Durability Assessment of a Bridge Substructure

Example

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

This report is a work product of the SHRP2 Solution, Service Life Design for Bridges (R19A). The product leads are Matthew DeMarco, Federal Highway Administration, and Patricia Bush, American Association of State Highway and Transportation Officials. This report was authored by Anne-Marie Langlois, P.Eng., P.E., ing, Buckland & Taylor, with support from Mike Bartholomew, CH2M Service Life Design Team Leader.

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Definitions

^{o}C	degrees Celsius
^{o}F	degrees Fahrenheit
AAR	alkali-aggregate reactions
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ASTM	American Society for Testing and Materials
Cl^{-}	Chloride ions
Cs	Chloride surface concentration
CO_2	Carbon Dioxide
DEF	delayed ettringite formation
FA	fly ash
ft	feet
GGBS	ground granulated blast furnace slag
in^2/s	square inches per second
kg/m^3	kilograms per cubic meter
lb/yd^3	pounds per cubic yard
m^2/s	square meters per second
mg/L	milligrams per liter
mm	millimeters
MPa	megapascal
OPC	ordinary Portland cement
O_2	Oxygen
psi	pounds per square inch
SI	International System of Units

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1.0 Introduction

Service Life Design for Bridges was developed through the second Strategic Highway Research program (SHRP2). The *Service Life* product provides transportation agencies with a framework, guidelines, and solutions that will allow them to systematically analyze the service life performance of their bridges, in whole and in part; evaluate solutions; and choose the best solution based on specific criteria. The design method product focuses on a systematic approach that will assist agencies in finding new or better ways to design new and retrofit projects that will last longer and need less maintenance.

Service Life provides a body of knowledge relating to bridge durability under different exposure conditions and constraints, establishes an array of options capable of enhancing service life, and applies life-cycle cost analysis to aid in selecting the overall most cost-effective design. A solution for a particular service life issue highly depends on many factors, which vary from location to location and state to state, because a solution depends on local practices, preferences, environmental conditions, and anticipated demands. Consequently, use of service life design practices is not intended to dictate a unique solution for any specific service life problem or to identify the "best and only" solution. Rather, it equips the reader with a body of knowledge for developing specific solutions best suited to the stated conditions and constraints.

The following report presents an overview to the service life design process. It identifies the key environmental exposure and deterioration mechanisms for typical bridge projects, and provides a summary of the parameters used in developing a full probabilistic service life design for durability.

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*. 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.

2.0 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 major maintenance is required; normal regular maintenance is expected during the service life. Normal maintenance is defined as either "good practice" directed toward prolonging the life of components that are performing as expected (for example, cleaning debris from horizontal surfaces) or local repairs resulting from unforeseen conditions. Normal maintenance is typically planned and described in a project-specific Operations and Maintenance Manual. Major maintenance is defined as maintenance required beyond normal maintenance, which is typically unplanned and a result of widespread systemic deterioration.

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 0 of this report. The following describes the two-phase service life model:

