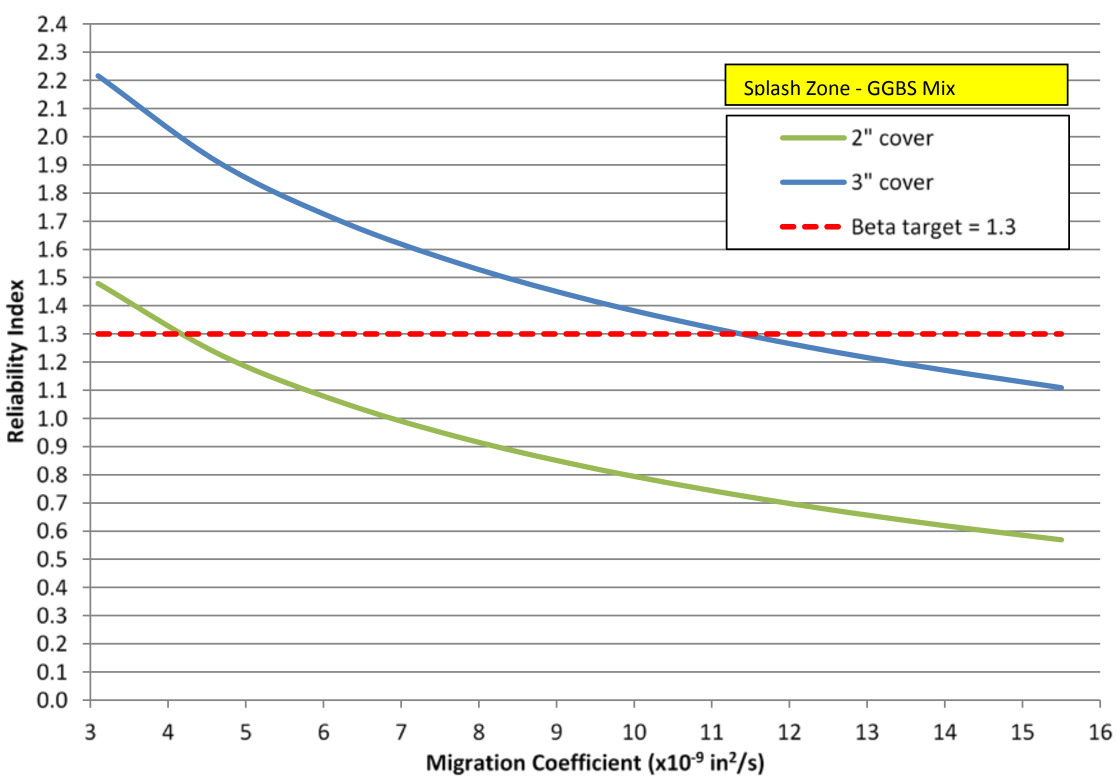


Figure 2-5: Reliability Index for 2- and 3-in. Cover using Different Migration Coefficients, Assuming 1.0 % Surface Chloride Concentration, 100-year Service Life, and Portland Cement with 30 to 45 % GGBS.

D2.5.3.2 Atmospheric Zone with Severe De-icing Salts

Results on Figure 2-6 present the reliability index for a 3-in. concrete cover and concrete migration coefficients assuming 3.0 % chloride surface concentration by mass of binder, a target service life of 100 years, and the use of Portland Cement with minimum 20 % FA by mass of total cementitious materials.

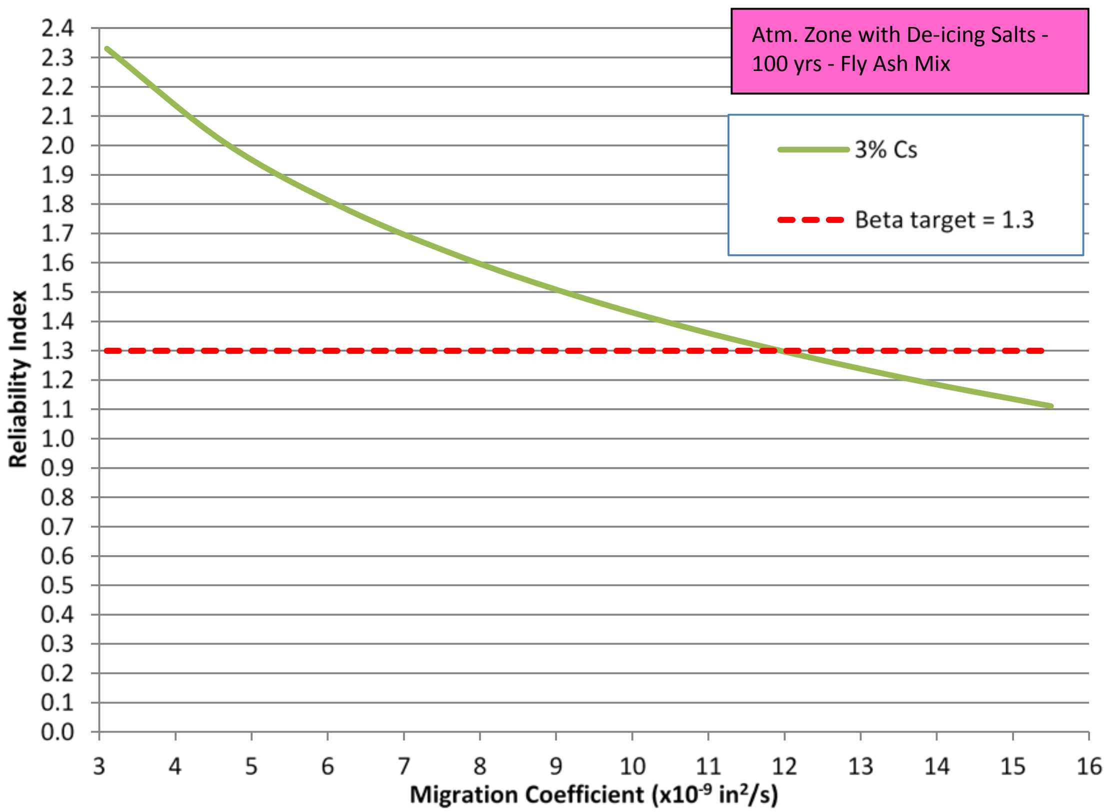


Figure 2-6: Reliability Index for 3-in. Cover using Different Migration Coefficients, Assuming 3.0 % Surface Chloride Concentration, 100-year Service Life, and Portland Cement with Minimum 20 % Fly Ash.

The target reliability index is 1.3. A reliability index greater than 1.3 means that corrosion has less than 10 % probability of initiation within 100 years.

Based on these results, the 100-year service life can be achieved for a surface chloride concentration of 3 % provided:

- Concrete cover is 3 inches (76 mm).
- Maximum chloride migration coefficient 11.8 x 10^{-9} in^2/s (7.6 x 10^{-12} m^2/s) at a concrete age of 28 days.

The target migration coefficient at 28 days at 11.8 x 10^{-9} in^2/s is expected to be achievable while respecting the local Standard Specifications for a maximum FA content of 20 % by mass of cementitious materials. Factors such as the w/c ratio, total cementitious content, and aggregate gradation can be varied as needed to achieve the required migration coefficient. Conformance with the required migration coefficient will be verified by testing in accordance with NT Build 492 [8].

The amount of FA could be increased should the target migration coefficient not be readily achievable within the local limits. It is recommended that the FA content should be limited to a maximum of 25 % by mass of cementitious materials (ACI 318). The local Standard Specifications limit the content of Class F FA to a maximum of 20 % of total cementitious materials. A relaxation of the local Standard Specifications will be required should Class F FA be used to an amount of 25 % by mass of cementitious materials.

D2.5.3.3 Atmospheric Zone without De-icing Salts

The exterior faces of the towers located in the atmospheric zone without de-icing salts will be subject to very limited chloride exposure. A 2-in. cover for the exterior tower surface will suffice as required by the structural design. As noted previously, it is recommended to provide a concrete with a chloride migration coefficient less than 15.5 x 10^{-9} in^2/s (10.0 x 10^{-12} m^2/s) to ensure durability of the concrete. This migration coefficient is expected to be achievable by all concrete mixes proposed in Section 2.5.1, except the drilled shaft mix where the exposure to de-icing salts is not applicable.

Analyses show that a concrete mix with 2-in. cover and a migration coefficient of 15.5 x 10^{-9} in^2/s (10.0 x 10^{-12} m^2/s) would achieve a 100-year service life when subject to a maximum airborne chloride concentration of 1.75 %. This level of chloride exposure is believed to be greater than the towers will experience during their service life.

D2.5.4 Summary of Probabilistic Assessment

Table 2-8 presents a summary of the concrete cover and maximum chloride migration coefficient at 28 days required to achieve the specified service life. The chloride migration coefficient obtained following the NT Build 492 test procedure shall be of equal or lesser value than specified in Table 2-8.

Table 2-8: Summary of Required Concrete Cover and Chloride Migration Coefficient based on a Probabilistic Assessment.

Exposure Zone	Structural Element	Required Cover for Service Life (in.)	Concrete Mix	Max. Chloride Migration NT Build 492 [8] at 28 days
Splash Zone/ Atmospheric with moderate de-icing salts	Pile Caps Tower Pedestals Piers	2	Portland Cement + 20-25% FA Class F	15.5×10^{-9} in^2/s (10.0×10^{-12} m^2/s)
		3	Portland Cement + 30-45% GGBS	11.3×10^{-9} in^2/s (7.3×10^{-12} m^2/s)
Atmospheric with severe de-icing salts	Towers (exterior) at deck level	3	Portland Cement + 20-25% FA Class F	11.8×10^{-9} in^2/s (7.6×10^{-12} m^2/s)
Atmospheric without de-icing salts	Towers (exterior) below deck level	2	Portland Cement with 20-25% FA Class F	15.5×10^{-9} in^2/s (10.0×10^{-12} m^2/s)
	Towers (exterior) above deck level	2		

D2.6 Concrete Durability Testing Requirements

D2.6.1 Cementitious Materials

Cementitious materials will be from approved material sources by the local authorities. In addition:

- Portland Cement will contain a low alkali content (<0.6% equivalent Na_2O) as defined in ASTM C150.
- FA will be Class F as defined by ASTM C618.

Portland Cement with a higher alkali content may be acceptable if other measures to mitigate AAR are provided. Alternative measures will be subject to review and approval. The limit on the alkali content does not apply to the drilled shafts concrete mix.

D2.6.2 Aggregates

Aggregates will be from the approved material sources and be approved for freeze-thaw by the local authorities. The local Standard Specifications require, in particular, that the expansion potential of the aggregates will be tested in accordance with relevant local standards. The beam expansion will be less than 0.06 % at 6 months.

Potential reactive aggregates will be addressed through the provisions of AASHTO R80-17 [24].

D2.6.3 Concrete

As a starting point, it is assumed that the concrete mixes will consist of the following:

- Portland Cement with 20 % to 25 % Class F FA by mass of total cementitious materials for all structural elements and exposure conditions except drilled shafts.
- As an alternative solution, Portland Cement with 30 % to 45 % GGBS by mass of total cementitious materials may be used for splash zone/atmospheric zone with moderate de-icing salts (pile caps, piers, tower pedestals). (The use of GGBS is not planned).
- Portland Cement with 45 % Class F FA by mass of total cementitious materials for the drilled shafts.

Concrete mixes will comply with the local Standard Specifications. The target migration coefficients at 28 days, as determined by this analysis, are expected to be achievable while respecting the local Standard Specifications for a maximum FA content of 20 % by mass of cementitious materials. Factors such as the w/c ratio, total cementitious content, and aggregate gradation can be varied to achieve the required migration coefficient. If additional supplementary cementitious materials are used to achieve the maximum chloride migration coefficient (more than 20 % Class F FA or more than 30 % GGBS), relaxation from the local Standard Specifications will be needed. These limits are primarily based on considerations that do not apply to the drilled shafts environment. Therefore, in agreement with the local authorities, limits have been waived for the drilled shafts.

Additional requirements to the local Standard Specifications for each concrete mix are summarized in Table 2-9.

Concrete cover larger than specified in Table 2-9 may be specified in the structural design. This results in the provided protection being greater than the minimum required protection determined by this service life assessment.

The following requirements, in addition to the project Standard Specifications, will be tested during the trial phase:

- For concrete subject to freezing and thawing (all mixes except the drilled shaft), the air-void system will be tested in accordance with ASTM C457 Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete using a magnification factor between 100 and 125. The air-void system of the concrete mix will be considered satisfactory when the average of all tests shows a spacing factor not exceeding 0.008 inches, with no single test greater than 0.010 inches, and air content greater than or equal to 3.0 % in the hardened concrete. For concrete with a water-to-cementitious materials ratio of 0.36 or less, the average spacing factor will not exceed 0.0098 inches, with no single value greater than 0.0118 inches.
- The acid soluble chloride content in fresh concrete as measured by ASTM C1152, will not exceed 0.10 % by mass of cementitious materials.

The following test will be part of the trial-phase testing and the production phase in addition to the local Standard Specifications:

- The chloride migration coefficient at 28 days as measured by NT Build 492 [8] will not exceed the required value determined by the *fib* Model Code analysis (values are summarized in Table 2-9).

The placement and curing methods will comply with the local Standard Specifications.

Temperature requirements will be stated in the Thermal Control Plan for any structural element, excluding drilled shafts, with its least plan dimension being 7 ft or greater. For concrete elements not included in the Thermal Control Plan and excepting drilled shafts, temperature requirements stated in the local Standard Specifications are applicable. In addition, the maximum internal temperature of foundations and substructure concrete during the hardening phase will be limited to 160^{o}F (71^{o}C).

Table 2-9: Summary of Exposure Zone, Concrete Mix Requirements and Test Requirements.

