IBC Workshop: W-8 Service Life Design

Worked Design Example – Handouts

June 14, 2018

Contents

1.0 Introduction

Commented [AS1]: Seems like this needs a two-sentence introduction – what this is, how it can be used, etc. Companion document to academic toolbox?

This document supports the presentation given at IBC Workshop “W-8 Service Life Design” on June 14, 2018. The workshop covers a worked design example of the service life design of a highway bridge structure.

Figure 1 shows the general arrangement of the Bridge:

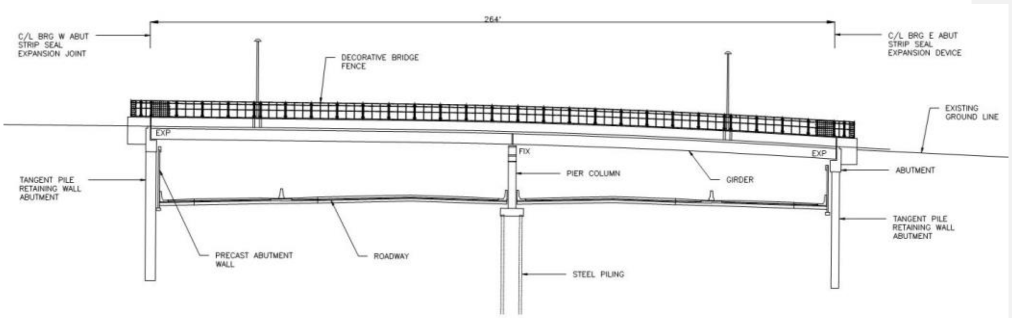

Figure 1: General arrangement of the Bridge.

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°C (52.7°F) [1].
- Heavy use of de-icing salts.
- Some sulfate present in soil: 0.14% by mass of water soluble sulfate was measured.

1.2 General Bridge Superstructure Characteristics

- 264 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 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.

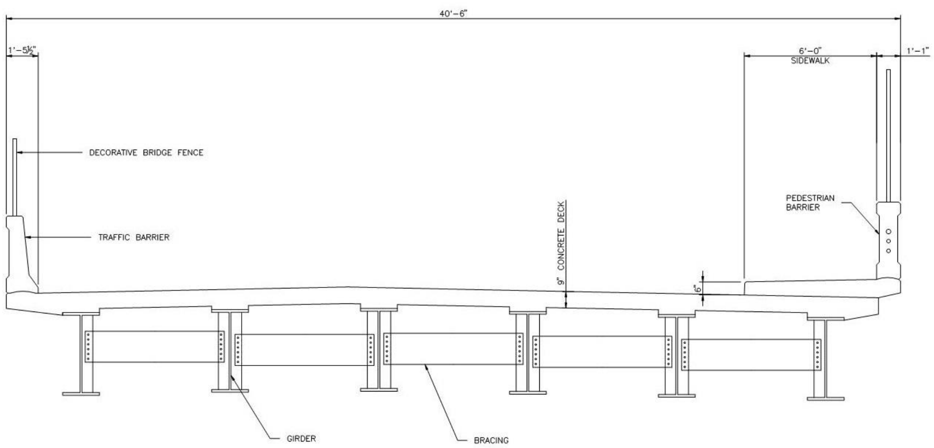

Figure 2: Typical section of superstructure of the Bridge.

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 3 shows a typical section of the pier.
- Abutments are supported by reinforced concrete tangent piles, see Figure 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.