- The Initiation Phase

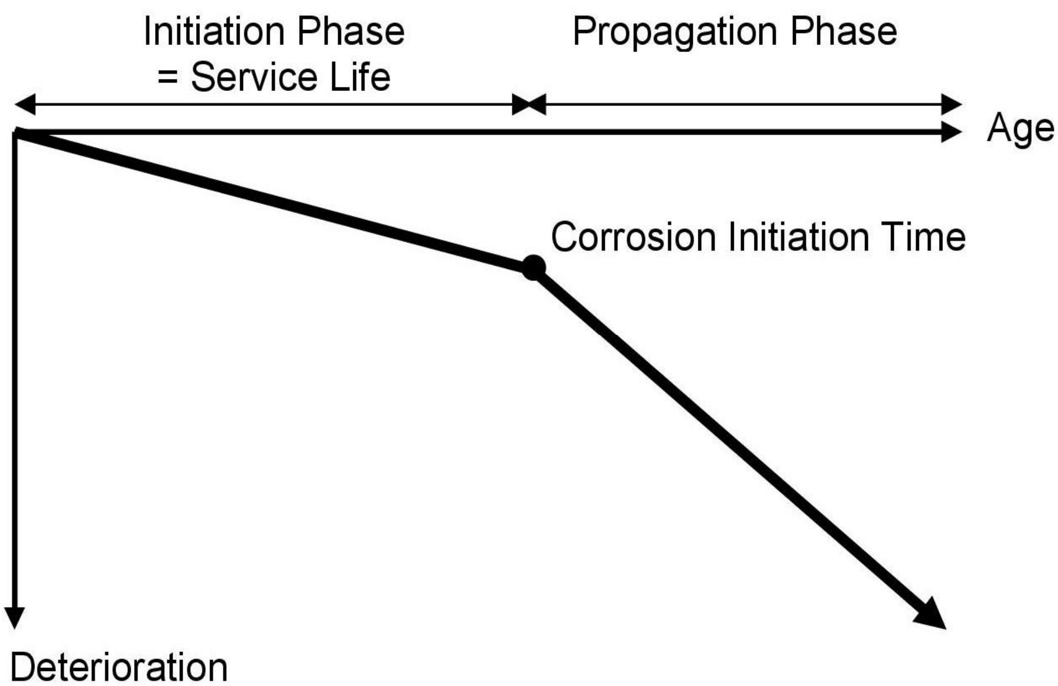
  During this phase no noticeable weakening of the material or the function of the structures occurs. Aggressive substances from the surrounding environment penetrate into the concrete and either accumulate with time in the outer concrete layer (such as sulfates) or alternatively, diffuse further inward, 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 corrosion 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 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 1: Two-Phase Modeling 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 a performance criteria for concrete quality (that is, chloride migration coefficient).
- Based upon the performance criteria, perform compliance tests for quality control 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 percent. 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, alkali-aggregate reactions (AAR), delayed ettringite formation (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.

3.0 Exposure Conditions

The Bridge will be subject to multiple concrete deterioration processes. The severity of the various processes are 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 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 presents the exposure zones for the drilled shafts, pile caps and towers. Figure 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 inch (25 millimeters [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 inches (152 mm) within the steel casing and 3 inches (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 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 on occasion 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 feet 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 starts where the towers become hollow, approximately 15 feet below the deck surface. This exposure zone is defined to extend up to 35 feet 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 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: Typical Exposure Zones for the Towers