Exposure Zone	Structural Element	Cover (in.)(3)	Concrete Mix	Max w/cm	Air Content ASTM C231 or ASTM C173	Max. Chloride Migration NT Build 492 [8] at 28 days(1)	Max. Chloride Content ASTM C1152	AAR Tests	Freeze-Thaw Tests	T°limit(2)
Submerged/Buried	Drilled Shafts	6/3	Portland Cement + 45% FA Class F	0.4	-	-	-	-	-	-
Splash Zone/ Atmospheric with moderate de-icing salts	Pile Caps Tower Pedestals Piers	2	Portland Cement + 20-25% FA Class F	0.4	6 ±2%	15.5×10^{-9} in²/s (10.0×10^{-12} m²/s)	0.1%	x	x	x
		3	Portland Cement + 30-45% GGBS	0.4	6 ±2%	11.3×10^{-9} in²/s (7.3×10^{-12} m²/s)	0.1%			x
Atmospheric with severe de-icing salts	Towers (exterior) at deck level	3	Portland Cement + 20-25% FA Class F	0.4	6 ±2%	11.8×10^{-9} in²/s (7.6×10^{-12} m²/s)	0.1%	x	x	x
Atmospheric without de-icing salts	Towers (interior)	1.5	Portland Cement with 20-25% FA Class F	0.4	6 ±2%	15.5×10^{-9} in²/s (10.0×10^{-12} m²/s)	0.1%	x	x	x
	Towers (exterior) below deck level	2								x
	Towers (exterior) > 35 ft above deck level	2								x

(1) Chloride migration coefficients are applicable only to the corresponding concrete mix design, cover and exposure conditions

(2) Temperature requirements for elements with a least dimension of 7 ft or greater are to be defined in the Thermal Control Plan.

(3) For ease of construction, structural design in some cases may specify greater concrete covers than required by the Service Life analysis.

D2.7 Conclusion

This document presents the durability assessment of the concrete substructure (drilled shafts, pile caps, towers, and piers) for the Bridge. The nonreplaceable concrete components of the Bridge are required to provide a 100-year service life.

A durability assessment has been performed in accordance with the Project Specifications to determine the required concrete cover and concrete quality. Exposure zones, identified in Section 2.3, are defined for each structural element: atmospheric without de-icing salts, atmospheric with severe de-icing salts, splash zone/ atmospheric zone with moderate de-icing salts, and submerged/buried zone.

The avoidance of deterioration approach, presented in Section 2.4, is implemented for the following concrete deterioration mechanisms: sulfate attack, AAR, DEF, and leaching. Freeze-thaw are addressed by the deemed-to-satisfy method.

Protection for chloride-induced reinforcement corrosion is based on a probabilistic approach addressed in Section 2.5. The *fib* Model Code is used to model chloride-induced reinforcement corrosion in order to determine concrete covers and maximum chloride migration coefficients at 28 days as measured by NT Build 492 [8].

As a starting point, it is assumed that the concrete mixes will consist of the following:

- Portland Cement with 20 % to 25 % Class F FA by mass of total cementitious materials for all structural elements and exposure conditions except drilled shafts.
- As an alternative solution, Portland Cement with 30 % to 45 % GGBS by mass of total cementitious materials may be used for splash zone only (pile caps, piers, tower pedestals).
- Portland Cement with 45 % Class F FA for the drilled shafts.

Table 2-9 summarizes the required concrete covers and maximum chloride migration coefficients for each exposure zone, structural element, and concrete mix based on this service life assessment. If alternate concrete mix designs are considered, calculations and assessments will need to be redone.

Concrete works will comply with the local Standard Specifications. Additional requirements necessary to achieve the target service life are specified in Section 2.6.

D2.8 References

[1] *fib* (2006). *Model Code for Service Life Design*. *fib* Bulletin 34. International Federation for Structural Concrete (fib), Lausanne, Switzerland, 1st edition. 126 pp.

[2] 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.

[3] ACI 318. "Structure Concrete Building Code and Commentary". American Concrete Institute, ACI 318M-11.

[4] AASHTO, "AASHTO LRFD Bridge Construction Specifications, U.S. Customary Units," 3rd Edition, with 2010, 2011, and 2012 Interim Revisions.

[5] DS-EN 1992-1-1. "Eurocode 2: Design of Concrete Structures - Part 1-1: General rules and rules for buildings." 227p.

[6] Gehlen, C. (2000). Probabilistische Lebensdauerbemessung von Stahlbetonbauwerken. Zuverlässigkeitsbetrachtungen zur wirksamen Vermeidung von Bewehrungskorrosion. DAfStb Heft 510. Deutscher Ausschuss für Stahlbeton, Berlin, Germany. 106 pp. German language

[7] DARTS – Durable and Reliable Tunnel Structures: Data, European Commission, Growths 2000, Contract G1RD-CT-2000-00467, Project GrD1-25633, 2004.

[8] NT BUILD 492 (1999). Concrete, mortar and cement-based repair materials: Chloride migration coefficient from non-steady-state migration experiments. NORDTEST method 492. NORDTEST, Espoo, Finland.

[9] 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.

[10] Weyers, R.E. (1998). "Service life model for concrete structures in chloride laden environments." ACI Materials Journal, 95(4), 445-451.Cady, P.D. and R.E. Weyers (1983). "Chloride penetration and deterioration of concrete bridge decks." Cement, Concrete and Aggregates, 5(2), 81-87.

[11] Cusson, D., Lounis, Z. and Daigle, L. (2011)."Durability monitoring for improved service life predictions of concrete bridge decks in corrosive environments." Computer-Aided Civil and Infrastructure Engineering, 26, 524-541.

[12] Langlois, A.M. (2010). "Influence of damages on the structural reliability of reinforced concrete bridges: case studies". M.Sc. Thesis, Department of Civil Engineering, Laval University, 227p. French language.

[13] Cady, P.D. and R.E. Weyers (1983). "Chloride penetration and deterioration of concrete bridge decks." Cement, Concrete and Aggregates, 5(2), 81-87.

[14] Coggins, F.B. and C.W. French (1990). "Chloride ion distribution in twenty-year-old prestressed bridge girders." ACI Materials Journal, 87(5), 479-488.

[15] Funahashi, M. (1990). "Predicting corrosion-free service life of a concrete structure in a chloride environment." ACI Materials Journal, 87(6), 581-587.

[16] Hoffmann, S. and R.E. Weyers (1994). Predicting critical chloride levels in concrete bridge decks. G.I. Shueller, M. Shinozuka et J.T.P. Yao, Structural Safety, ICOSSSAR'93, Innsbruck, Balkema.

[17] Lounis, Z. and L. Amleh (2004). "Reliability-based prediction of chloride ingress and reinforcement corrosion of aging concrete bridge decks". NRCC-47011, Conseil national de recherche du Canada.

[18] Williamson, G.S., R.E. Weyers, M.C. Brown, A. Ramniceanu and M. Sprinkel (2008)." Validation of probability-based chloride-induced corrosion service-life model." ACI Materials Journal, 105(4), 375-380.

[19] Hooton, R.D., Bentz, E.C., and Jojundic, T. (2010). "Long-term Chloride Penetration of Silica Fume Concretes Based on Field Exposure". Proceedings, Service Life Design for Infrastructure RILEM PRO 70, Vol.1, pp.503-512.

[20] CUR 215 (2005). *Durability of Concrete Structures in Marine Environment*. CUR Publication 215. Civieltechnisch Centrum Uitvoering Research en Regelgeving, CUR Voorschriftencommissie 81, Gouda, Netherlands. 132 pp. Dutch language.

[21] CUR LD 1 (2009). *Duurzamheid van constructief beton met betrekking tot chloride-geintieerde wapeningscorrosie*. CUR-Leidraad 1. Civieltechnisch Centrum Uitvoering Research en Regelgeving, CUR Voorschriftencommissie 81, Gouda, Netherlands. 65 pp. Dutch language.

[22] DAfStb (2008). ‘Positionspapier des DAfStb zur Umsetzung des Konzepts von leistungsbezogenen Entwurfsverfahren unter Berücksichtigung von DIN EN 206-1, Anhang J’. In *Beton- und Stahlbetonbau, Issue 12*. Berlin, Germany: Deutscher Ausschuss für Stahlbeton. German language.

[23] ACI Committee 221 (1998), Report on Alkali-Aggregate Reactivity (ACI 221.1R-98) ACI Manual of Concrete Practice, page 7, 31 pp.

[24] AASHTO R80. Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction. Washington DC: American Association of State Highway and Transportation Officials (AASHTO); 2017.

D3 Example 3 – Service Life Design Calculation Booklet for a Conventional Multi-Span Prestressed Concrete Girder Bridge

Contents

D3.1 Scope of the work

This document summarizes the calculations performed for the service life design of the concrete member of a conventional multi-span pre-stressed concrete girder bridge located in a bayou in the Southeast United States.

A performance-based service life approach is used to address chloride-induced corrosion for the concrete components of the bridge.

D3.2 Exposure Zones of Components

For the purposes of this design calculation book, the bridge is assumed to be located near the Cedar Key in Florida as shown on Figure 3-1.

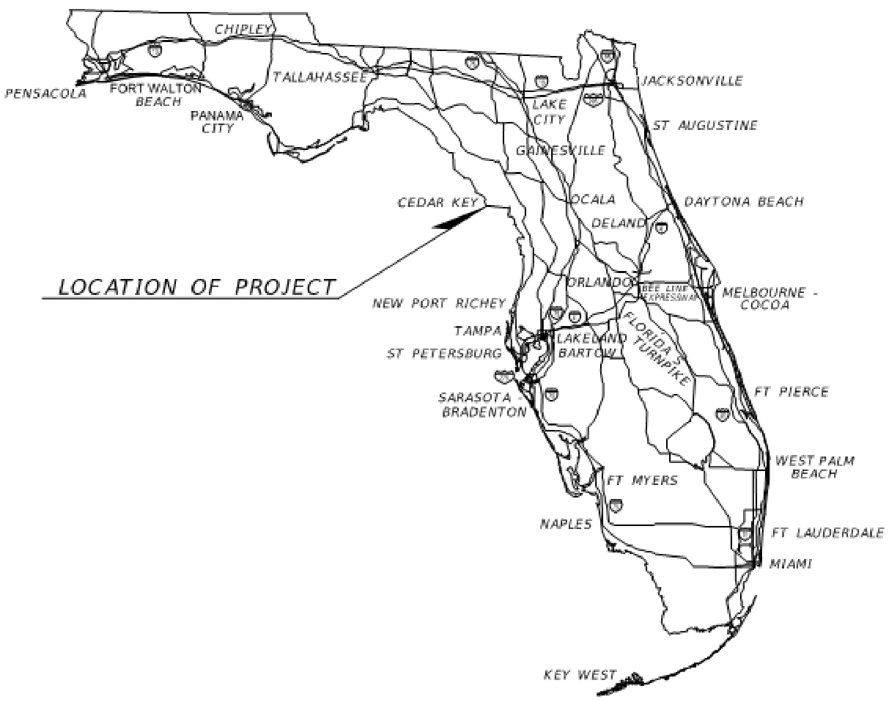


Figure 3-1: Location of the bridge in Cedar Key, Florida, USA

The following exposure zones, shown on Figure 3-2, have been identified for the main concrete members of the bridge:

- Submerged and Buried: Permanently buried in soil or submerged in the water: Prestressed piles with stainless strand and stainless reinforcement. Stainless steel itself has approximately 10-times the corrosion resistance of black bars and considered to be an "avoidance approach" for chloride-induced corrosion. Therefore, no calculations were performed for piles.

- Splash, Spray and Tidal: Not permanently submerged in the water, subject to wet-dry cycles. According to FDOT structure design guidelines part of the bridge, this zone is the vertical distance from 4 ft below mean low water level (MLW) to 12 ft above mean high water level (MHW).
- Groundwater and Soil: Exposed to soil and/or groundwater

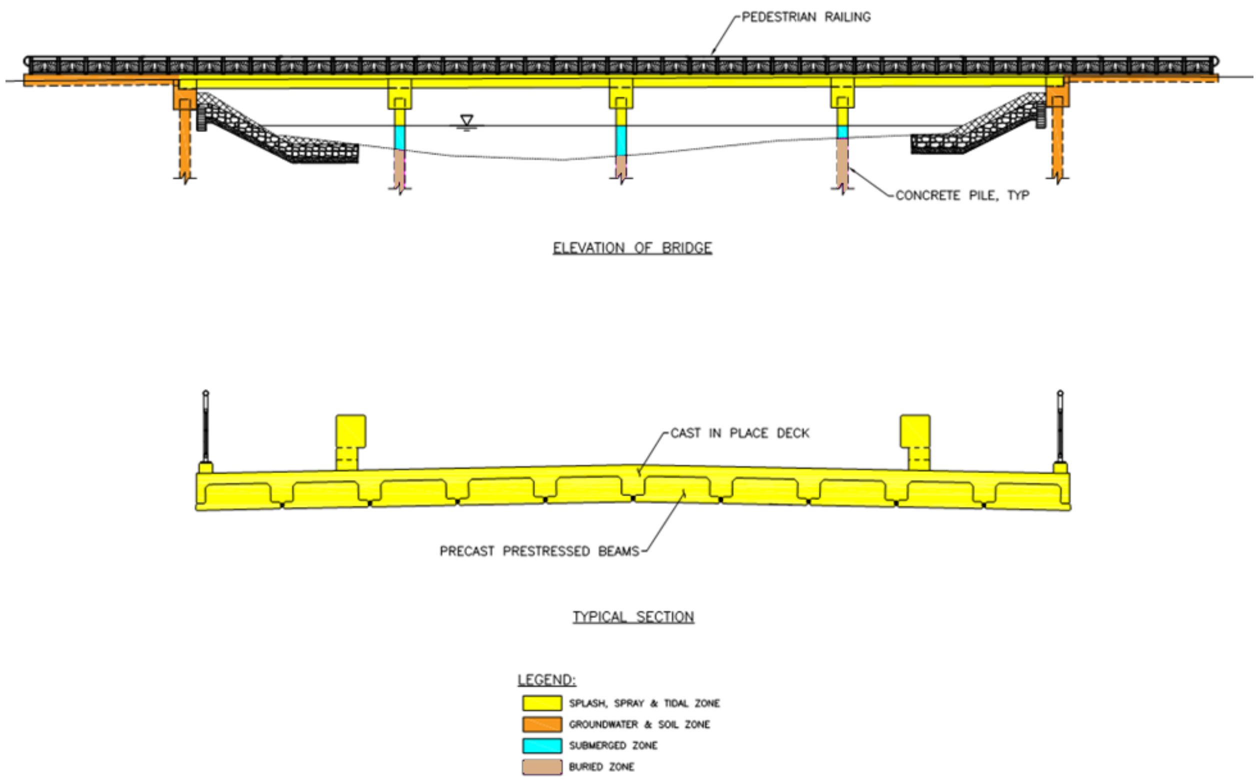


Figure 3-2: Exposure zones of concrete members.

Table 3-1 shows structural members covered in this calculation and the minimum required service life specified in the Project Agreement.

Table 3-1: Structural Members and Required Service Life.