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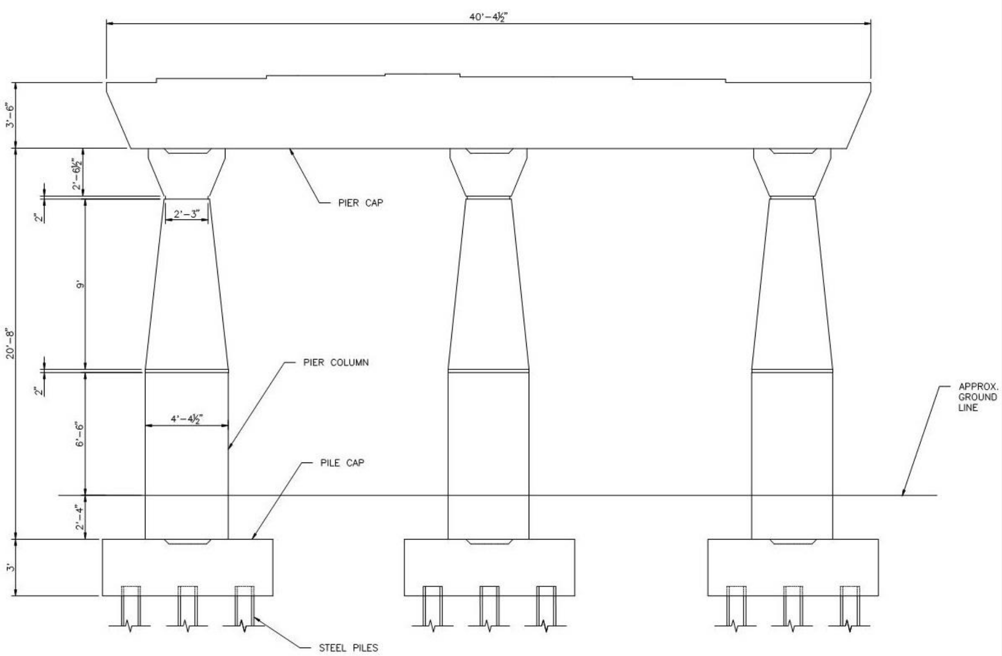

Figure 3: Pier elevation of the Bridge.

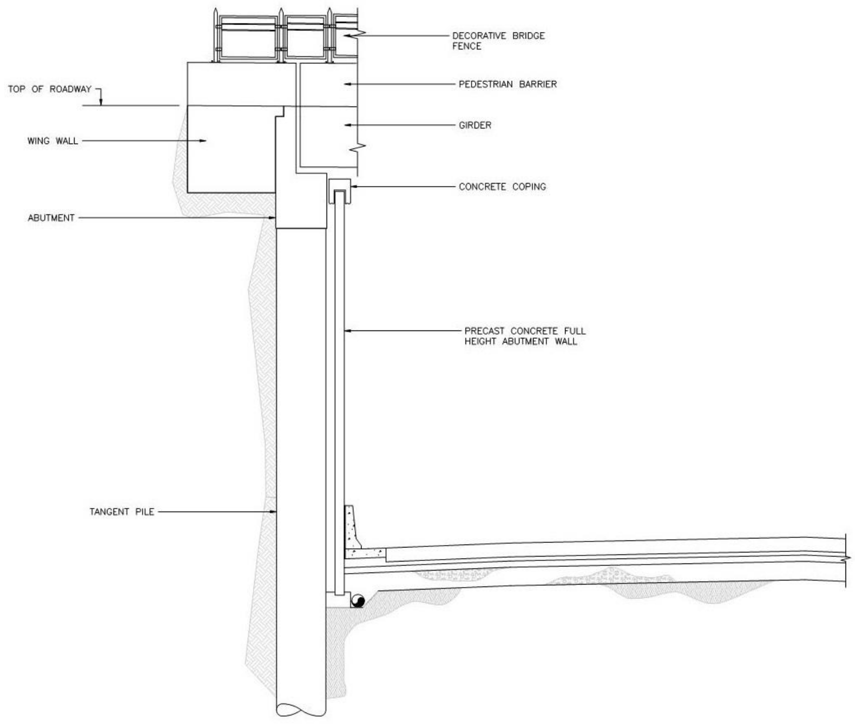

Figure 4: Typical section at abutment of the Bridge.