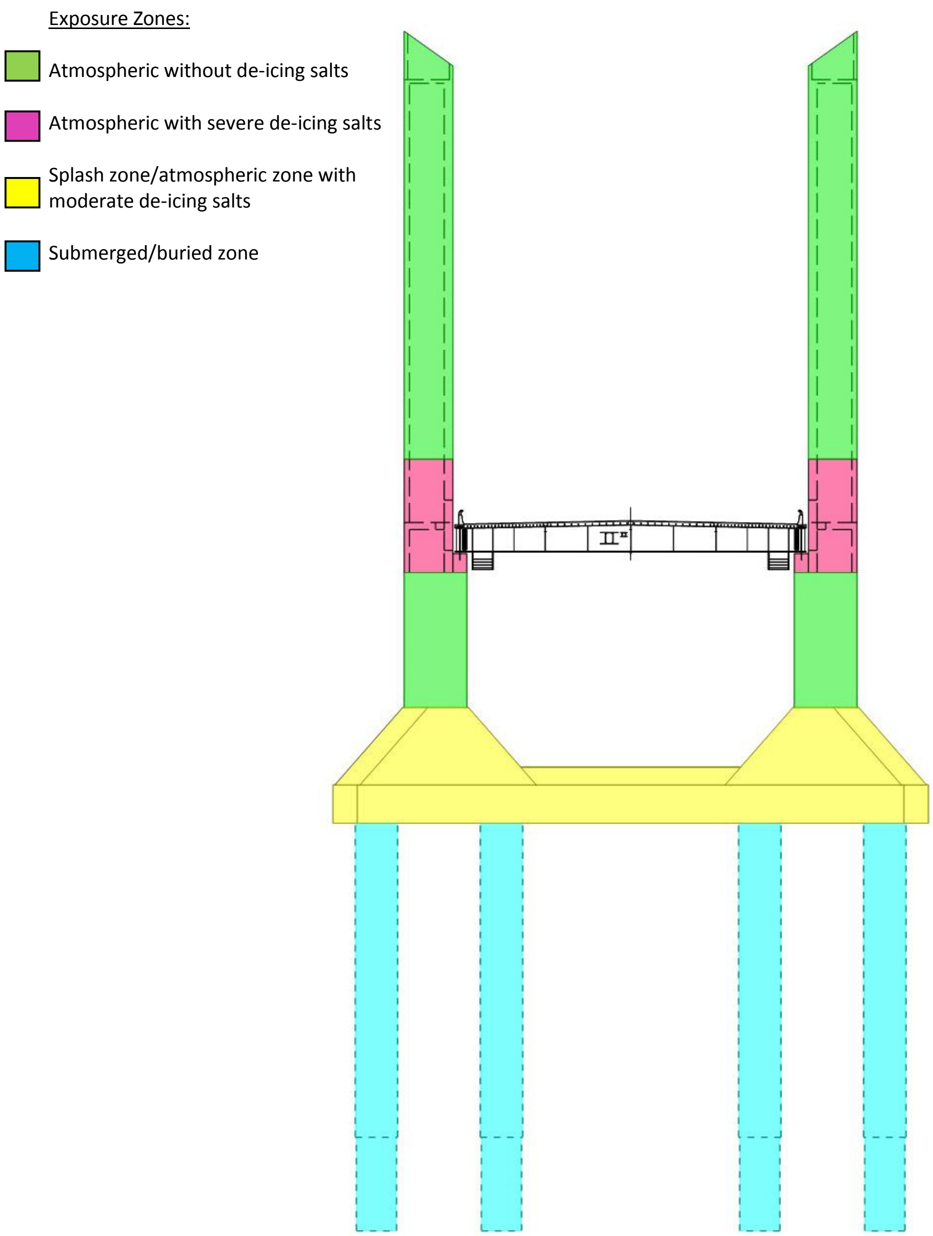
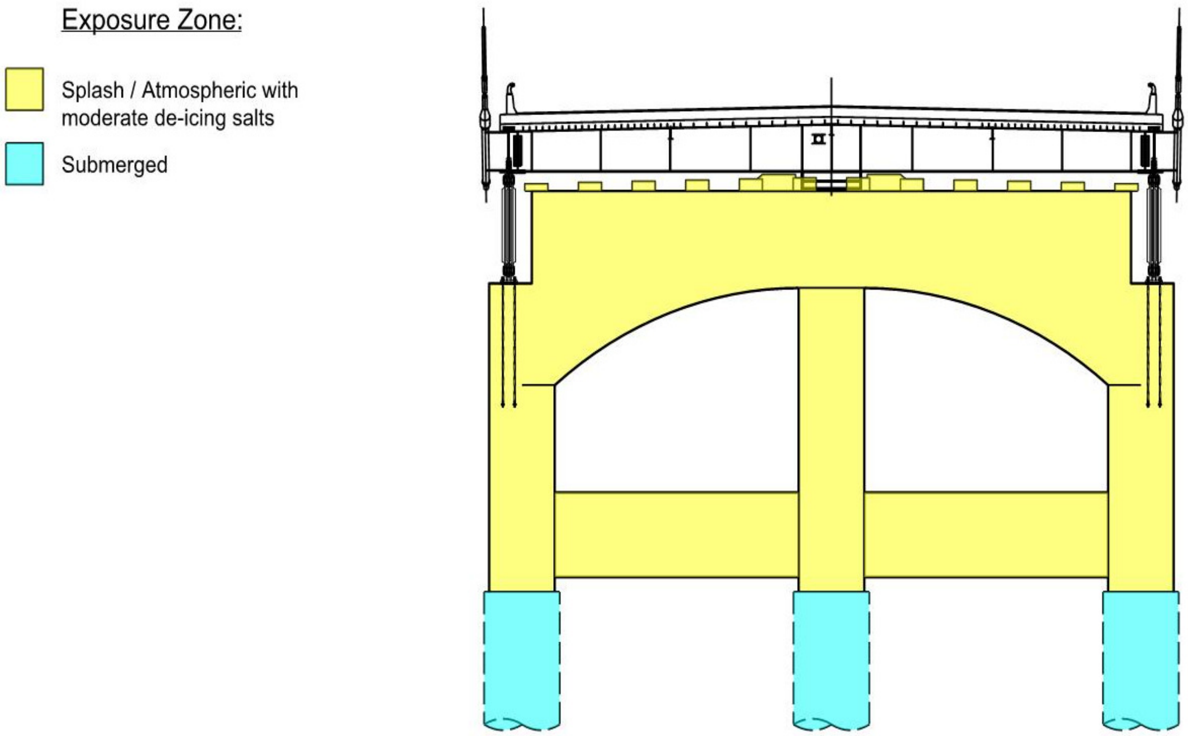


Figure 3: Typical Exposure Zones for the Anchor Piers



4.0 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 0. 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 1. Because of the lack of a comprehensive document for durability requirements, durability requirements are reviewed based on American Concrete Institute

(ACI) 318, and American Association of State Highway and Transportation Officials (AASHTO) PP65 when applicable. The owner's Standard Specifications are reviewed to ensure they adequately address each deterioration mechanism. Additional requirements are specified to supplement these specifications in some instances. A summary of the different documents consulted for each deterioration mechanisms is presented as follows:

- AAR: AASHTO PP65
- Sulfate attack: ACI 318
- DEF: project specifications
- Freeze-thaw: ACI 318

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 percent internal relative humidity or the reaction will cease [23].