Structural member	Required service life by Project Agreement
End and Intermediate Bents	100
Deck	50
Precast prestressed beams	50
Approach slab	100
Barriers	25

D3.3 Defining Input Parameters

This section presents the choice of input parameters for the time to corrosion model.

D3.3.1 Temperature

The average annual temperature was calculated using the data summarized in Table 3-2. This information adopted from the NOAA website [3] is from the closest station (name of station is 'Lower Suwannee') to the project site and goes back to 2003. It is noted that temperature data from the weather station is not available for each year since 2003; however, enough data is available to obtain a reasonable estimate of the average value and standard deviation, which is concluded by comparing the data to a larger data set from a nearby weather station in Cedar Key. This station yields an average temperature of 70.95˚F and a standard deviation of 1.24˚F from 1908-1975.

Table 3-2: Temperature Data from the Closest Weather Station to the Project Site. Data is not Available for Each Year.

Year	Average Temperature ˚F (˚C)
2003	69.2 (20.67)
2004	68.9 (20.5)
2005	68.7 (20.39)
2009	69.0 (20.56)
2012	70.3 (21.28)
2014	68.7 (20.39)
2015	71.9 (22.17)
2016	71.1 (21.72)
2017	71.1 (21.72)
Average	69.9 (21)
Standard Deviation	2.2 (1.24)

The average annual temperature in the job site is set to follow a normal distribution with a mean of 21˚C and standard deviation of 1.24˚C based on the temperature information provided above.

D3.3.2 Surface Chloride Concentration

The following surface chloride concentrations are considered for each identified exposure zone:

D3.3.2.1 Ground water and Soil

Project-specific testing of the water under the bridge provides a salinity of 30,000 parts per million. Salinity of the groundwater and soil is assumed to be controlled by the salinity of the seawater. For the zone exposed to the soil and groundwater, the mean value of surface chloride concentration is considered as 3% with a standard deviation of 1.5%, following a log-normal distribution.

D3.3.2.2 Splash, Spray and Tidal Zone

Members in this zone are subject to wet-dry cycles. A chloride surface concentration with a mean value of 3.5% w/w_{cm} and standard deviation of 1.75% is assumed, following a log-normal distribution.

D3.3.3 Background Chloride Concentration

The concrete's initial total chloride content is taken to be deterministic and set equal to 0.10% chloride by weight of cementitious materials (%w/w_{cm}) for cast in place concrete and 0.06%w/w_{cm} for precast prestressed beams. These limits are based on ACI 301 and the FDOT Standard Specifications for Road and Bridge Construction (Jan 2018). The FDOT limit for initial chloride content for prestressed concrete is 0.40 lbs/ yd^3 concrete, which would correspond to roughly 0.06% w/w_{cm} while the content of cementitious materials is not less than approximately 600 lbs/yd^3.

D3.3.4 Chloride Threshold Value

The *fib* Bulletin 34 [1] suggests a mean value of 0.6% by mass of cementitious materials for carbon 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 as suggested by *fib* Bulletin 34.

D3.3.5 Transfer function

Elements in the Groundwater and Soil zone are assumed to be exposed to cycling wetting and drying due to variations in the groundwater level while elements in the Buried Zone (i.e., elements in the seabed) are assumed permanently submerged. Therefore, the transfer function Δx has been taken as specified as follow based on fib Bulletin 34:

- For Splash, Spray and Tidal zone as well as Groundwater and Soil zone: beta distribution with a mean value of 8.9 mm, standard deviation of 5.6 mm with parameter A = 0.0 and B = 50.0.
- For buried and submerged zone: deterministic value of 0.

D3.3.6 Selected Binder Compositions and Age Factor

In general, the following three binder compositions were investigated for the service life design of concrete members in the bridge. For the deck elements (precast and in situ cast portions) and the barriers only the OPC + 20-35% FA binder composition described as follows was considered:

- OPC + 20-35% FA: Portland Cement with 20%-35% Type F FA by mass of total cementitious materials, FA content for decks and barriers is limited to 20%.
- OPC + 36-65% Slag cement (GGBS): Portland Cement with 36-65% GGBS grade 100 or higher by mass of total cementitious materials
- OPC + 66-80% GGBS: Portland Cement with 66%-80% GGBS grade 100 or higher by mass of total cementitious materials

The choice of the age factor is based on values provided in *fib* Bulletins 34 and 76 [2]:

Table 3-3: Age Factor.

Concrete Mixes	Distribution	Splash, Spray, Tidal, Submerged, Buried, Groundwater and Soil Zones	
		Parameters	Mean (μ)
Portland Cement + 20-35% FA	beta	σ=0.15, A=0, B=1	0.60
Portland Cement + 36-65% GGBS	beta	σ=0.18, A=0, B=1	0.40
Portland Cement + 66-80% GGBS	beta	σ=0.20, A=0, B=1	0.45

D3.3.7 Concrete Cover

The first round of calculations was done using concrete covers provided in FDOT structure design guidelines, summarized in Table 3-4. All concrete covers are modeled using a normal distribution with a standard deviation to account for the variability of as-constructed cover. If the FDOT covers yielded overly onerous chloride migration coefficient requirements, higher cover thicknesses were examined and recommended based on the calculation results. In Table 3-4 “Mean” cover corresponds to the nominal design cover given in the Issued for Construction (IFC) drawings and “Construction Tolerance” corresponds to the maximum shortfall in cover that is permitted in the as-constructed work. The true minimum cover that is permitted in the as-constructed work is therefore the mean (or specified) cover minus the permitted construction tolerance.

Table 3-4: Initial Concrete Cover used in the Calculations.

Member	Cover for OPC and FA		
	Mean	Std. dev.	Tolerance
	in	in	in
Top surface of the deck	2.5*	0.2	0.4
Bottom surface of the deck	2	0.2	0.4
Interior and exterior pre-stressed girders	2	0.2	0.4
Barriers	2	0.2	0.4
Abutment footings, faces of abutments permanently buried	4.5	0.3	0.5

* Cover used in calculations and shown in the table excludes a 0.5-in. allowance for future milling

D3.4 Calculations

Calculations were completed using the full probabilistic model for chloride-induced corrosion from fib Bulletin 34. The model, based on Fick's 2nd law of diffusion, approximates chlorides distribution in concrete. The concrete is assumed to be a homogenous semi-infinite material with a constant diffusion coefficient and surface chloride concentration. The chloride concentration, C, at time t and distance x from the surface is given by the following expression:

$$C(x,t)=C_0+\left(C_{s,\Delta x}-C_0\right)\left(1-\mathrm{erf}\left[\frac{x-\Delta x}{2\sqrt{D_{app,C}\cdot t}}\right]\right) \tag{3-1}$$

where:

$$D_{app,C}=k_e\cdot D_{RCM,0}\cdot k_t\cdot A(t) \tag{3-2}$$

$$k_e=\exp\left(b_e\left(\frac{1}{T_{ref}}-\frac{1}{T_{real}}\right)\right) \tag{3-3}$$

$$A(t)=\left(\frac{t_0}{t}\right)^a \tag{3-4}$$

where:

- $C_{s,\Delta x}$ denotes the surface chloride concentration at a depth Δx from the surface (mass-% of total cementitious materials)
- C_0 is the initial chloride concentration (mass-% of total cementitious materials)
- $D_{app,C}$ is the apparent coefficient of chloride diffusion through concrete (mm²/year)
- Δx is the depth of the convection zone (transfer function) (mm)

- $D_{RCM,0}$ is the chloride migration coefficient (mm²/year)
- k_t is a transfer parameter (-) with a value defined by fib Bulletin 34
- b_e is a regression variable (K) with a value defined by fib Bulletin 34
- T_{ref} is the standard test temperature (K) with a value defined by fib Bulletin 34
- T_{real} is the temperature of the structural element or the ambient air (K)
- t_0 is the reference point of time (years) with a value defined by fib Bulletin 34
- a is the age factor (-)

Corrosion is assumed to be 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:

$$g(z,t_{SL})=C_{crit}-C(c,t_{SL}) \tag{3-5}$$

where c is the cover depth, t_{SL} is the target service life, C_{crit} is the chloride threshold of the reinforcement, and z is the vector of stochastic variables, such as, the concrete cover thickness, surface chloride concentration, diffusion coefficient or other variables. The limit state is to achieve the defined service life with a target confidence level of 90% (reliability index of 1.3).

The above controlling equations were programmed in a commercially available statistical analysis software, Comrel TI 8.1, which was used for calculations. A spreadsheet tool, available on the SHRP2 website, can also be used to perform such calculations, see Example 1 for additional details on the application of this tool. Table 3-5 shows a summary of input parameters and calculated maximum allowable chloride migration coefficient. As described in Section 3.3.7, in case overly onerous chloride migration coefficient requirements were yielded using FDOT concrete cover thicknesses, the cover thickness was increased at 1/2-in. increments. This was the case for the deck and the precast prestressed beams.

Table 3-5: Summary of Input Parameters and Calculated Maximum Allowable Chloride Migration Coefficient.

Target service life (years)	Temperature	Exposure Zone	Surface Concentration, Cs	Member	Cover, c			Maximum Allowed Chloride Migration Coefficient at 28 days (x10^{-12} m^2/s)		
	Normal dist.		Lognormal (σ=0.5μ)		Mean	Std. Dev.	Tolerance	OPC + 20-35% FA	OPC + 36-65% GGBS	OPC + 66-80% GGBS
	(° C)		(%wt/w_{cm})		In.	In.	In.	Age factor: μ=0.60, σ=0.15	Age factor: μ=0.40; σ=0.18	Age factor: μ=0.45; σ=0.2
100	μ = 21.0 σ = 1.24	Splash, Spray and Tidal Zone From elevation +13.21' to -5.62' for elements over water (+12' over MHW (+1.21') and -4 under MLW (-1.62')) Δx=8.9	3.5	Intermediate bents - all faces Approach Span - Bottom surface	4.5	0.30	0.50	13	2.5	2.9
50			3.5	Deck - Top and bottom surface (1/2 in. for future milling on top surface ignored in calculations)	2			2.6*		
					2.5			4.9		
			3.5	Precast prestressed beams - Reinforcement	2	0.15	0.25	3*		
					2.5			5.4		
25			3.5	Barrier	2	0.30	0.50	3.9		
100		Groundwater & Soil Δx=8.9	3	End bents	4.5	0.30	0.50	14.8	2.9	3.3
			3	Approach slab - Bottom and side surfaces	4.0			11.3	2.2	2.5

* The computed chloride migration coefficient may be difficult to achieve and therefore an increased cover thickness was considered.

D3.5 Conclusion

The final minimum requirements of concrete covers and maximum allowable chloride migrations coefficients are summarized in Table 3-6. The covers used in the structural design and shown on IFC drawings can be greater than the cover used in the durability analysis or have a smaller construction tolerance than shown below.

Prestressing strand in the precast prestressed beams has an additional 0.7 in. concrete cover compared to the carbon steel reinforcement. As shown in Table 3-5, for a 2.5-in. mean cover to the reinforcement (i.e., 3.2-in. cover to strand), the maximum allowable chloride migration coefficient is 5.4 x 10^{-12} m²/s. Using this chloride migration coefficient requirement for the precast concrete, a 3.2-in. mean cover thickness, and assuming the same chloride threshold as described in Section 3.3.4 applies to prestressing strand, a reliability index of >1.8 is computed.

Table 3-6: Minimum Requirements for the Concrete Cover and Maximum Allowable Chloride Migration Coefficient.

Member	Cover		Maximum Allowed Chloride Migration Coefficient at 28 days (x10^{-12} m²/s)		
	Mean	Tolerance	OPC + 20-35% FA	OPC + 36-65% GGBS	OPC + 66-80% GGBS
	In.	In.	Age factor: μ=0.60, σ=0.15	Age factor: μ=0.40; σ=0.18	Age factor: μ=0.45; σ=0.2
Intermediate bents - all faces Approach Span - Bottom surface	4.5	0.50	13	2.5	2.9
Deck - Top and bottom surface (1/2 in. for future milling on top surface ignored in calculations)	2.5	0.50	4.9		
Precast prestressed beams - Reinforcement	2.5	0.25	5.4		
Barrier	2	0.50	3.9		
End bents	4.5	0.50	14.8	2.9	3.3
Approach slab - Bottom and side surfaces	4.0	0.50	11.3	2.2	2.5

D3.6 References

[1] fib (2006). Model Code for Service Life Design. fib Bulletin 34. International Federation for Structural Concrete (fib), Lausanne, Switzerland, 1st edition. 126 pp.

[2] fib Bulletin 76. Benchmarking of deemed-to-satisfy provisions in standards. International Federation for Structural Concrete (fib), Lausanne, Switzerland, 2015. 189 pp.

[3] National Centers for Environmental Information, National Oceanic and Atmospheric Administration (NOAA). Accessed on Dec 11, 2018. https://www.ncdc.noaa.gov/cdo-web/

D4 Example 4 – Partial Safety Factor Method for Chloride-Induced Corrosion

Contents

D4.1 Introduction

Structural engineers are familiar with the partial safety factor (PSF) method as most structural design codes and standards, such as AASHTO LRFD, are based on this method. The method has become so common that it is now considered *the* way to perform a structural design and the method reminds structural engineers of the actual background for the method and its roots in reliability analysis. The PSF method is also referred to as a semi probabilistic design method and it is based on and calibrated against a full probabilistic design method in which the variables (load and resistance) are modelled as random variables. In such design, safety requirements and limit states are set up to ensure the desired level of reliability. The reliability index, β, expresses the probability of failure and depends on the desired reliability of the design. For example, 1 failure out of 10 trials (probability of failure = 10%) corresponds to β=1.28, whereas 1 failure out of 10^6 trials (probability of failure = 0.0001%) corresponds to β=4.75.

The PSF method is a significant simplification of the complex full probabilistic design (FPD) method. The PSF approach is a deterministic design where PSFs are used to account for the probabilistic nature of the problem; thus, rather than modelling variables as random variables they are assigned a PSF that is calibrated to provide the same reliability or greater, instead of provided from the full probabilistic design. This means that the partial factor method includes simplifications of the full probabilistic approach on the safe side. The use of the full probabilistic method can lead to more economical solutions, but it requires larger expenses for the quantification of the input parameters. The calculation itself and the PSF method is therefore the preferred method in structural codes and standards [1].