1.4 Service Life Requirements

- Non-replaceable components must meet a minimum service life of 75 years. Service life is defined as the period without major repairs or maintenance. Normal routine maintenance is expected.
- 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 *fib* Bulletin 34.
- The replaceable components must meet a minimum service life as shown in Table 1.

Table 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

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; and,
4. Define mitigation methods for deterioration mechanisms for steel components.

2.0 Exposure Zones

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.

2.2 Color Code to Identify Exposure Zones

- Typical exposure conditions are presented in Figure 5 to Figure 8.

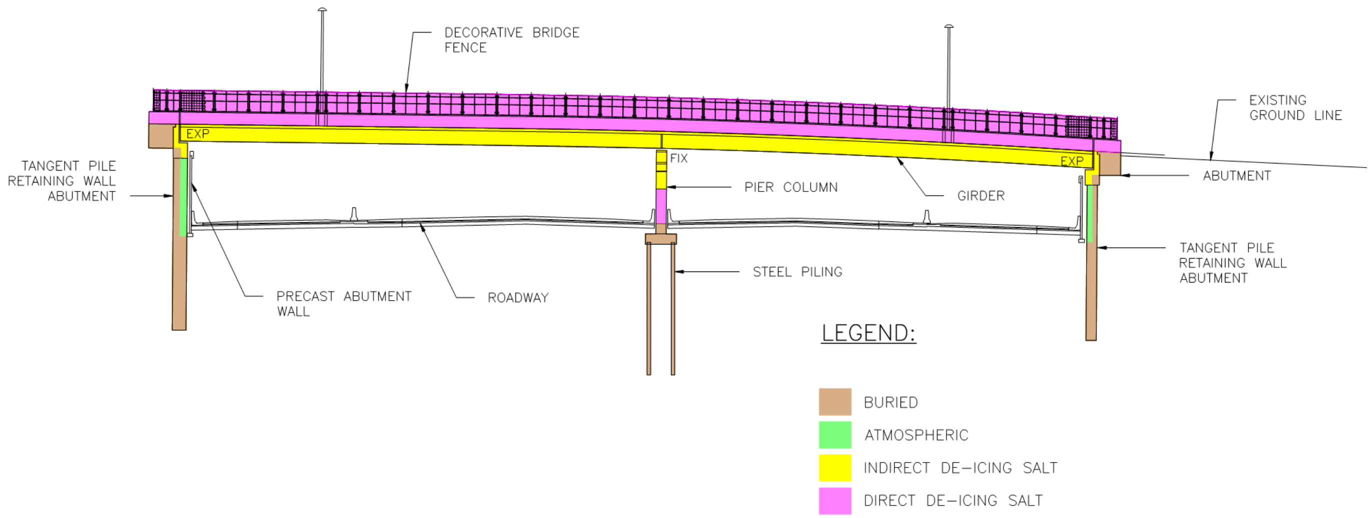

Figure 5: Exposure zones for the Bridge, longitudinal section.