Aggregates will come from material sources approved by the local authorities. Potential reactive aggregates will be addressed through the provisions of AASHTO PP 65 [24]. Risk of AAR is minimized by selecting a nonreactive aggregate as defined in AASHTO PP 65 using American Society for Testing and Materials (ASTM) C 1260 or selection of adequate preventative measures in accordance with AASHTO PP 65.

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 water/cement ratio should be below 0.50 and a minimum concrete compressive resistance of 4,000 pounds per square inch (psi) (28 megapascal [MPa]) should be provided (ACI 318). Both of these parameters will be met.

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 fly ash and/or ground granulated blast furnace slag cement (GGBS).

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 degrees Fahrenheit (^{o}F) (71 degrees Celsius [^{o}C]). This requirement should be incorporated into the project's Special Provisions.

For the Bridge, mass concrete is considered any concrete placement, excluding drilled shafts, with a plan dimension at least 7 feet 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).

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-water-to-cement 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 water-cement 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 percent is recommended for aggregate sizes of 0.75 inch to 1.0 inch (19 to 25 mm) [3]. This is consistent with the local Standard Specifications where a fresh concrete air content of 6 percent ± 2 percent is specified for all concrete classes and 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 water-cement ratio of 0.40 is recommended as explained in Section 5.

Only Class F fly ash will be used. The local Standard Specification limits the Class F fly ash content to 20 percent by mass of total cementitious materials. AASHTO LRFD [4] and ACI 318 [3] would allow this limit to be raised to 25 percent by mass of total cementitious material. It is recommended not to go over 35 percent, as higher amounts of fly ash 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 percent by mass of total cementitious materials. ACI 318 allows this limit to be raised to 50 percent 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 percent 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.

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 in order to mitigate other deterioration mechanisms, such as freeze-thaw cycles and chloride-induced reinforcement corrosion.

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 reduces the 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. Therefore, 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.

5.0 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 to be 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 0.

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

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.

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/DARTS 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 percent to 25 percent Class F fly ash 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 percent to 45 percent GGBS by mass of total cementitious materials has been included in the discussion for the splash zone only (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 percent 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 percent 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 percent is subject to acceptance by the local authorities.

The concrete mix for the drilled shafts contains 45 percent Class F fly ash by mass of total cementitious materials. The local Standard Specifications state that a maximum of 20 percent Class F fly ash or 30 percent Class C fly ash by mass of cementitious materials may be used in the concrete mix. As a reference, ACI 318 limits the content of fly ash to 25 percent 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 fly ash 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, fly ash, and silica fume is not preferred because of potential problems related to handling, compaction and early age cracking related to silica fume. However, if difficulties are encountered in achieving the required chloride migration coefficient, then the addition of silica fume in a limited quantity (estimated at 5 to 7 percent by mass of cementitious materials) could be investigated.

5.1.1 Concrete Properties

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} square meters per second [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 percent to 25 percent Class F fly ash 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.

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. 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: 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 + fly ash 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

5.1.1.3 Initial Chloride Content

The maximum total initial chloride concentration is assumed to be deterministic and equal to 0.1 percent 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 percent by mass of cement for acid soluble chlorides (ASTM C1152) or 0.08 percent by mass of cement for water soluble chlorides (ASTM C1218M). The limit on acid soluble chloride is used here.

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 percent to 8.34 percent by mass of cementitious materials have been reported [9]. The *fib* Model Code suggests a mean value of 0.6 percent by mass of cementitious materials and is based on uncoated steel reinforcement. The variability of this parameter is considered by using a beta distribution with a standard deviation of 0.15, a lower bound of 0.2, and an upper bound of 2.0 [1] as suggested by *fib.*

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 inch for cover. Therefore, the analysis with a standard deviation of 0.24 inch is conservative since it assumes that approximately

16 percent of the bars (the proportion outside of one standard deviation) do not actually meet the specified construction tolerance of 1/4 inch.