For service life design, *fib* Bulletin 34 [1] identifies two different strategies or four different approaches that can be followed when performing service life design of concrete structures:

- Strategy 1: Design to resist deterioration:
  - Level 1: Full probabilistic approach
  - Level 2: Partial safety factor design (semi-probabilistic) calibrated with level 1
  - Level 3: Deemed-to-satisfy design corresponding to recommendations in current codes and standards
- Strategy 2: Avoidance of deterioration

Level 1 and Level 2 are like the approaches described above for structural design. For service life modelling, the full probabilistic method is typically used to model chloride-induced corrosion for larger concrete structures such as bridges, tunnels, and marine structures, since this is commonly decisive for the service life of such structures. Whereas *fib* Bulletin 34 offers a well-developed method for the full probabilistic design of chloride-induced corrosion, the same is not true for the PSF design even though it would simplify the calculations considerably. The development of PSFs requires a very solid set of data that includes the combination of several exposures and requires robust calibration.

D4.1.1 Scope of document

This document presents an example of how the PSF method *could* be used in the future when sufficient data has been collected to substantiate load and resistance factors. The scope of the study is to present a PSF design for chloride-induced corrosion in agreement with *fib* Bulletin 34 based on the three design examples given in the previous sections. Based on these examples, multiple full probabilistic calculations are performed by varying chloride loading and concrete material durability resistance parameters to develop theoretical load and resistance factors to be used in the partial factor design methodology.

D4.1.2 Limitations

The PSFs given in this document are to be considered as preliminary and may be subject to change should the following limitations be exceeded. The PSF approach presented herein was developed considering a limited number of combinations of loads, resistances, and limit states as follows:

- A target service life of t_{SL} = 75 and 100 years
- A reliability index of β=1.3
- Two binder combination types: Portland Cement Type I or Type II with 20%-35% Type F FA by mass of total cementitious materials (OPC+20-35% FA) and Portland Cement Type I or Type II with 66-80% ground granulated blast-furnace slag by mass of total cementitious materials (OPC+66-80% GGBS)
- A mean temperature interval of 11.5°C – 21.0°C
- Two exposure situations: Atmospheric exposure zone and exposure to chloride from seawater and de-icing salt with a range of surface chloride concentrations considered of 2.0 – 6.0 wt.-%/c

D4.2 Partial Safety Factor Method

The aim of the PSF method is to enable a design that can be carried out as a simple calculation without additional considerations concerning the probabilistic distributions of input parameters.

Partial safety factors are a practical way to express the design values in terms of characteristic values. Partial safety factors, γ_i, can be defined as:

$$\gamma_i = \frac{x_{d,i}}{x_{c,i}} \tag{4-1}$$

where $x_{d,i}$ is the design value and $x_{c,i}$ is the characteristic value of parameter i. The design value can be computed by use of advanced statistics; however, this is outside the scope of this example. The reliability software Comrel TI 8.1 used for the analysis presented herein utilizes Equation (**4-1**) to derive PSFs for every random variable in the analysis by assuming that the characteristic value of a parameter equals the mean value.

D4.2.1 *fib* Bulletin 34 Recommendation on Partial Safety Factor Design

Even though *fib* Bulletin 34 [1] does not specifically consider the PSF design method for chloride-induced corrosion, it does provide general recommendations regarding verification by the PSF design method:

- The rules for the PSF method are given in EN 1990 Section 6 and can be used for service life design without the limitations given in EN 1990 Clause 6.2.
- The same models as for the full probabilistic design method, based on design values, shall be used for the PSF method. Simplifications on the safe side are possible.
- The PSF format separates the treatment of uncertainties and variabilities originating from various causes. The design values of the fundamental basic variables are expressed differently whether they express design values of actions, material and product properties, or geometrical quantities.
- When using the PSF method, it shall be verified that the target minimum reliability index is not exceeded when the design values for actions or effects of actions and resistances are used in the design models.

This document follows the recommendations provided by *fib* Bulletin 34 except that the suggested approach does not distinguish between whether the values express actions, material properties etc. This is further discussed in Section 4.6.

D4.3 *fib* Bulletin 34 Design Equations for Chloride-Induced Corrosion

The full probabilistic modelling of chloride-induced corrosion according to *fib* Bulletin 34 [1] is based on Fick's 2nd law of diffusion and contains improvements to yield a good approximation of chlorides distribution in concrete. The concrete is assumed to be a homogenous semi-infinite material with a constant diffusion coefficient and surface chloride concentration. The chloride concentration, C, at time t and distance x from the surface is given by the following expression:

$$C(x,t)=C_0+\left(C_{s,\Delta x}-C_0\right)\left(1-\mathrm{erf}\left[\frac{x-\Delta x}{2\sqrt{D_{app,C}\cdot t}}\right]\right) \tag{4-2}$$

where:

$$D_{app,C}=k_e\cdot D_{RCM,0}\cdot k_t\cdot A(t) \tag{4-3}$$

$$k_e=\exp\left(b_e\left(\frac{1}{T_{ref}}-\frac{1}{T_{real}}\right)\right) \tag{4-4}$$

$$A(t)=\left(\frac{t_0}{t}\right)^a \tag{4-5}$$

where:

- $C_{s,\Delta x}$ denotes the surface chloride concentration at a depth Δx from the surface (mass-% of cement).
- C_0 is the initial chloride concentration (mass-% of cement) with a value typically set by the local standard concrete specification.
- $D_{app,C}$ is the apparent coefficient of chloride diffusion through concrete (mm²/year).
- Δx is the depth of the convection zone (transfer function) (mm) with a value defined by *fib*.
- erf is the error function.
- $D_{RCM,0}$ is the chloride migration coefficient (mm²/year).
- k_t is a transfer parameter (-) with a value defined by *fib*.
- b_e is a regression variable (K) with a value defined by *fib*.
- T_{ref} is the standard test temperature (K) with a value defined by *fib*.
- T_{real} is the temperature of the structural element or the ambient air (K).
- t_0 is the reference point of time (years) with a value defined by *fib.*
- a is the age factor (-) with a value defined by *fib* and depends on the type of cementitious materials.

These variables are modelled according to recommendations given in *fib* Bulletin 34. Many variables are treated as stochastic parameters except C_0, k_t, t_0, and T_{ref}, which are deterministic. The Δx is also deterministic in cases where the convection zone is not considered (i.e., off). Some of the variables depend on the details of the specific design (such as $C_{s,\Delta x}$, $D_{RCM,0}$, and T_{real}), while other variables attain the same value regardless of the design. Table 4-1 lists values used for parameters that typically do not depend on the details of the design.

Table 4-1: Values used for *fib* Bulletin 34 Parameters that Do Not Typically Vary with the Details of the Design.

Variable		Distribution	Unit	Mean	Standard Deviation and Function Parameters
C_o		Deterministic	Mass-% of total cementitious materials	0.1*	-
Δx	Δx on (convection zone considered)	Beta	mm (in)	8.9 (0.35)	σ = 5.6 (0.22) A=0; B=50
	Δx off (convection zone not considered)	Deterministic	mm	0	-
T_{ref}		Deterministic	°C (°F)	20 (68)	-
k_t		Deterministic	-	1	-
b_e **		Normal	K	4800	700
t_0		Deterministic	years	0.0767	-
C_{crit}		Beta	Mass-% of total cementitious materials	0.6	σ = 0.15 A=0.2; B=2

* Typically used value, however, other values may be assigned based on the local standard concrete specification.

** 3500 K < b_e < 5500 K

D4.3.1 Limit State Function

Corrosion is assumed to be 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:

$$g(z,t_{SL})=C_{crit}-C(c,t_{SL}) \tag{4-6}$$

where c is the cover depth, t_{SL} is the target service life, C_{crit} is the chloride threshold of the reinforcement, and z is the vector of stochastic variables, such as, the concrete cover thickness, surface chloride concentration, diffusion coefficient or other variables. It is noted that $C(c,t_{SL})$ is computed using Equation (4-2).

This limit state is to achieve the defined service life with a target confidence level of 90% (reliability index of 1.3) per *fib* Bulletin 34.

The following parameters are defined in the design phase by the designer to obtain the desired reliability index:

- $C_{S,\Delta x}$ - surface chloride concentration
- $D_{RCM,0}$ - chloride migration coefficient
- c - cover depth
- T_{real} - temperature of the structural element or the ambient air
- a - the age factor

D4.4 Development of Partial Safety Factors for Design Examples

The development of a general PSF method for chloride-induced corrosion is possible, provided that sufficiently long-term experience is gained, or a sufficient amount of data becomes available for a calibration. The development of such general method is outside the scope of this document and for this example, the limitations listed in Section 4.1.2 have therefore been made.

It should be emphasized that the PSFs developed in this study are to be considered as preliminary since, even with the limitations in Section 4.1.2, they build on a limited data set and are based on theoretical structures rather than measurements from actual structures. However, the procedure outlined in the following sections provides insight into a potential PSF design method for chloride-induced reinforcement corrosion and the steps and considerations necessary in the development of suitable PSFs.

D4.4.1 Methodology for Developing Partial Safety Factors

Figure 4-1 shows a flowchart of the methodology adopted for this example to develop PSFs. As seen on the flow chart, the variables t_{SL}, T_{real}, and Δx are fixed at predetermined values in order to limit the necessary number of full probabilistic calculations, whereas c and $C_{s,\Delta x}$ are varied with preset reasonable ranges to investigate their influence on the PSFs.

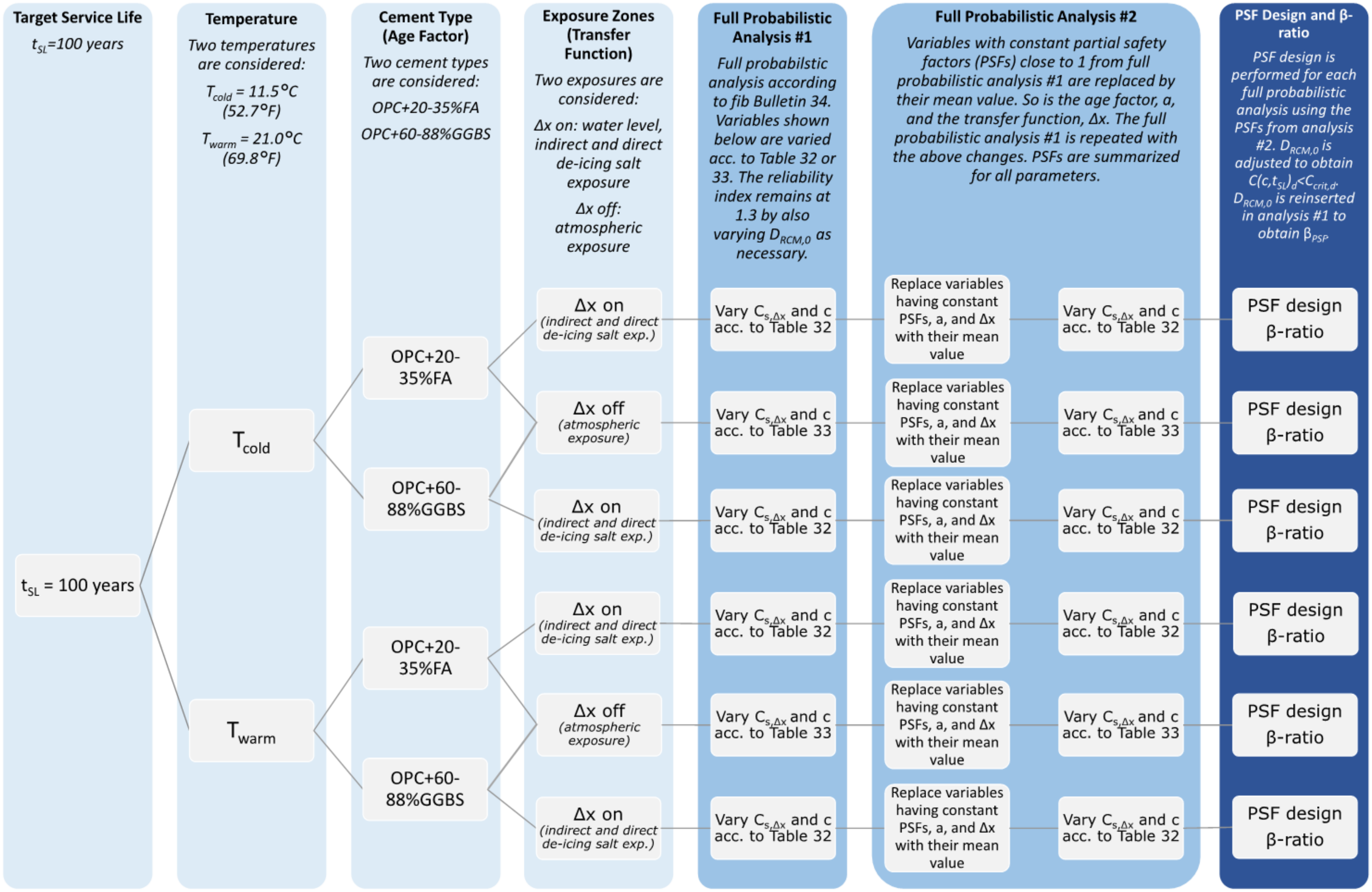


Figure 4-1 Overview of Procedure used to Develop Partial Safety Factors (PSFs).

The methodology is based on a target service life of 100 years and two mean temperatures that correspond to the mean temperatures from the other design examples in Sections 1-3 of this appendix. Table 4-2 shows how the temperatures are modelled as stochastic variables in the full probabilistic analysis.

Table 4-2: Input for Modelling of Temperatures as Stochastic Variables in Full Probabilistic Analysis.

Variable		Distribution	Unit	Mean	Standard Deviation and Function Parameters
T_{real}	T_{cold}	Normal	°C (°F) (K)	11.5 (52.7) (284.7)	2.0 (3.6) (2.0)
	T_{warm}	Normal	°C (°F) (K)	21.0 (69.8) (294.2)	1.24 (2.2) (1.24)

For each temperature, two different types of cement are considered: Portland Cement Type I or Type II with 20%-35% Type F FA by mass of total cementitious materials (OPC+20-35% FA) and Portland Cement Type I or Type II with 66-80% ground granulated blast-furnace slag by mass of total cementitious materials (OPC+66-80% GGBS). For each cement type, two exposures are considered: Δx "on" and Δx "off", which correspond to the two fundamentally different exposure zones typically considered in service life design. Δx "on" includes the convection zone in the analysis whereas Δx "off" does not. The convection zone is typically included for zones exposed directly or indirectly to de-icing salts and for water level zones, whereas it is not included for atmospheric zones [1]. According to *fib* Bulletin 34, the type of cement in combination with the exposure zone controls the age factor to be used for the full probabilistic analysis. Table 4-3 shows the age factors considered for this example, extracted from *fib* Bulletin 34.