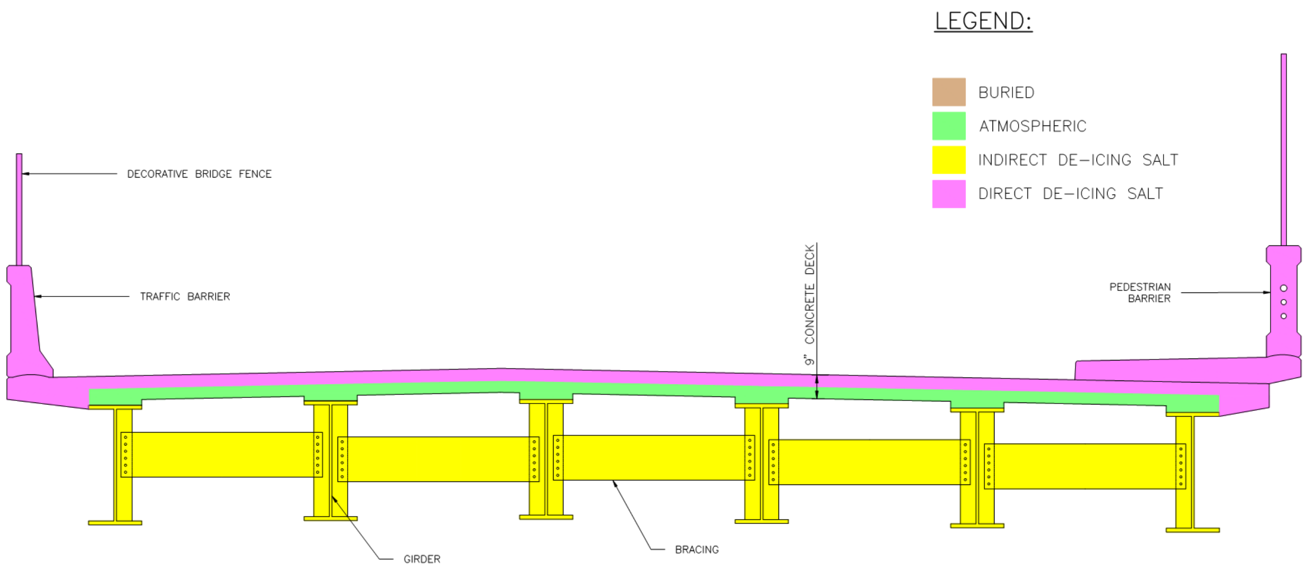

Figure 6: Exposure zones for superstructure.

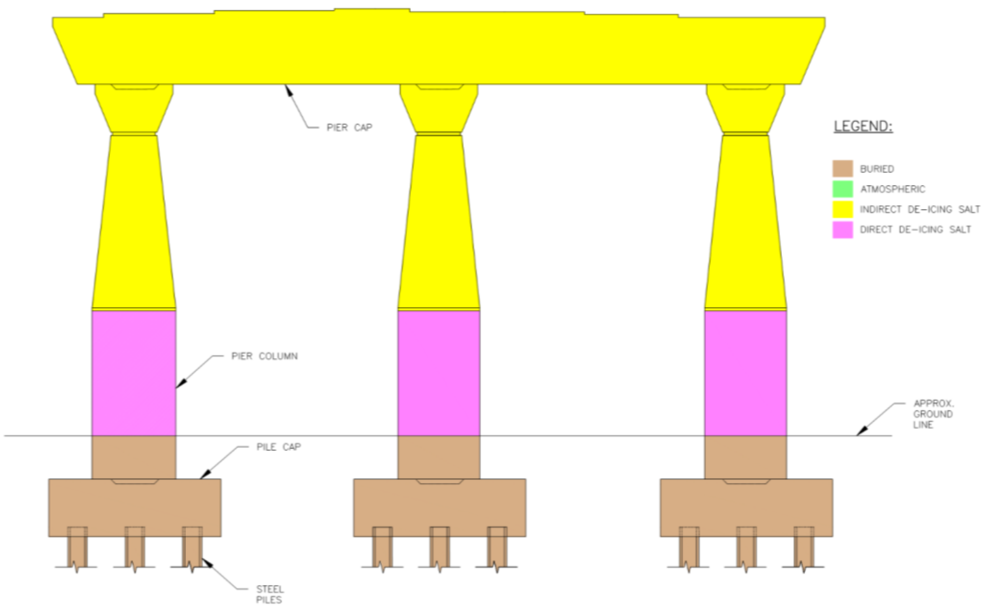

Figure 7: Exposure zones for substructure.