Table 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 inch) 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 3: Summary of Nominal Concrete Covers

Structural Elements	Specified Construction Tolerance (inch)	Mean Concrete Cover Used in Durability Analysis (inch)	Standard Deviation Used in Analysis (inch)
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 inches above deck level - hollow section)	±0.25	2.0	0.24
Piers (columns and cap)	±0.25	2.0, 3.0	0.24

5.1.2 Reinforcing Steel

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

5.1.3 Exposure Conditions

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.

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 inch (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].

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.

5.1.3.4 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/year. Therefore, 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 up 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 percent 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. As a result, 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 percent by mass of binder. This value is modeled as a lognormal distribution with a coefficient of variation of 0.5. It is expected that this value is more than what the pile caps and pier caps will actually experience.

5.1.3.5 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 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 kilograms per cubic meter (kg/m^3) of concrete. The data in Table 4 was transformed into percentage by mass of cementitious materials assuming a content of 620 pounds per cubic yard (lb/yd^3) (368 kg/m^3), which is the minimum cement content required by the local Standard Specifications for superstructure concrete. It should be noted that because of this transformation, the actual chloride concentrations for the reported structures may be less than that shown in Table 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 and different locations on the structures, and the structures were of different concretes and different ages.

A chloride surface concentration of 1.5 percent 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 percent for Type I cement (Portland Cement) and 2.8 percent 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 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	
	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 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 percent 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 percent with a coefficient of variation of 0.5.

5.2 Summary of Input Parameters

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

Table 5 presents the input parameters for the splash zone.

Table 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_{cr}	Beta	Mass % of binder	0.6	0.15 a(1)=0.2; b(1)=2
Initial Chloride Concentration	Co	Deterministic	Mass % of binder	0.1	-
Aging Factor	a	Beta	-	Table 2	Table 2
Temperature	T	Normal	^oF (^oC)	58.3 (14.6)	15.7 (8.7)
Cover	-	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 for upper and lower bound.

1.1.1 5.2.2 Atmospheric Zone with Severe De-icing Salts

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

Table 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_{cr}	Beta	Mass % of binder	0.6	0.15 a=0.2; b=2
Initial Chloride Concentration	Co	Deterministic	Mass % of binder	0.1	-
Aging Factor	a	Beta	-	Table 2	Table 2
Temperature	T	Normal	^oF (^oC)	58.3 (14.6)	15.7 (8.7)
Cover	-	Normal	Inch (mm)	3.00 (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 for upper and lower bound.

5.2.3 Atmospheric Zone without De-icing Salts

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

Table 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 5.3.3	0.50μ
Critical Chloride Concentration	C_{cr}	Beta	Mass % of binder	0.6	0.15 $a^{(1)}$=0.2; $b^{(1)}$=2
Initial Chloride Concentration	Co	Deterministic	Mass % of binder	0.1	-
Ageing Factor	a	Beta	-	Table 2	Table 2
Temperature	T	Normal	^oF (^oC)	58.3 (14.6)	15.7 (8.7)
Cover	-	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 for upper and lower bound.