Table 4-3: Age factors as Stochastic Variables for Full Probabilistic Analysis.

Concrete mixes	Distribution	Δx on Water Level, Direct and Indirect De-icing Salts Zones		Δx off Atmospheric Zone	
		Mean (μ)	Parameters	Mean (μ)	Parameters
OPC+20-35%FA	Beta	0.60	σ=0.15, A=0; B=1	0.65	σ=0.15, A=0; B=1
OPC+66-80%GGBS	Beta	0.45	σ=0.2, A=0; B=1		

For the atmospheric zone, the age factor is independent on the type of cement; thus, for each temperature, three age factors are considered in the present example. With two mean temperatures (see Table 4-2), this gives a total of six different combinations of temperature, cement type, and exposure zone for the present example, also seen on Figure 4-1.

With all of the above assumptions, only three further parameters remain as variable according to *fib* Bulletin 34 and Equations (4-2)-(4-4): the cover depth, c, the surface chloride content, $C_{s,\Delta x}$, and the chloride migration coefficient, $D_{RCM,0}$. In the full probabilistic analysis, $D_{RCM,0}$ is varied along with cover depth to obtain a reliability index of β=1.3 and therefore this variable should not be fixed in advance. The chloride migration coefficient is modelled as a normal distribution with a COV of 0.20. Left to vary is the cover depth and the surface chloride content. Typical values for those variables depend on the exposure zone and therefore the following analysis will distinguish between whether Δx is "on" or "off". Table 4-4 and Table 4-5 show the values considered for c and $C_{s,\Delta x}$ in the development of PSFs. The values in each table should be combined in any possible way such that, for each table, there is a total of 3x3=9 combinations.

Table 4-4: Cover Depths, c, and Surface Chloride Contents, $C_{s,\Delta x}$, Used for Examination of Indirect and Direct De-icing Salt Exposure Zone (Δx "on").

Variable	Distribution	Value #1		Value #2		Value #3	
		Mean [μ]	Std. dev. [σ]	Mean [μ]	Std. dev. [σ]	Mean [μ]	Std. dev. [σ]
c [mm]	Normal	50	10	70	10	90	20
$C_{s,\Delta x}$ [wt-%/c]	Log-normal	2.0	1.0	4.0	2.0	6.0	3.0

Table 4-5: Cover Depths, c, and Surface Chloride Contents, $C_{s,\Delta x}$, Used for Examination of Atmospheric Exposure Zone (Δx "off").

Variable	Distribution	Value #1		Value #2		Value #3	
		Mean [μ]	Std. dev. [σ]	Mean [μ]	Std. dev. [σ]	Mean [μ]	Std. dev. [σ]
c [mm]	Normal	30	10	50	10	70	10
$C_{s,\Delta x}$ [wt-%/c]	Log-normal	1.0	0.5	2.0	1.0	3.5	1.75

The full probabilistic analysis and the PSF design is carried out in the following steps:

- **Full probabilistic design #1:** FPD analysis is done according to *fib* Bulletin 34 and in agreement with Equations (4-2)-(4-4). Variables are modelled in accordance with Table 4-1-Table 4-5. For each combination, $D_{RCM,0}$ is varied such that the reliability index is β=1.3. In practice, β-values between 1.29-1.32 are considered acceptable. The actual reliability index for each analysis is denoted β_{FPD}. PSFs for each random variable are determined as described in Section 4.2. The mean and representative quartiles for the PSFs are summarized.
- **Full probabilistic design #2**: To limit the number of variables to which PSFs need be applied, any variables found to have constant PSFs, close to 1, from the full probabilistic design #1 analysis are replaced by a deterministic variable, equal to its mean value. Also, Δx is assigned a PSF of 1. The PSF on $D_{app,C}$ is determined by replacing the age factor with a deterministic value equal to its mean value such that the only random variable in the determination of $D_{app,C}$ is the chloride migration coefficient, $D_{RCM,0}$. This is further justified in Section 4.4.4. In this step the remaining variables are modelled in accordance with Table 4-1-Table 4-5 and all calculations described in full probabilistic design #1 are repeated. Again, $D_{RCM,0}$ is varied such that the reliability index is β=1.3, with values between 1.29-1.32 considered acceptable. Partial safety factors for all remaining random variables are determined. The mean and representative quartiles for the PSFs are summarized.
- **PSF design and β-ratio**: Based on the determined PSFs from the reduced random variables, PSF design is carried out in accordance with recommendations given in Section 4.4.4. $D_{RCM,0}$ is optimized such that the design chloride content at the rebar level at the target service life reaches (or is close to) the design critical chloride content. As a check of the PSF approach developed, the optimized value of $D_{RCM,0}$ is reinserted into the original full probabilistic design #1 analysis to determine the corresponding reliability index, β_{PSF}. This reliability index is then compared to the original reliability index from the full probabilistic design #1 analysis as follows:

$$\beta\text{-ratio}=\frac{\beta_{PSF}}{\beta_{FPD}} \tag{4-7}$$

If the β-ratio is greater than 1, the PSF design is on the safe side, whereas the opposite is true if the β-ratio is less than 1. The PSFs are calibrated such that β-ratio > 1 for all considered combinations.

D4.4.2 Findings from Full Probabilistic Analysis #1

Analysis #1 is a full probabilistic analysis carried out in agreement with *fib* Bulletin 34. All input parameters are modelled in accordance with Table 4-1-Table 4-5 and the chloride migration coefficient, $D_{RCM,0}$, is adjusted such that a reliability index of 1.3 is obtained. A total of 54 analyses are carried out which correspond to nine different analyses for each of the six combinations shown on Figure 4-1. Table 4-6 shows the input and result for one such analysis considering T_{cold}, OPC+20-35%FA, Δx "on", c(mean) = 50 mm, and $C_{s,\Delta x}$(mean) = 2.0 wt-%/c.

Table 4-6: Example of Full Probabilistic Analysis #1 for the Combination: T_{cold}, OPC+20-35%FA, Δx "on", c(mean) = 50 mm, and $C_{s,\Delta x}$(mean) = 2.0 wt-%/c. t_{SL} = 100 years.

Variable	Distribution	Input parameters	Partial Safety Factor
c [mm]	Normal	μ = 50, σ = 10	0.90
T_{real} [K]	Normal	μ = 284.65, σ = 2.0	1.00
$C_{s,\Delta x}$ [wt-%/c]	Log-normal	μ = 2.0, σ = 1.0	1.08
$D_{RCM,0}$ [mm²/yr]	Normal	μ = 138.6, σ = 27.7	1.03
a [-]	Beta	μ = 0.6, σ = 0.15 A = 0, B = 1	0.73
C_0 [wt-%/c]	Deterministic	0.1	-
Δx [mm]	Beta	μ = 8.9, σ = 5.6 A = 0, B = 50	1.10
k_t [-]	Deterministic	1	-
b_e [K]	Normal	μ = 4800, σ = 700	0.99
t_0 [years]	Deterministic	0.0767	-
T_{ref} [K]	Deterministic	293	-

Variable	Distribution	Input parameters	Partial Safety Factor
C_{crit} [wt-%/c]	Beta	$\mu = 0.60$, $\sigma = 0.15$ A = 0.2, B = 2	0.91
β_{FPD}	1.30		

The PSFs in Table 4-6 are based on the computed PSFs determined as described in Section 4.2 from the full probabilistic analysis. Figure 4-2 shows an example of these computed PSFs.

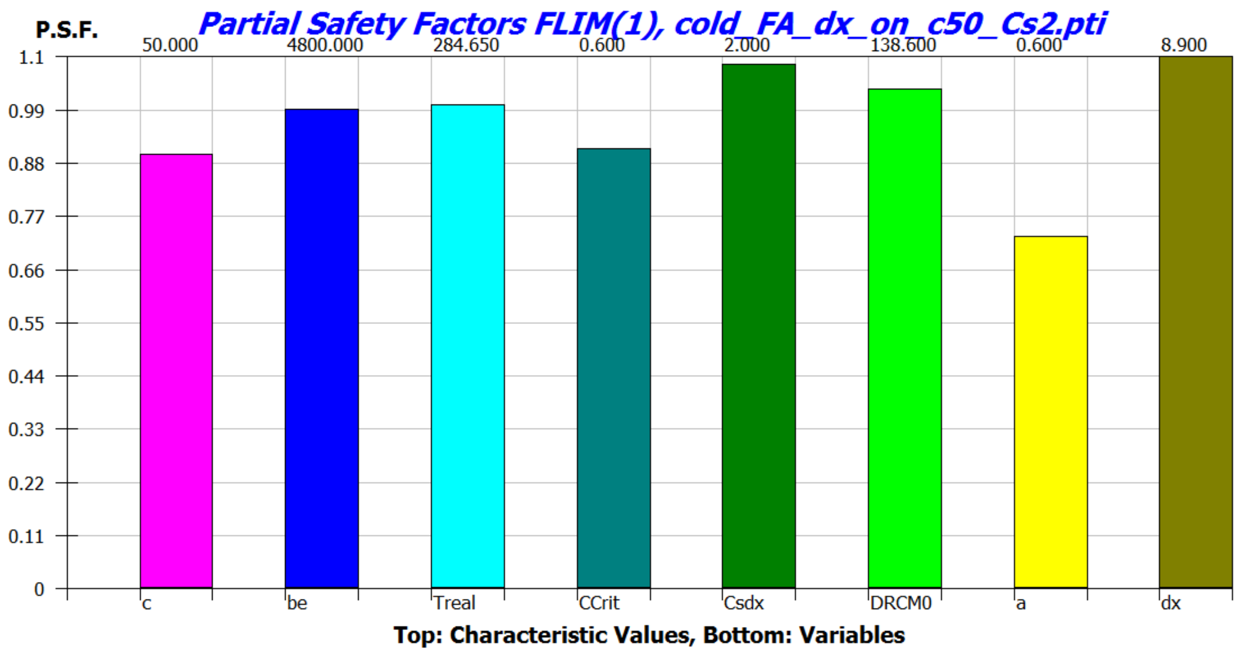


Figure 4-2: Example of Partial Safety Factors from the Full Probabilistic Analysis performed in COMREL.

D4.4.2.1 Summary of Partial Safety Factors from Full Probabilistic Analysis #1

Table 4-7 shows a summary of the mean PSFs from the full probabilistic analysis #1 for all input parameters modelled as random variables. The table distinguishes between the two main temperatures (T_{cold} and T_{warm}), the two different cement types (OPC+20-35% FA and OPC+66-80% GGBS), and the two different exposure zones considered (Δx "on" and Δx "off").

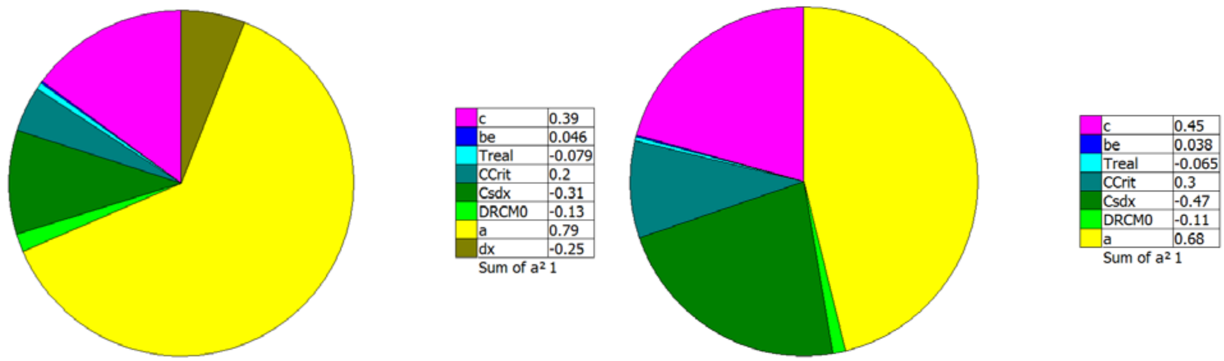
Table 4-7: Partial Safety Factors from Full Probabilistic Analysis #1. Values are Shown as Mean Values of All 9 Analyses for Each Temperature/Cement Type Combination.

fib 34 Para-meter	T_{cold}			T_{warm}		
	OPC+20-35%FA Δx on	OPC+66-80%GGBS Δx on	OPC+20-35%FA/OPC+66-80%GGBS Δx off	OPC+20-35%FA Δx on	OPC+66-80%GGBS Δx on	OPC+20-35%FA/OPC+66-80%GGBS Δx off
c	0.91	0.91	0.89	0.91	0.91	0.89
T_{real}	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**
$C_{s,\Delta x}$	1.04	1.04	1.11	1.04	1.04	1.11
$D_{RCM,0}$	1.04	1.04	1.03	1.04	1.04	1.03
a	0.71	0.46	0.74	0.71	0.46	0.74
Δx	1.04	1.04	-	1.05	1.04	-
b_e	**0.99**	**0.99**	**0.99**	**1.00**	**1.00**	**1.00**
C_{crit}	0.92	0.92	0.90	0.92	0.92	0.90

Table 4-7 shows that the difference between the PSFs determined for a range of temperatures from T_{cold} to T_{warm} is minor and in the following, the results relating to both temperatures will be considered together. Another characteristic from Table 4-7 is that the mean value of the PSF of T_{real} and b_e is approximately 1 regardless of the type of cement and exposure zone. Looking further at the data behind the numbers in Table 4-7, it is observed that T_{real} and b_e is approximately 1 in all of the 54 full probabilistic analyses.

D4.4.3 Variables Governing Service Life Design from Full Probabilistic Analysis #1

From the service life modelling in COMREL, an output from COMREL directly shows the relative influence each input variable has on the overall reliability of the design. Figure 4-3 shows the governing variables for two scenarios: one scenario including the transfer function and one scenario not including the transfer function. The examples relate to OPC+20-35% FA but similar conclusions are found for the analyses based on OPC+66-80% GGBS and for all examined cover depths and surface chloride contents. Of the parameters defined by the designer during the design phase it is seen that the age factor, a, the cover depth, c, the surface chloride content, $C_{s,\Delta x}$, and the critical chloride content, C_{crit}, for both exposure zones have a great influence on the calculated reliability, whereas the temperature, T_{real}, and the regression coefficient, b_e, have a minor influence.



(a) Δx on **(b) Δx off**

Figure 4-3: Relative Influence of Input Parameters to the fib Bulletin 34 Service Life Model for Two Different Exposure Conditions with Cement Type OPC+20-35%FA.