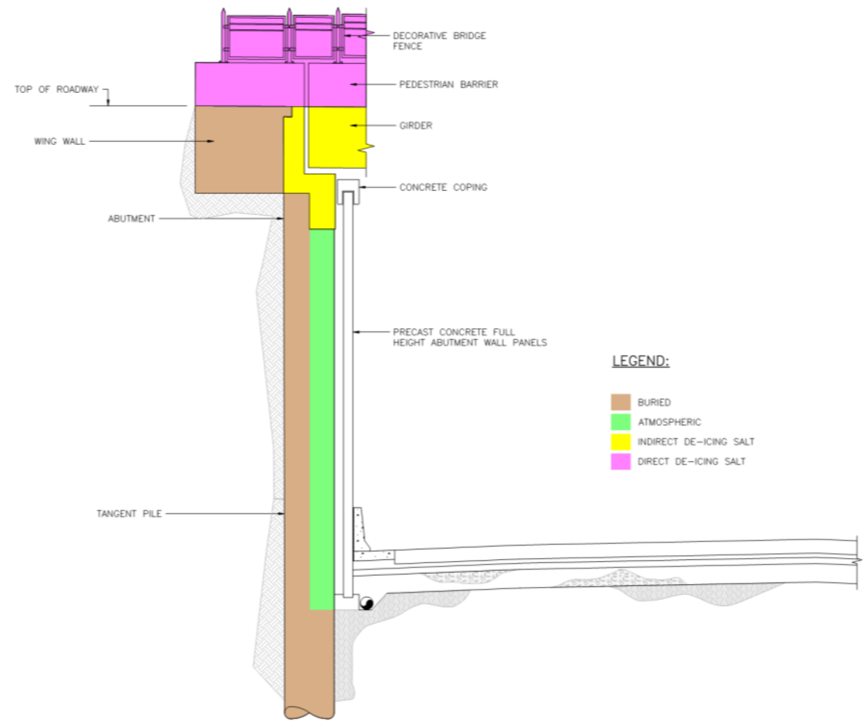

Figure 8: Exposure zones for abutments supported by tangent piles.

3.0 Deterioration Mechanisms

3.1 Considered Deterioration Mechanisms

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.

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- **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 flow (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.

3.1.2 Considered Deterioration Mechanisms for Steel

- The main deterioration mechanism for steel is corrosion.

3.2 Deterioration Mechanisms for Different Components

- Table 2 specifies which components are exposed to different types of deterioration along with the severity categories for each exposure condition. Exposure zones from ISO 12944 [2] for steel elements and 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.
- Delayed Ettringite Formation (DEF) must also be included if any of the concrete components are precast.

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

Exposure Zone	Examples of Elements	Exposure Conditions	Steel Corrosivity Category ISO 12944-2 [2]	Potential Concrete Deterioration Mechanisms						
				Exposure Zones ACI 318-14 [3]	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 O2 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							

4.0 Mitigation Methods for Concrete Components

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

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4.1 Full Probabilistic Modeling of Chloride-Induced Corrosion

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:

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

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

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

- Table 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 lbs/yd^3 (350 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 by mass of total cementitious materials.

- Table 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.

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

Deterioration Mechanism	Alkali-Aggregate Reaction (AAR)	Delayed Ettringite Formation (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 fly ash (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; 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% fly ash 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.
Requirements in U.S. Codes and Standards	Guidance from AASHTO R80-17 [5] can be used.	N.A.	Requirements according to ACI 318-14 for concrete classified as S1: - Maximum water-cement ratio of 0.50 and a minimum compressive strength of 4000 psi (28 MPa). - ASTM C150 Type II cement is allowed. Types I and III are also allowed if the C_3A-content is less than 8%.	Requirements according to ACI 318-14: - For concrete classified as F1: a maximum water-cement 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''. - For concrete classified as F2: a maximum water-cement 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''. - For concrete classified as F3: a maximum water-cement 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''.	N.A.	Requirements according to ACI 318-14 for concrete classified as C2: - Maximum water-cement 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).