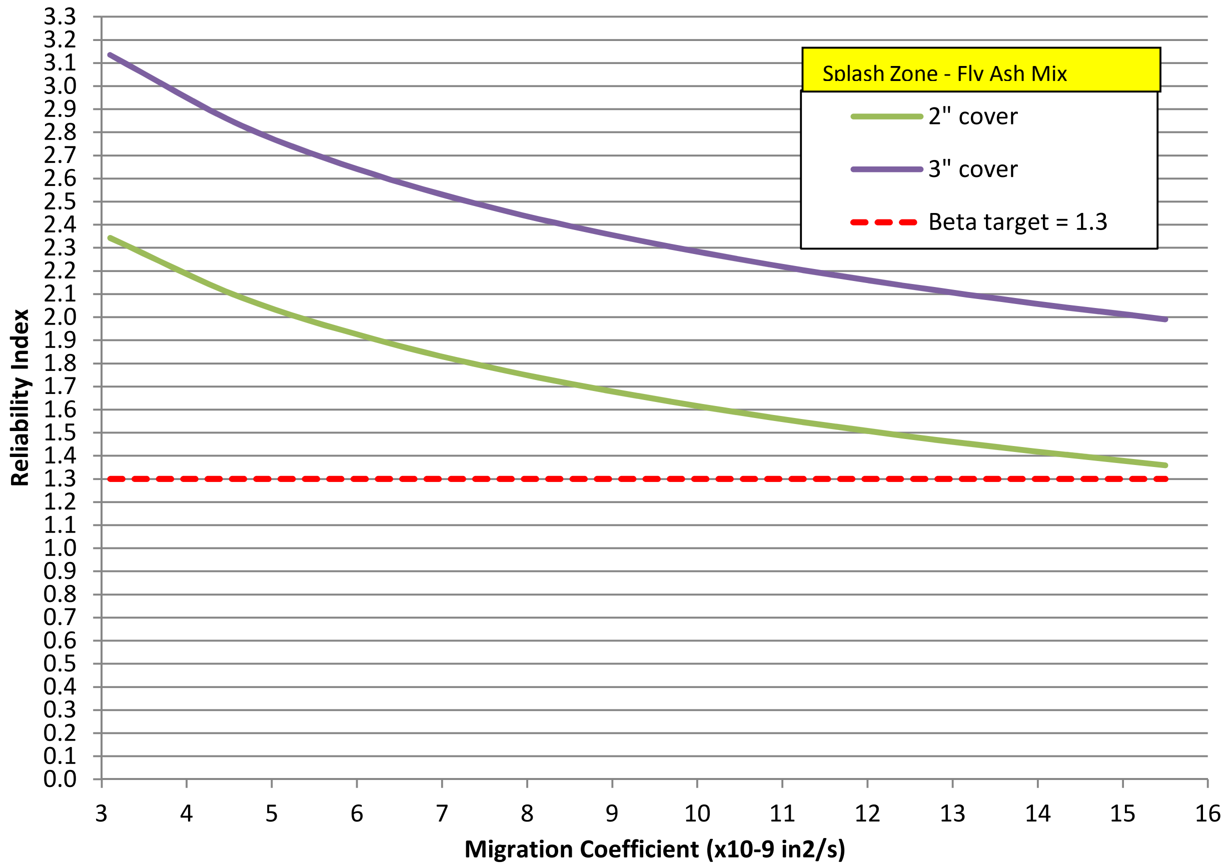
5.3 Results

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]).