D4.4.4 Findings from Full Probabilistic Analysis #2

Analysis #2 is carried out as a full probabilistic analysis with a, Δx, b_e, and T_{real} treated as deterministic values. Remaining input parameters are modelled in accordance with Table 4-1 and the chloride migration coefficient, $D_{RCM,0}$, is adjusted such that a reliability index of 1.3 is obtained. A total of 54 analyses are carried out which correspond to nine different analyses for each of the six combinations shown on Figure 4-1. Table 4-8 shows the input and result for the same example as illustrated in Table 4-6.

Table 4-8: Example of Full Probabilistic Analysis #2 for the Combination: T_{cold}, OPC+20-35%FA, Δx on, c(mean) = 50 mm, and $C_{s,\Delta x}$(mean) = 2.0 wt-%/c. t_{SL} = 100 years.

Variable	Distribution	Input parameters	Partial Safety Factor
c [mm]	Normal	μ = 50, σ = 10	0.80
T_{real} [K]	Deterministic	284.65	-
$C_{s,\Delta x}$ [wt-%/c]	Log-normal	μ = 2.0, σ = 1.0	1.21
$D_{RCM,0}$ [mm²/yr]	Normal	μ = 138.6, σ = 27.7	2.41*
a [-]	Deterministic	0.6	-
C_0 [wt-%/c]	Deterministic	0.1	-
Δx [mm]	Deterministic	8.9	-
k_t [-]	Deterministic	1	-

Variable	Distribution	Input parameters	Partial Safety Factor
b_e [K]	Deterministic	4800	-
t_0 [years]	Deterministic	0.0767	-
T_{ref} [K]	Deterministic	293	-
C_{crit} [wt-%/c]	Beta	μ = 0.60, σ = 0.15 A = 0.2, B = 2	0.87
β	1.30		

* In the full probabilistic analysis #2, several random input parameters are replaced by their mean value. This causes the reliability index to change from 1.3 when using the same random variables as in the full probabilistic analysis #1. Because the PSFs determined in COMREL depend on the reliability index, a reliability index of β=1.3 needs to remain and this is done by also varying $D_{RCM,0}$ in the full probabilistic analysis #2. The ratio between the $D_{RCM,0}$ mean value used in the full probabilistic analysis #1 and #2 is then included in the PSF for $D_{RCM,0}$ by simply multiplying the PSF from COMREL with this ratio. This is necessary to link $D_{RCM,0}$ to the original value found in analysis #1, which can be considered the 'real' value.

All PSFs from the full probabilistic analysis #2 are summarized in Table 4-9. The table shows the mean value of the PSFs as well as either the 25% fractile value (=first quartile for PSFs less than 1) or the 75% fractile value (=third quartile for PSFs greater than 1). The PSF design must be on the safe side compared to the full probabilistic design and therefore the quartiles are also considered since those values provide more safety than using the mean values.

Table 4-9: Partial Safety Factors (PSFs) from Full Probabilistic Analysis #2 based on β=1.3 and t_{SL} = 100 Years. Values are Shown as Mean and 25% Fractile (for PSF<1) or 75% Fractile (for PSF>1) for Each Cement Type and Exposure Zone Considered.

fib 34 Parameter	Δx on (Water Level, Indirect and Direct De-icing Salts Zones)				Δx off (Atmospheric Zone)	
	OPC+20-35%FA		OPC+66-80%GGBS		OPC+20-35%FA/ OPC+66-80%GGBS	
	Mean	Quartile	Mean	Quartile	Mean	Quartile
c	0.80	0.77	0.80	0.77	0.78	0.69
$C_{s,\Delta x}$	1.17	1.22	1.17	1.22	1.26	1.33
$D_{RCM,0}$	2.60	2.66	3.78	3.86	2.22	2.48
C_{crit}	0.88	0.87	0.88	0.87	0.87	0.85

D4.4.5 Partial Safety Factor Design Method

The semi probabilistic counterpart to the full probabilistic limit state function in Equation **(4-5)** is given by the following expression:

$$C(c,t_{SL})_d \leq C_{crit,d} \quad (4\text{-}8)$$

where $C(c,t_{SL})_d$ is the design value of the chloride content at the cover depth at the target service life and $C_{crit,d}$ is the design value of the critical chloride content.

The results in Table 4-9 indicate that the PSF for $C_{crit} \neq 1$ and therefore a factored value, $C_{crit,d}$ is defined as follows:

$$C_{crit,d}=\gamma_{C_{crit}} \cdot C_{crit,c} \quad (4\text{-}9)$$

where:

- γ_{Ccrit} is the PSF of the critical chloride content; and,
- $C_{crit,c}$ is the characteristic mean value of the critical chloride content based on *fib* Bulletin 34. For carbon steel the recommended value is $C_{crit,c}(\mu) = 0.6$.

Several variables serve as an input to the determination of the chloride content $C(c,t_{SL})$ as shown in Equations **(4-2)-(4-4)**; however some of these are lumped into the apparent diffusion coefficient, $D_{app,C}$, that takes into account that chloride diffusion is not constant but time dependent. In general, the apparent diffusion coefficient is a constant average value representing the materials diffusion resistance from the start of exposure, t_0, to the time considered, t. Section 4.3 shows that $D_{app,C}$ is a linear function of A(t) and A(t) function of time to the power a (age factor). To limit the number of necessary PSFs, a PSF can be applied directly to $D_{app,C}$ rather than to several of the input parameters to $D_{app,C}$. From the findings of the full probabilistic analysis #1 it is clear that the temperature T_{real}, and the regression coefficient, b_e (that are both part of the definition of $D_{app,C}$) have a minor influence on the overall reliability of the service life model. Table 4-7 shows that regardless of the combination of cover depth, surface chloride concentration, cement type, and mean temperature, the PSF of T_{real} and b_e are both consistently ≈1. Therefore, these variables are simply replaced by their mean values. The mean value of T_{real} depends on the design but the mean value of b_e is 4800 K regardless of the design. As seen on Figure 4-3, the age factor, a, has a significant influence on the overall reliability of the evaluation of $C(c,t_{SL})$; however, because $D_{app,C}$ is a non-linear expression with the age factor, a, being the non-linear term, the PSF should not be applied directly on the age factor. This is because the PSF would then not have the same influence on $D_{app,C}$ at different points in time. Instead, the variability of $D_{app,C}$ is brought into the assessment of PSFs through the chloride migration coefficient, $D_{RCM,0}$. Because $D_{app,C}$ is linear with $D_{RCM,0}$, a PSF applied to $D_{RCM,0}$ can be considered a PSF applied to $D_{app,C}$. In the full probabilistic analysis #2, all other parameters than $D_{RCM,0}$ in the expression for $D_{app,C}$ are set equal to their mean value and the PSF for $D_{RCM,0}$ is determined by including the ratio between the mean values of $D_{RCM,0}$ from analysis #1 and #2 as described in Table 4-8. Thus, the determination of the PSF on $D_{RCM,0}$ lumps the uncertainty on all of the input parameters to $D_{app,C}$ into that of $D_{RCM,0}$; therefore, more accurate to apply the PSF to $D_{app,C}$.

To minimize the number of variables that are assigned a PSF and because Δx is already assigned a predetermined mean value according to the full probabilistic design in *fib* Bulletin 34 (see Table 4-1), Δx is also replaced by its mean value in the semi probabilistic design equations.

The cover depth, c, and the surface chloride content, $C_{s,\Delta x}$, are assigned a PSF because they contribute significantly to the reliability index according to Figure 4-3 and because they are both measurable and quantified by the designer.

The design input values ($C_{s,\Delta x,d}$, c_d, and $D_{app,c,d}$) for determining $C(c,t_{SL})_d$ become:

$$C_{s,\Delta x,d}=\gamma_{C_s}\cdot C_{s,\Delta x,c} \tag{4-10}$$

$$c_d=\gamma_c\cdot c_c \tag{4-11}$$

$$D_{app,C,d}=\gamma_{D_{app,C}}\cdot D_{app,C} \tag{4-12}$$

where the characteristic values c_c and $C_{s,\Delta x,c}$ are equal to the mean value of the distribution recommended by *fib* Bulletin 34 and the characteristic value $D_{app,C}$ is equal to Equation (**4-3**) with all input parameters replaced by deterministic values equal to their mean value. $\gamma_{D_{app,c}}$ is the PSF on the chloride migration coefficient, γ_c is the PSF on the cover, and γ_{C_s} is the PSF on the surface chloride content.

The design equations for the PSF method are similar to those for the full probabilistic design except that the input parameters are now all deterministic:

$$C(c,t_{SL})_d=C_0+\left(C_{S,\Delta x,d}-C_0\right)\left(1-\mathrm{erf}\left[\frac{c_d-\Delta x}{2\sqrt{D_{app,C,d}\cdot t_{SL}}}\right]\right) \tag{4-13}$$

where:

$$D_{app,C,d}=\gamma_{D_{app,C}}\cdot k_e\cdot D_{RCM,0}\cdot k_t\cdot A(t) \tag{4-14}$$

$$k_e=\exp\left(b_e\left(\frac{1}{T_{ref}}-\frac{1}{T_{real}}\right)\right) \tag{4-15}$$

$$A(t)=\left(\frac{t_0}{t}\right)^a \tag{4-16}$$

In the above equations, Δx, b_e, T_{real}, and a are given as the mean value defined in *fib* Bulletin 34. Table 4-10 summarizes these values for the cement types and exposure conditions considered in the present PSF design.

Table 4-10: Deterministic Values of Δx, b_e, T_{real}, and a to be used in Partial Safety Factor Design.

Input Parameter	Δx on (Water Level, Indirect and Direct De-icing Salts Zones)		Δx off (Atmospheric Zone)
	OPC+20-35%FA	OPC+66-80%GGBS	OPC+20-35%FA/ OPC+66-80%GGBS
Δx [mm]	8.9	8.9	0
b_e [K]	4800	4800	4800
T_{real} [K]	Mean temperature (project specific)	Mean temperature (project specific)	Mean temperature (project specific)
a	0.60	0.45	0.65

D4.4.6 Partial Safety Factors

In Table 4-9, the PSFs are developed from a trial-and-error approach. The method described in Section 4.4.7 is tried for different combinations of the PSFs and the combination providing the required level of safety for all analyses (β-ratios greater than 1 according to Equation (**4-7**) is chosen. The PSF values are mainly based on the quartile rather than the mean values in Table 4-9, since using only the mean values provided an insufficient level of safety. Table 4-11 shows the PSFs recommended for the PSF design method in the present study.

Table 4-11: Partial Safety Factors Calibrated to a Reliability Index of 1.3 at a Target Service Life of 75 or 100 Years.

Partial Safety Factor	Δx on (Water Level, Indirect and Direct De-icing Salts Zones)		Δx off (Atmospheric Zone)
	OPC+20-35%FA	OPC+66-80%GGBS	OPC+20-35%FA/OPC+66-80%GGBS
γ_{C}	0.75		0.75
$\gamma_{Cs,\Delta x}$	1.20		1.30
$\gamma_{Dapp,C}$*	2.65	3.85	2.50
γ_{Ccrit}	0.90		0.85

* Because $D_{app,C}$ is a linear function of $D_{RCM,0}$, the PSF determined for $D_{RCM,0}$ in the full probabilistic analysis #2 can also be considered a PSF for the entire expression for $D_{app,C}$ as described in Section 4.4.5.

The PSFs in Table 4-11 have been quantified for a reliability of β=1.3 at a target service life of 100 years with respect to the limit state "depassivation of reinforcement due to chlorides". If a higher reliability is desired, the PSFs must be modified accordingly.

D4.4.6.1 Partial Safety Factor for Target Service Life of 75 Years

The PSFs in Table 4-11 relate to a target service life of 100 years, which is often the preferred target service life for large infrastructure projects such as bridges. For less prestigious projects, such as the highway bridge considered in the first design example in Section 1, a lower target service life of 75 years is sometimes used. To investigate how the PSFs change when the target service life is reduced to 75 years the above calculations are rerun with the only change that the target service life is now 75 years instead of 100 years. Also, because the above calculations indicated that the PSFs were the same for T_{cold} and T_{warm}, only T_{cold} is considered. The PSFs relating to a target service life of 75 years are shown in Table 4-12.

Table 4-12: Partial Safety Factors (PSFs) from Full Probabilistic Analysis #2 based on β=1.3 and t_{SL} = 75 years. Values are Shown as Mean and 25% Fractile (for PSF<1) or 75% Fractile (for PSF>1) for Each Cement Type and Exposure Zone Considered.

fib 34 Parameter	Δx on (Water Level, Indirect and Direct De-icing Salts Zones)				Δx off (Atmospheric Zone)	
	OPC+20-35%FA		OPC+66-80%GGBS		OPC+20-35%FA/ OPC+66-80%GGBS	
	Mean	Quartile	Mean	Quartile	Mean	Quartile
c	0.81	0.77	0.80	0.77	0.79	0.69
$C_{s,\Delta x}$	1.17	1.21	1.17	1.22	1.26	1.33

fib 34 Parameter	Δx on (Water Level, Indirect and Direct De-icing Salts Zones)				Δx off (Atmospheric Zone)	
	OPC+20-35%FA		OPC+66-80%GGBS		OPC+20-35%FA/ OPC+66-80%GGBS	
	Mean	Quartile	Mean	Quartile	Mean	Quartile
$D_{RCM,0}$	2.43	2.53	3.54	3.74	2.14	2.39
C_{crit}	0.88	0.87	0.88	0.87	0.86	0.84

Compared with the values in Table 4-9, the PSFs characterizing t_{SL} = 75 years and t_{SL} = 100 years are similar with only minor differences. Therefore, the PSFs in Table 4-11 are recommended for target service lives of both 75 and 100 years.

D4.4.7 Partial Safety Factor Design and β-Ratios

The same analyses that were carried out as full probabilistic analyses are now repeated by use of the PSF design. The PSF design is based on Equation (**4-8**), where design values of the chloride content at rebar level, $C(c,t)_d$, and the critical chloride content, $C_{crit,d}$, are compared. At the target service life year, the chloride content at rebar level should be less than the critical chloride content. In the PSF design, the chloride migration coefficient, $D_{RCM,0}$, is optimized such that, at the target service life year, $C(c,t_{SL})_d$ is equal (or close) to $C_{crit,d}$.