Deterioration Mechanism	Alkali-Aggregate Reaction (AAR)	Delayed Ettringite Formation (DEF)	Sulfate Attack	Freeze-Thaw and Scaling	Carbonation-induced Corrosion	Chloride-induced Corrosion
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 psf (0.8 kg/m^2) can be used as a passing criterion.	N.A.	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 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	a	Concrete thickness measured from concrete surface to the surface of the outermost steel reinforcement.	*Fib* Bulletin 34 recommends that the distribution function for large cover depths be typically chosen as a normal distribution, whereas for small cover depths, distributions excluding negative values should be chosen, such as the lognormal function. For this example, covers from AASHTO LRFD are used as starting point. It is assumed that 90% of the cover is within the construction tolerance of ± 0.5 inches. 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.	*Fib* Bulletin 34 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.	*Fib* Bulletin 34 states that the initial chloride content in the concrete is not only caused by chloride ingress from the surface, but can also be due to chloride contaminated aggregates, cements, or water used for the concrete production. 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.	*Fib* Bulletin 34 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.	*Fib* Bulletin 34 recommends the standard deviation of the chloride migration coefficient to be 0.2 times the mean value. The mean value is assumed in the model such that the desired reliability index is obtained.	Normal	x 10^{-12} m^2/s	7.0	1.4

<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>Aging Factor</td><td>α</td><td>The age factor describes the time-dependent change of the migration coefficient as concrete matures.</td><td>Fib Bulletin 34 and fib Bulletin 76 [15] recommend the following aging factors for concrete with an equivalent water-cement ratio between 0.40-0.60:
<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>Beta</td><td>-</td><td>0.6</td><td>0.15
a=0; b=1</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>Fib Bulletin 34 recommends the following values for the transfer function:
- For water level, direct and indirect de-icing salts zones: beta distribution with a mean value of 8.9 mm, standard deviation of 5.6 mm with parameter a = 0.0 and b = 50.0.
- For buried, submerged, and atmospheric zones: deterministic value of 0.</td><td>Beta</td><td>mm
(in)</td><td>8.9
(0.35)</td><td>5.6
(0.22)
a=0; b=50</td></tr>
<tr><td>Critical Chloride Concentration</td><td>C_{cr}</td><td>Concentration required to break down the passive layer protecting the steel reinforcement.</td><td>Fib Bulletin 34 recommends using a beta distribution with a mean value of 0.6% by mass of cementitious materials (based on uncoated carbon steel reinforcement), a standard deviation of 0.15, a lower bound of 0.2, and an upper bound of 2.0.</td><td>Beta</td><td>Mass-% of total cementitious materials</td><td>0.6</td><td>0.15
a=0.2; b=2</td></tr>
<tr><td>Transfer Parameter</td><td>k_t</td><td>-</td><td>Fib Bulletin 34 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>Fib Bulletin 34 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>Fib Bulletin 34 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>Fib Bulletin 34 defines T_{ref} to be constant with a value of 293K (= 20°C).</td><td>Deterministic</td><td>°C
(°F)</td><td>20
(68)</td><td>-</td></tr>
</table>

Table 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 4 for all structural elements.

Structural Element	Descrip-tion	Exposure zone	Cover			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.	Mean	Std. dev.	Distr.	Mean	Std. dev.
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 col-umn	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	Atmos-pheric	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 confidently 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 6.

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

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

Calculating the average of the values presented in Table 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.

Commented [AS13]: I can't remove the following change without changing all the text – a tech editor will need to do it.

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.

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

- Input to spreadsheet based on values in Table 4:

Commented [AS14]: I assume this is something for a handout? Can't read otherwise – ditto the one below.

	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):

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

Trial	$D_{RCM,0}$ (mm²/yr) rand 0-1	$D_{RCM,0}$ (mm²/yr) RESULT	b_e (°K) rand 0-1	b_e (°K) RESULT	T_{real} (°K) rand 0-1	T_{real} (°K) RESULT	k_e	α rand 0-1	α RESULT	$A(t_{SL})$	$D_{app,C}$ (mm²/yr)	C_o (mass% of binder) rand 0-1	C_o (mass% of binder) RESULT	$C_{s,\Delta x}$ (mass% of binder) rand 0-1	$C_{s,\Delta x}$ (mass% of binder) RESULT	Δx (mm) rand 0-1	Δx (mm) RESULT	cover (mm) rand 0-1	cover (mm) RESULT	C_{crit} (mass% of binder) rand 0-1	C_{crit} (mass% of binder) RESULT	$C(x=cov,t_{SL})$ RESULT	Pass (1) /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.