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

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

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

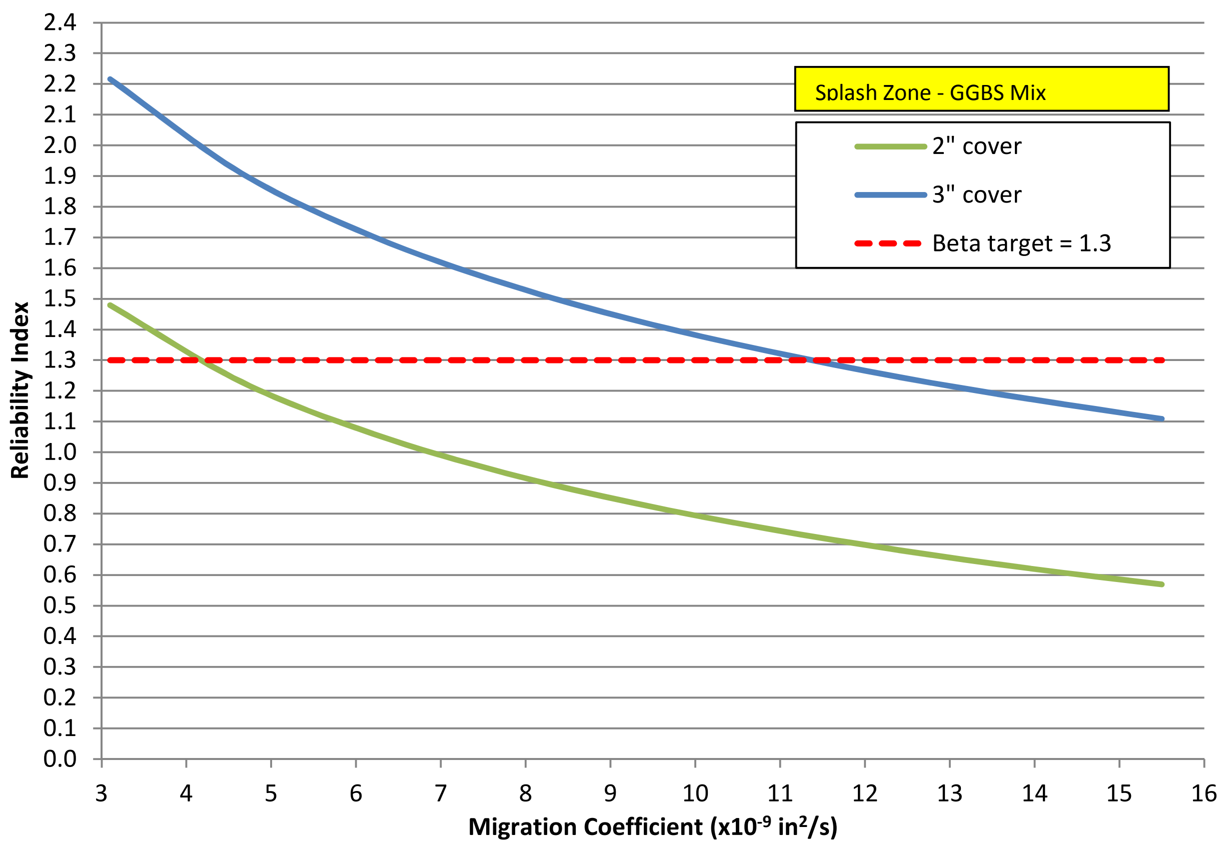


The target reliability index is 1.3. A reliability index greater than 1.3 means that corrosion has less than 10 percent probability of initiation within 100 years. As noted previously, concrete with a chloride migration coefficient greater than 15.5 x 10^{-9} in^2/s (10.0 x 10^{-12} m^2/s) is not recommended.

Based on the results shown in Figure 4, it can be seen that the 2-inch 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) is sufficient to achieve a 100-year service life for a surface chloride concentration of 1.0 percent. This is based on a concrete mix design with a minimum of 20 percent fly ash by mass of total cementitious materials.

Figure 5 illustrates the different requirements that apply if GGBS is used. Based on the results shown in Figure 5, it can be seen that a 3-inch 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) is required to achieve a 100-year service life for a surface chloride concentration of 1.0 percent in the splash zone. This is based on a concrete mix design with Portland Cement and 30 to 45 percent GGBS by mass of total cementitious materials. A 2-inch 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.

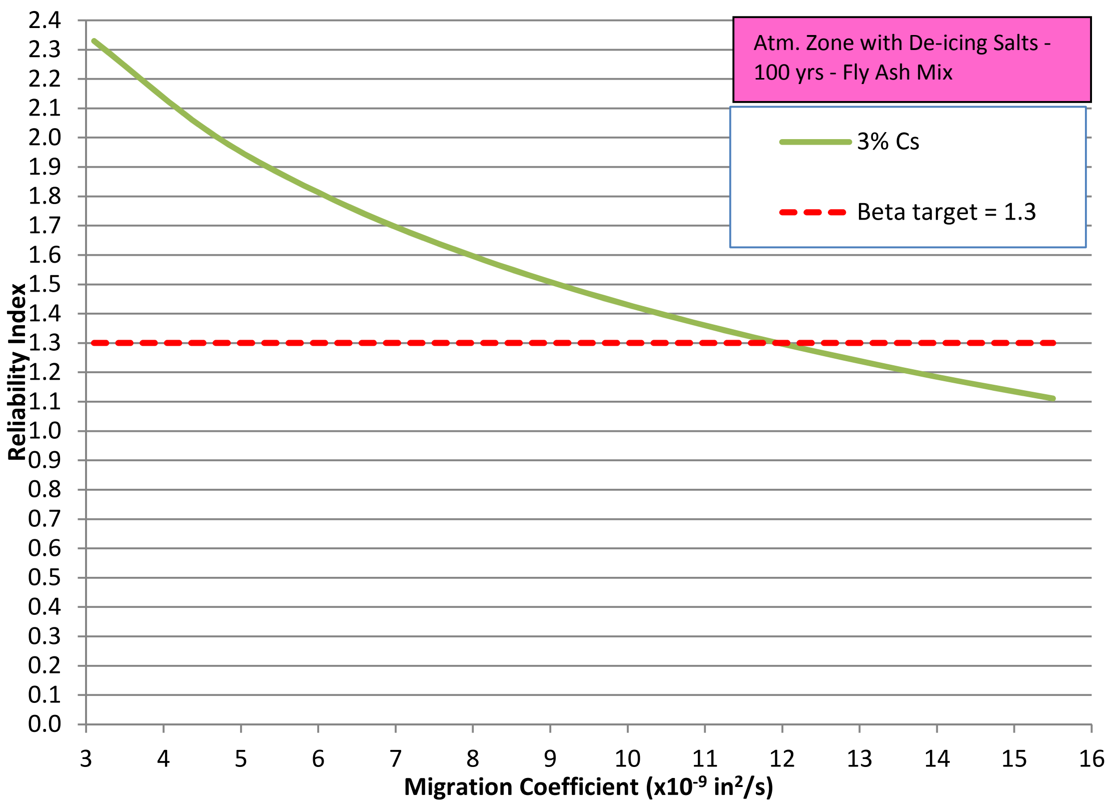
Figure 5: Reliability Index for 2- and 3-inch Cover Using Different Migration Coefficients, Assuming 1.0 percent Surface Chloride Concentration, 100-year Service Life, and Portland Cement with 30 to 45 percent GGBS