An example of the PSF method is shown by studying the same example as considered in Table 4-6 and Table 4-8 relating to the combination T_{cold}, OPC+20-35%FA, Δx on, c(mean) = 50 mm, and $C_{s,\Delta x}$(mean) = 2.0 wt-%/c. Table 4-13 shows the input parameters to the PSF design. These are all based in Table 4-8 and Table 4-11.

Table 4-13: Example of Input Parameters to a Partial Safety Factor Design and the Corresponding Full Probabilistic Analysis.

Variable	Partial Safety Factor Design		Full Probabilistic Analysis #1 with Chloride Migration Coefficient Determined in Partial Safety Factor Design	
	Distribution	Input Parameters	Distribution	Input Parameters
c [mm]	Deterministic	50 γ_c = 0.75	Normal	μ = 50, σ = 10
T_{real} [K]	Deterministic	284.65	Normal	μ = 284.65, σ = 2.0
$C_{s,\Delta x}$ [wt-%/c]	Deterministic	2.0 $\gamma_{Cs,\Delta x}$ = 1.20	Log-normal	μ = 2.0, σ = 1.0
$D_{RCM,0}$ [mm²/yr]	Deterministic	108.0 $\gamma_{Dapp,C}$ = 2.65	Normal	μ = 108.0, σ = 21.6

Variable	Partial Safety Factor Design		Full Probabilistic Analysis #1 with Chloride Migration Coefficient Determined in Partial Safety Factor Design	
	Distribution	Input Parameters	Distribution	Input Parameters
a [-]	Deterministic	0.6	Beta	μ = 0.6, σ = 0.15 A = 0, B = 1
C_0 [wt-%/c]	Deterministic	0.1	Deterministic	0.1
Δx [mm]	Deterministic	8.9	Beta	μ = 8.9, σ = 5.6 A = 0, B = 50
k_t [-]	Deterministic	1	Deterministic	1
b_e [K]	Deterministic	4800	Normal	μ = 4800, σ = 700
t_0 [years]	Deterministic	0.0767	Deterministic	0.0767
T_{ref} [K]	Deterministic	293	Deterministic	293
C_{crit} [wt-%/c]	Deterministic	0.60 γ_{Ccrit} = 0.90	Beta	μ = 0.60, σ = 0.15 A = 0.2, B = 2
β_{PSF}	-		1.46	

The PSF design is based on Equations (4-8)-(4-16). First, the relevant design values are determined:

$$C_{crit,d} = 0.90 \cdot 0.60 \text{ wt-\%/c} = 0.540 \text{ wt-\%/c} \tag{4-17}$$

$$c_d = 0.75 \cdot 50 \text{ mm} = 37.5 \text{ mm} \tag{4-18}$$

$$C_{s,\Delta x,d} = 1.20 \cdot 2.0 \text{ wt-\%/c} = 2.40 \text{ wt-\%/c} \tag{4-19}$$

Subsequently, the design value of the apparent diffusion coefficient is determined. The value of the chloride migration coefficient used in the calculations should be optimized such that Equation (4-8) is fulfilled with the greatest possible value $D_{RCM,0}$. The following calculations use the already optimized value of $D_{RCM,0}$:

$$A(t) = \left(\frac{0.0767 \text{ years}}{100 \text{ years}}\right)^{0.60} = 0.0135 \tag{4-20}$$

$$k_e = \exp\left(4800\text{K}\left(\frac{1}{293\text{K}} - \frac{1}{284.65\text{K}}\right)\right) = 0.618 \tag{4-21}$$

$$D_{app,C,d} = 2.65 \cdot 0.618 \cdot 108.0 \frac{\text{mm}^2}{\text{yr}} \cdot 1 \cdot 0.0135 = 2.39 \frac{\text{mm}^2}{\text{yr}} \tag{4-22}$$

Finally, the design value of the chloride content at the depth of the rebar at the target service life year can be calculated as:

$$C(c,t_{SL})_d = 0.1\text{wt-\%/c} + (2.40\text{wt-\%/c} - 0.1\text{wt-\%/c})\left(1-\text{erf}\left[\frac{37.5\text{mm}-8.9\text{mm}}{2\sqrt{2.39\frac{\text{mm}^2}{\text{yr}}\cdot 100\text{yr}}}\right]\right) = 0.539\text{wt-\%/c} \tag{4-23}$$

This design value is compared to the design value of the critical chloride content:

$$C(c,t_{SL})_d = 0.539\text{wt-\%/c} \leq C_{crit,d} = 0.540\text{wt-\%/c} \tag{4-24}$$

The equation is fulfilled, and the service life design is valid.

The PSF design is compared to the original full probabilistic service life design by determining the reliability index that the chloride migration coefficient determined from the PSF design would yield if that had been used in the full probabilistic design. Table 4-13 shows the input parameters to the full probabilistic analysis when using the chloride migration coefficient, $D_{RCM,0}$, from the PSF design. Rerunning the original full probabilistic analysis #1 with the new chloride migration coefficient gives a reliability index of β_{PSF} = 1.46. Compared to the reliability index from the original full probabilistic design shown in Table 4-6, the β-ratio becomes:

$$\beta\text{-ratio} = \frac{\beta_{PSF}}{\beta_{FPD}} = \frac{1.46}{1.30} = 1.12 \tag{4-25}$$

Because the β-ratio is greater than 1, the PSF design is on the safe side. The same is true for all remaining PSF designs of the combinations shown on Figure 4-1. All β-ratios are however less than 1.30, which means that even though the PSFs design is on the safe side it yields a maximum reliability index of 1.7. Thus, the PSFs in Table 4-11 are accepted to provide the necessary conservatism compared to the full probabilistic design.

D4.5 Application of Partial Safety Factor Method on Design Examples

The PSF method developed in the previous sections are now applied to the three design examples considered in this appendix:

- Design example 1: New York, multi-span composite-deck highway overpass bridge
- Design example 2: Mideast US, two-span cable-stayed bridge
- Design example 3: Southeast US, multi-span prestressed concrete girder bridge

All three examples are independent on the development of the PSFs (except that the development was based on the same temperatures as used for design example 1 and 3) and therefore serve as a good test of the developed PSF methodology. For all design examples, only the analyses relating to a target service life of 75 years or 100 years are included. Moreover, only elements in either a seawater or de-icing salts zone or an atmospheric zone are included since only those exposure zones were considered in the development of the PSF method.

When performing a service life design of chloride-induced corrosion, most of the input parameters to Equations (4-8) to (4-16) have predetermined values or are based prevailing environmental conditions (e.g., temperature, T_{real}, and surface chloride content, $C_{s,\Delta x}$). In practice, the concrete cover, c, and chloride migration coefficient $D_{RCM,0}$, are the variables adjusted to obtain the desired level of reliability. In the following examples, generally the concrete cover thicknesses from the original design example are utilized, while the required chloride migration coefficient, $D_{RCM,0}$, is determined. Section 4.5.5 also shows an example of maintaining the same chloride migration coefficient as used in the original design, while adjusting the required cover depth. In actuality it might be necessary to modify both variables (c and $D_{RCM,0}$) in the same design to obtain the desired reliability; however, for this example the two variables are considered separately to give an impression of the main principles.

D4.5.1 Design Example 1 – New York

Table 4-14 shows the characteristics of the service life design for the elements considered from the New York design example. For further details on the input parameters, reference is made to Section 1 describing the design example.

Table 4-14: Characteristics of Analyses from Design Example 1 – New York. Only the Mean Value of Cover Depth, c, and Surface Chloride Content, $C_{s,\Delta x}$, is Given. Reference is Made to Section 1 Describing the Design Example for Further Details.

No.	Element	Exposure Zone	Cement	t_{SL} [yr]	c (mean) [mm] (in)	$C_{s,\Delta x}$ (mean) [wt-%/c]
1-1	Piers, bottom part of column	Direct de-icing salts	OPC+20-35%FA	75	76 (3.0)	4.0
1-2	Top of deck	Direct de-icing salts	OPC+20-35%FA	75	70 (2.75)	4.0
1-3	Piers, column and pier cap	Indirect de-icing salts	OPC+20-35%FA	75	76 (3.0)	2.0

No.	Element	Exposure Zone	Cement	t_{SL} [yr]	c (mean) [mm] (in)	$C_{s,\Delta x}$ (mean) [wt-%/c]
1-4	Underside of deck	Atmospheric	OPC+20-35%FA	75	44 (1.75)	1.5

The original service life modelling performed for design example 1 was based on a full probabilistic analysis. Table 4-15 summarizes the chloride migration coefficient, $D_{RCM,0}$, and the corresponding reliability index from the full probabilistic analysis (see Section 1).

Table 4-15: Summary of Chloride Migration Coefficients, $D_{RCM,0}$, and Corresponding Reliability Indices, β, from the Original Design, the Original Design Modified to Provide β=1.3, and Partial Safety Factor Design for Design Example 1 – New York. The β-ratio Compares the Reliability Indices from the Original Design (with β=1.3) and the PSF Design.

No.	Original Design		Original Design with β=1.3		PSF design		β-ratio (β_{PSF}/β_{FPD})
	$D_{RCM,0}$ [x 10^{-12} m/s^2]	$\beta_{FPD,0}$	$D_{RCM,0}$ [x 10^{-12} m/s^2]	β_{FPD}	$D_{RCM,0}$ [x 10^{-12} m/s^2]	β_{PSF}	
1-1	7.0	1.42	8.0	1.32	6.5	1.48	1.12
1-2	7.0	1.38	7.5	1.32	5.3	1.61	1.22
1-3	10.0	1.58	14.5	1.32	10.7	1.54	1.16
1-4	7.0	1.78	13.5	1.32	8.65	1.63	1.24

In a full probabilistic service life design, the mean value describing the chloride migration coefficient is always based on typically anticipated values relating to the w/c ratio of the concrete and if, for example, FA or ground granulated blast-furnace slag is added to the cement. It is well-known that the chloride migration coefficient of concrete using Portland Cement mixed with FA is less than that of concrete mixed with only Portland Cement and even less if the cement is mixed with ground granulated blast-furnace slag. Sometimes the full probabilistic design allows the use of a greater chloride migration coefficient than what is typically anticipated for a specific cement type. However, to make the design as realistic as possible, the chloride migration coefficient is often restricted to the typical value even though this causes the reliability index to be greater than 1.3 because it is conservative and also explains why some of the reliability indices from the original service life design of design example 1 in Table 4-15 are greater than 1.3. Even though this is on the safe side, it makes it impossible to compare the design with the PSF method that is calibrated with and quantified for a reliability index of β=1.3. For the purpose of verifying the PSF design, it is necessary to modify the chloride migration coefficients of the original full probabilistic design such that a reliability index of 1.3 is achieved even though the chloride migration hereby becomes greater than the typically assumed value. Table 4-15 also shows the values of the modified design.

Moreover, Table 4-15 shows the chloride migration coefficient, $D_{RCM,0}$, determined from the PSF design achieved by using the PSFs in Table 4-11. The PSF design is carried out exactly as described in Section 4.4.7 only are the input parameters now based on the details of design example 1. Table 4-15 also shows the corresponding reliability index that this chloride migration coefficient, $D_{RCM,0}$, yields in a full probabilistic design.

Finally, Table 4-15 shows the β-ratio, which compares the reliability index determined from the PSF method with that of the original (modified) design.

D4.5.2 Design Example 2 – Mideast US

Table 4-16 shows the characteristics of the service life design for the elements considered from the Mideast US design example. For further details on the input parameters, reference is made to Section 2 describing the design example.

Table 4-16: Characteristics of Analyses from Design Example 2 – Mideast US. Only the Mean Value of Cover Depth, c, and Surface Chloride Content, $C_{s,\Delta x}$, is Given. Reference is Made to Section 2 Describing the Design Example for Further Details.

No.	Element	Exposure Zone	Cement	t_{SL} [yr]	c (mean) [mm] (in)	$C_{s,\Delta x}$ (mean) [wt-%/c]
2-1	Pile caps, tower pedestals	Moderate de-icing salts	OPC+20-35%FA	100	50.8 (2.0)	1.0
2-2	Towers at deck level	Severe de-icing salts	OPC+20-35%FA	100	76.2 (3.0)	3.0
2-3	Towers below deck	Atmospheric zone with no de-icing salts	OPC+20-35%FA	100	50.8 (2.0)	1.0

The original service life modelling performed for design example 2 was based on a full probabilistic analysis. Table 4-17 summarizes the chloride migration coefficient, $D_{RCM,0}$, and the corresponding reliability index from Section 2. As described for Table 4-15, Table 4-17 also shows the chloride migration coefficient and reliability index for the modified original design yielding β=1.3. Table 4-17 also shows the chloride migration coefficient determined from the PSF design achieved by using Equations (4-8) to (4-16) and the PSFs in Table 4-11 as exemplified in Section 4.4.7. Table 4-11 also shows the corresponding reliability index that the chloride migration coefficients found in the PSF design would yield in a full probabilistic design. Finally, Table 4-17 shows the β-ratio which compares the reliability index from the PSF method with that of the original (modified) design.

Table 4-17: Summary of Chloride Migration Coefficients, $D_{RCM,0}$, and Corresponding Reliability Indices, β, from the Original Design, the Original Design Modified to Provide β=1.3, and Partial Safety Factor

Design for Design Example 2 – Mideast US. The β-ratio Compares the Reliability Indices from the Original Design (with β=1.3) and the Partial Safety Factor Design.

No.	Original design		Original design with β=1.3		PSF design		β-ratio (β_{PSF}/β_{FPD})
	$D_{RCM,0}$ [x 10^{-12} m/s^2]	$\beta_{FPD,0}$	$D_{RCM,0}$ [x 10^{-12} m/s^2]	β_{FPD}	$D_{RCM,0}$ [x 10^{-12} m/s^2]	β_{PSF}	
2-1	10.0	1.40	11.5	1.31	7.1	1.61	1.22
2-2	7.6	1.38	8.3	1.31	5.8	1.60	1.22
2-3	10.0	1.40	11.5	1.30	7.5	1.69	1.30

D4.5.3 Design Example 3 – Southeast US

Table 4-18 shows the characteristics of the service life design for the elements considered from the Southeast US design example. For further details on the input parameters, reference is made to Section 3 describing the design example.

Table 4-18: Characteristics of Analyses from Design Example 3 – Southeast US. Only the Mean Value of Cover Depth, c, and Surface Chloride Content, $C_{s,\Delta x}$, is Given. Reference is Made to Section 3 Describing the Design Example for Further Details.