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

- Input to spreadsheet based on values in Table 5:

INPUT PARAMETERS

Parameter	Description	Units	Distribution Function	Normal Distr Coefficients: Mean, μ	Normal Distr Coefficients: Std Dev, σ	Normal Distr Coefficients: Coeff of Variation, σ/μ	Log-Normal Distr Coeffs: $\ln\mu - \ln((\sigma/\mu)^2+1)/2$	Log-Normal Distr Coeffs: $\sqrt{\ln((\sigma/\mu)^2+1)}$	Beta Distr Coeffs: Lower Bound, a	Beta Distr Coeffs: Upper Bound, b	Beta Distr Coeffs: α	Beta Distr Coeffs: β
		in²/yr		0.064	0.013	0.20						
	Chloride Migration Coefficient (from Nordtest NT	mm²/yr		41.0	8.2							
$D_{RCM,0}$	Build 492 - results are given in m²/sec)	m²/sec	Normal	1.30E-12	2.60E-13							
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 - 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				
		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:

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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 x 10^{-12} m^2/s. It is, however, not possible to design an OPC concrete mix which 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 as repeated in Table 7.

Table 7: Normally Anticipated Values for the Chloride Migration Coefficient, $D_{RCM,0}$, for Different Types of Cement. From *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	N.A	8.9	10.0	15.8	19.7	25.0
OPC + FA (k = 0.5)	N.A	5.6	6.9	9.0	10.9	14.9
OPC + SF (k = 2.0)	4.4	4.8	N.A	N.A	5.3	N.A
OPC + 66-80% GGBS**	N.A	1.4	1.9	2.8	3.0	3.4

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

** GGBS = ground granulated blast-furnace slag.

4.2 Requirements for Concrete Mixes

- Table 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 for that component.
- All concrete mix designs will have a maximum allowed water-cementitious material ratio of 0.40 to achieve the service life.

Table 8: Summary of Requirements for Concrete Mixes.

Structural Element	Description	Cover		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} m²/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

Commented [MSL15]: Shouldn't the table only show one exposure zone for each component (the governing)??

Commented [AL16R15]: no leave as is.

5.0 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; or
  - Increased steel area (corrosion allowance).
- Table 9 shows the mitigation methods identified for the steel components in this example.

Table 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.

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 10 summarizes corrosion allowances using different references.

Table 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" for fill or disturbed natural soils*
FDOT Structures Design Guidelines – Section 3.1 [18]	0.075"	0.15"	0.225"
EN 1993-5, Eurocode 3: Design of Steel Structures, Part 5: Piling – Section 4.4 [19]	0.07" for undisturbed natural soils	0.18" for polluted natural soils and industrial sites	0.35" for non-compacted and aggressive fills

* A corrosion rate of 0.003" 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.

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 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 11 shows a typical painting sequence as recommended by the NACE paper 7422.

Table 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 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 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 **Error! Reference source not found.**, the time until maintenance replacement and full repaint is determined for the different coated steel components as shown in Table 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 13: Estimated Service Life to Maintenance Repaint, M, and Full Repaint.
C4 and C5-M refer to the steel corrosivitiy 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.
- 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.

6.0 Construction

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

Quality control and quality assurance 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, aggregates source reports, etc., 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 quality control: placement, finishing, and curing procedures for concrete structures; for coatings, these are surface preparation, application procedures, and monitoring procedures. The quality control and quality assurance of these operations should be described in the Project Specifications.

7.0 References

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