5.3.2 Atmospheric Zone with Severe De-icing Salts

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

Figure 6: Reliability Index for 3-inch Cover Using Different Migration Coefficients, Assuming 3.0 percent Surface Chloride Concentration, 100-year Service Life, and Portland Cement with Minimum 20 percent Fly Ash



The target reliability index is 1.3. A reliability index greater than 1.3 means that corrosion has less than 10 percent 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 percent 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 fly ash content of 20 percent by mass of cementitious materials. Factors such as the water-cement 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 fly ash could be increased should the target migration coefficient not be readily achievable within the local limits. It is recommended that the fly ash content should be limited to a maximum of 25 percent by mass of cementitious materials (ACI 318). The local Standard Specifications limit the content of Class F fly ash to a maximum of 20 percent of total cementitious materials. A relaxation of the local Standard Specifications will be required should Class F fly ash be used to an amount of 25 percent by mass of cementitious materials.

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-inch 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 0, except the drilled shaft mix where the exposure to de-icing salts is not applicable.

Analyses show that a concrete mix with 2-inch 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 percent. This level of chloride exposure is believed to be greater than the towers will experience during their service life.

5.4 Summary of Probabilistic Assessment

Table 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 8.

Table 8: Summary of Required Concrete Cover and Chloride Migration Coefficient Based on a Probabilistic Assessment

Exposure Zone	Structural Element	Required Cover for Service Life (inch)	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% Fly Ash Class F	15.5×10^{-9} in²/s (10.0×10^{-12} m²/s)
		3	Portland Cement + 30-45% GGBS	11.3×10^{-9} in²/s (7.3×10^{-12} m²/s)
Atmospheric with severe de-icing salts	Towers (exterior) at deck level	3	Portland Cement + 20-25% Fly Ash Class F	11.8×10^{-9} in²/s (7.6×10^{-12} m²/s)
Atmospheric without de-icing salts	Towers (exterior) below deck level	2	Portland Cement with 20-25% Fly Ash Class F	15.5×10^{-9} in²/s (10.0×10^{-12} m²/s)
	Towers (exterior) above deck level	2		

6.0 Concrete Durability Testing Requirements

6.1 Cementitious Materials

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

- Portland Cement will contain a low alkali content (<0.6% equivalent Na_2O) as defined in ASTM C150.
- Fly ash 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.

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 percent at 6 months.

Potential reactive aggregates will be addressed through the provisions of AASHTO PP 65 [24].

6.3 Concrete

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

- Portland Cement with 20 percent to 25 percent Class F fly ash by mass of total cementitious materials for all structural elements and exposure conditions except drilled shafts.
- As an alternative solution, Portland Cement with 30 percent to 45 percent 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 percent Class F fly ash 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 fly ash content of 20 percent by mass of cementitious materials. Factors such as the water-cement 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 percent Class F fly ash or more than 30 percent 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 9.

Concrete cover larger than specified in Table 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 percent 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 percent 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 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 feet 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 9: Summary of Exposure Zone, Concrete Mix Requirements, and Test Requirements

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

(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 feet 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.

7.0 Conclusion

This document presents the durability assessment of the concrete substructure (drilled shafts, pile caps, towers, and piers) for the Bridge. The non-replaceable 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 0, 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 0, 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 0. 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 percent to 25 percent Class F fly ash by mass of total cementitious materials for all structural elements and exposure conditions except drilled shafts.
- As an alternative solution, Portland Cement with 30 percent to 45 percent GGBS by mass of total cementitious materials may be used for splash zone only (pile caps, piers, tower pedestals).
- Portland Cement with 45 percent Class F fly ash for the drilled shafts.

Table 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 0.

8.0 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.

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