No.	Element	Exposure Zone*	Cement	t_{SL} [yr]	c (mean) [mm] (in)	$C_{s,\Delta x}$ (mean) [wt-%/c]
3-1	Intermediate bents etc.	Splash	OPC+20-35%FA	100	114 (4.5)	3.5
3-2	Intermediate bents etc.	Splash	OPC+66-80%GGBS	100	114 (4.5)	3.5
3-3	End bents	Splash	OPC+20-35%FA	100	114 (4.5)	3.0
3-4	End bents	Splash	OPC+66-80%GGBS	100	114 (4.5)	3.0
3-5	Approach slab	Groundwater and soil	OPC+20-35%FA	100	102 (4.0)	3.0
3-6	Approach slab	Groundwater and soil	OPC+66-80%GGBS	100	102 (4.0)	3.0

* All exposure zones are considered as 'water level'.

The original service life modelling performed for design example 3 was based on a full probabilistic analysis. Table 4-19 summarizes the chloride migration coefficient, $D_{RCM,0}$, and the corresponding reliability index from Section 3 and shows the chloride migration coefficient determined from the PSF design achieved by using Equations (4-8) to (4-16) and the PSFs in Table 4-11 as exemplified in Section 4.4.7. The Table 4-19 also shows the corresponding reliability index that the chloride migration coefficients found in the PSF design would yield in a full probabilistic design. Finally, Table 4-19 shows the β-ratio which compares the reliability index from the PSF method with that of the original design.

Table 4-19: Summary of Chloride Migration Coefficients, $D_{RCM,0}$, and Corresponding Reliability Indices, β, from the Original Design and Partial Safety Factor Design for Design Example 3 – Southeast US. The β-ratio Compares the Reliability Indices from the Original Design and the Partial Safety Factor Design.

No.	Original design		PSF design		β-ratio (β_{PSF}/β_{FPD})
	$D_{RCM,0}$ [x 10^{-12} m/s^2]	β_{FPD}	$D_{RCM,0}$ [x 10^{-12} m/s^2]	β_{PSF}	
3-1	13.0	1.31	9.4	1.57	1.20
3-2	2.9	1.32	2.2	1.55	1.17
3-3	14.8	1.30	10.4	1.58	1.22
3-4	3.3	1.31	2.4	1.57	1.20
3-5	11.3	1.30	8.0	1.58	1.21
3-6	2.5	1.32	1.85	1.57	1.19

D4.5.4 Summary of Partial Safety Factor Design for Design Examples

Figure 4-4 shows the β-ratios in Table 4-15, Table 4-17, and Table 4-19 as a function of the cover depth for all design example analyses using cement of the type OPC+20-35% FA. Figure 4-4 includes both exposure zones with Δx "on" and "off". For all considered analyses, the PSF design is on the safe side compared to the full probabilistic design (β-ratios > 1).

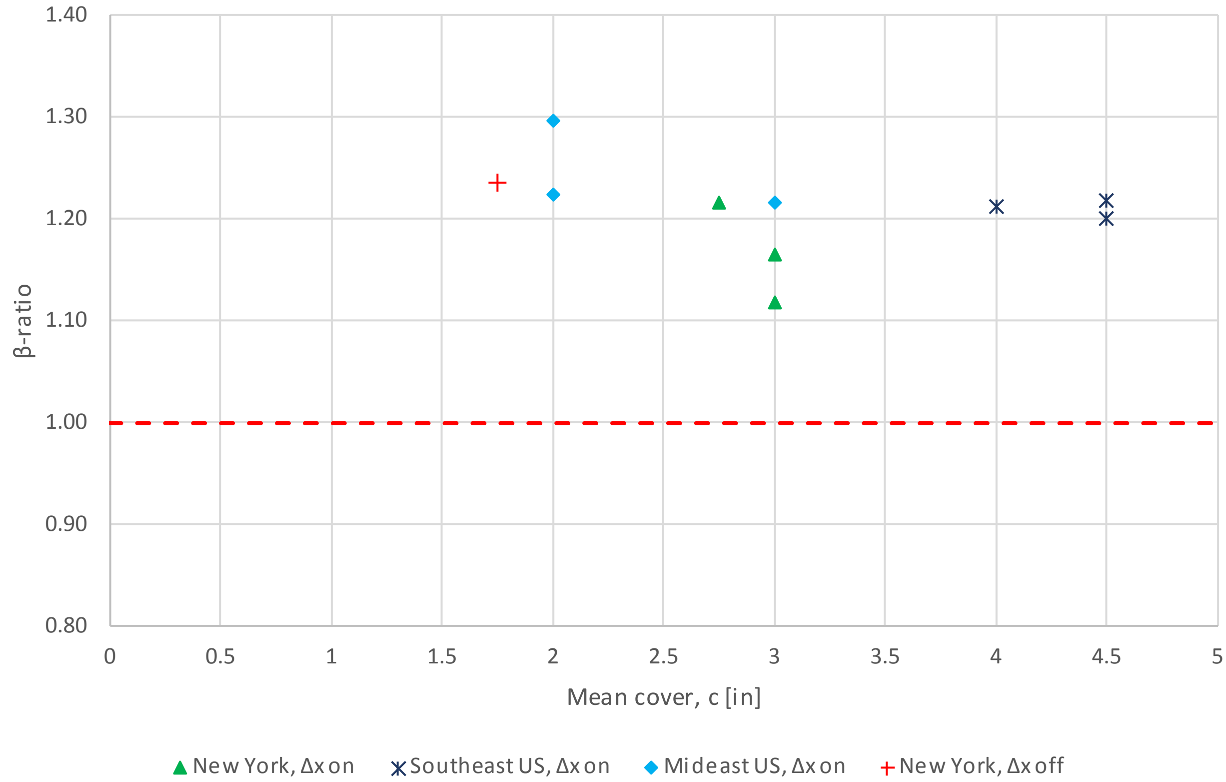


Figure 4-4: Summary of β-ratios for All Considered Design Examples with Cement Type OPC+20-35%FA.

Figure 4-5 shows the β-ratios in Table 4-19 as a function of the cover depth for all the design example analyses using cement of the type OPC+66-80% GGBS. Only the Southeast US design example considered this cement type for the Δx "on" exposure zone. Based on Figure 4-5, the PSF design is on the safe side compared to the full probabilistic design (β-ratios > 1).

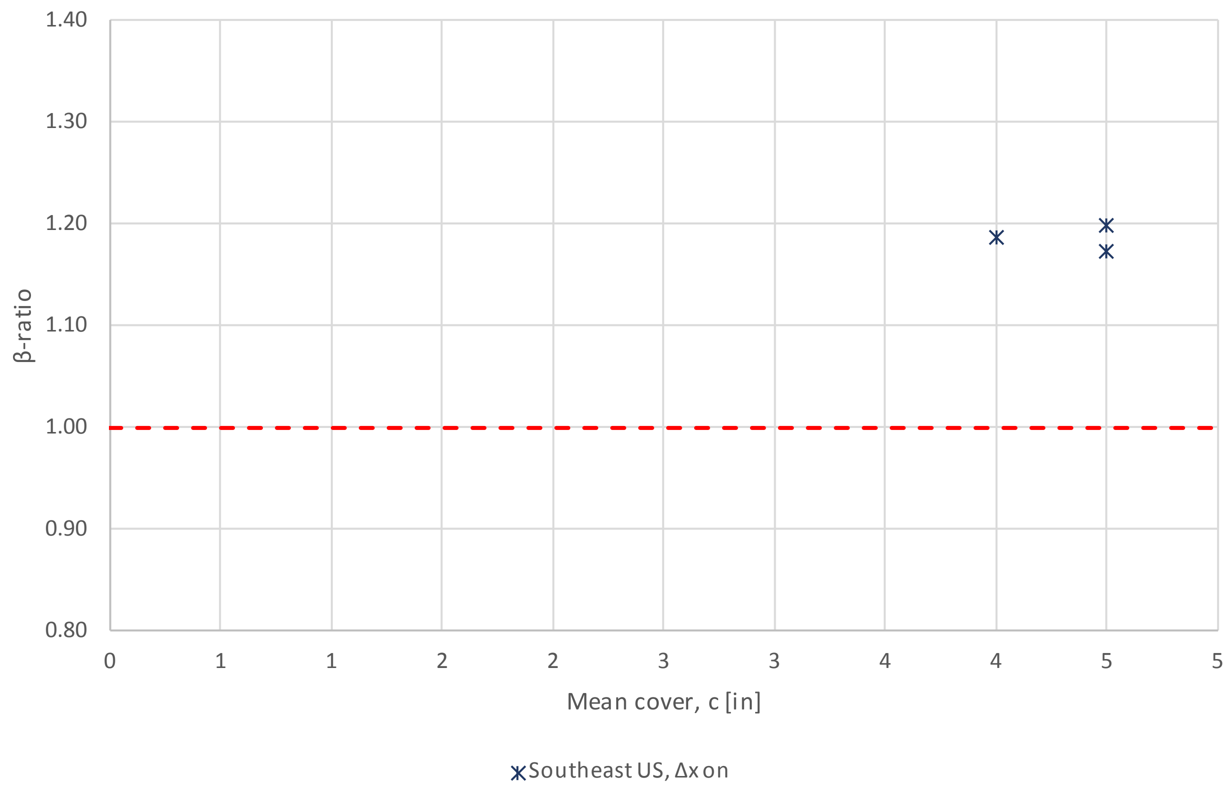


Figure 4-5: Summary of β-ratios for All Considered Design Examples with Cement Type OPC+66-80%GGBS.

Based on Figure 4-4 and Figure 4-5, it is concluded that the PSF design suggested in Section 4.4 using the PSFs in Table 4-11 provide a service life design that are conservative to the full probabilistic design for all three design examples; however, the conservatism covers an increase in the value of the reliability indices with a maximum of 30%.

D4.5.5 Partial Safety Factor Design by Varying Cover Depth

In this section, the PSF method is again applied to design example 3 (Southeast US). However, in this section the chloride migration coefficient, $D_{RCM,0}$ is held constant and the difference in required cover thickness, c, is assessed. The methodology is similar to what was applied and described in Section 4.5.3 except that now $D_{RCM,0}$ is set equal to the mean value from the original full probabilistic design, and the corresponding cover requirement is determined.

Table 4-20 summarizes the chloride migration coefficient from the full probabilistic and the associated cover depth, c, and the reliability index, β_{FPD} from Section 3. Table 4-20 also shows the cover depth determined from the PSF design achieved by using Equations (4-8)-(4-16) and the PSFs in Table 4-11. Moreover, the table shows the corresponding reliability index that the cover depth found in the PSF design would yield in a full probabilistic design. Finally, Table 4-20 shows the β-ratio which compares the reliability index from the PSF method with that of the original design.

Table 4-20: Summary of Cover Depths, c, and Corresponding Reliability Indices, β, from the Original Design and Partial Safety Factor Design for Design Example 3 – Southeast US. The β-ratio Compares the Reliability Indices from the Original Design and the Partial Safety Factor Design.

No.	$D_{RCM,0}$ [x 10^{-12} m/s^2]	Original design		PSF design		β-ratio (β_{PSF}/β_{FPD})
		c [mm] (in)	β_{FPD}	c* [mm] (in)	β_{PSF}	
3-1	13.0	114 (4.5)	1.31	132 (5.2)	1.57	1.20
3-2	2.9	114 (4.5)	1.32	129 (5.1)	1.55	1.17
3-3	14.8	114 (4.5)	1.30	134 (5.3)	1.58	1.22
3-4	3.3	114 (4.5)	1.31	131 (5.1)	1.57	1.19
3-5	11.3	102 (4.0)	1.30	119 (4.7)	1.58	1.21
3-6	2.5	102 (4.0)	1.32	115 (4.5)	1.57	1.17

* The cover depths shown correspond to a design where $C(c,t_{SL})_d = C_{crit,d}$. In reality, the cover depths would be rounded up to the closest 1/4'' causing the β-values to increase.

Table 4-20 directly shows which cover depths would be required if the service life design was based on the PSF method compared to a full probabilistic analysis. For all cases, a greater cover depth is required which is conservative and as expected based on the results from Section 4.5.3. Comparing the β-ratios in Table 4-19 to those in Table 4-20, an excellent agreement is observed. This exemplifies that the PSF design, as expected, provides a similar degree of reliability independent on which input parameter is varied.

D4.6 Final Remarks

This document presents a PSF design for chloride-induced corrosion modelled in agreement with *fib* Bulletin 34 based on the three design examples given in Sections 1-3 of this appendix. The PSFs are derived from multiple full probabilistic analyses by varying chloride loading and concrete material durability resistance parameters. The partial factor method includes simplifications of the full probabilistic approach on the safe side. The β-ratios in Table 4-15 through Table 4-19 vary from 1.12 to 1.24, which is a tight set of results. The PSFs may need to be conservative for the typical cases to limit potential non-conservatism in less typical cases. However, since no results lower than 1.12 have been demonstrated, the proposed PSFs might yield overly conservatism. This should be investigated in future further refined analyses supported by field performance. Thus, the PSFs given in this document are to be considered as preliminary and will most likely be changed when future studies containing a substantial set of data that include multiple scenarios have been developed.

The suggested PSF design in the present study represents just one way of performing a PSF design. Instead of applying PSFs to the variables chosen in this example (c, $C_{s,\Delta x}$, $D_{app,C}$, and C_{crit}), the PSF design could have considered other variables. ISO 2394 [2] provides recommendations on general principles on reliability for structures, including recommendations on the partial factors format.
According to ISO 2394, the governing variables in a design equation must be determined and design values obtained for those values. These design values are obtained in different ways depending on whether the variable represents an action, material property, or geometrical quantities. For actions and material properties, ISO 2394 recommends using a partial safety factor, whereas for geometrical quantities, ISO 2394 recommends that the design value is determined as the characteristic value minus additive geometric quantities taking into account the deviations from the characteristic value. This is similar to the *fib* Bulletin 34 recommendations. For example, the design value of the cover would be found as the nominal concrete cover minus the tolerance. This would mean the 5% fractile to be considered the design value of the cover which is much stricter than what has been assumed in the PSF method developed in the present study. On the other hand, the PSFs on other variables, such as the chloride migration coefficient would then be lessened as a consequence. The most optimal PSF method to be used for chloride-induced modelling requires much more work and further analysis; however, the present study has provided an example of one such method and shown the overall and general principles in the PSF design approach.

D4.7 References

[1] *fib* Bulletin 34. Model Code for Service Life Design. Lausanne, Switzerland: International Federation for Structural Concrete (*fib*); 1st edition. 2006. 126 pp.

[2] ISO 2394. General Principles on Reliability for Structures. 1998.