A Comparison of Service Life Models for Concrete Corrosion Durability

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16. Abstract

This study compares the service life models of concrete structures against chloride-induced corrosion. Empirical, stochastic, and mechanistic-based service life models were studied. Two popular mechanistic service life models, Life-365 and fib Bulletin 34, were further evaluated. From the design of experiment and statistical analysis, the important parameters from highest to least importance were found to be diffusion coefficient, concrete cover, aging exponent, surface chloride concentration, and temperature based on the conditions in Florida. This study recommends the fib-34-Bulletin-based model for service life modeling of concrete structures against chloride-induced corrosion due to its inherent ability to consider the associated uncertainties. These uncertainties can arise from adopting simplified models or the unavailability of reliable data. Deterministic service life prediction using Life-365 is a better choice if the user is confident about the field observations and knows the input parameters with a significant degree of certainty. In addition, this study also provided suggestions and recommendations regarding the input parameters for the fib-34-Bulletin-based model.

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EXECUTIVE SUMMARY

Reinforced concrete (RC) structures can deteriorate prematurely due to poor durability performance, with corrosion of reinforcing steel being one of the significant threats to the marine environment. This can lead to cracking, staining, and spalling of the concrete cover, compromising the structure's safety, stability, and aesthetics. Service life models for RC structures are necessary to quantify the condition and level of deterioration at different times during their service life. Several service life models have been developed for RC structures to determine their durability against corrosion. The primary objective of this project was to perform a comprehensive literature search of various service life models and recommend one for adoption by the Florida Department of Transportation (FDOT). In addition, the recommendation for region-specific inputs of key parameters for Florida was also evaluated.

Based on the available literature, empirical, stochastic, and mechanistic service life models of concrete structures for chloride-induced corrosion were studied. Empirical models based on regression could be appropriate for assessing the RC structures at the network level. However, they may need frequent calibration for new out-of-bounds data. Life-365 and fib Bulletin 34 are popular mechanistic service life models based on one-dimensional Fick's diffusion law. Life-365 is a deterministic model, while fib Bulletin 34 is a full probabilistic model. Both models were used to perform sensitivity analyses where one input parameter was varied at a time. However, while this approach provides a general idea of the relative importance of variables, it cannot determine the order of input importance. To address this limitation, a full factorial design of the experiment was conducted using the fib-34-Bulletin-based model followed by statistical analysis. From the statistical analysis, the most important to least important input parameters are found to be diffusion coefficient, concrete cover, aging exponent, surface chloride concentration, and temperature.

While deterministic service life prediction using Life-365 is recommended if the user is confident about field observations and knows the input parameters precisely, it is not ideal when there are variations in material or region-specific input parameters, or the user is less confident about the field observations. In such cases, a probabilistic analysis that considers input variations through distributions and standard deviations is more appropriate. Therefore, the study recommends using the fib-34-Bulletin-based model to consider uncertainties in the analysis inherently.

This study also provided recommendations and suggestions for the input parameters for the fib-34-Bulletin-based service life model. Material-specific input values should be used for diffusion migration coefficient, aging exponent, critical and initial chloride content based on the cement type, water cement ratio, and presence of fly ash, slag, and admixtures. A region-specific input of temperature was not found to be significant for Florida.

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1 INTRODUCTION

Concrete structures can deteriorate prematurely due to poor durability performance. The greatest threat for reinforced concrete is corrosion of reinforcing steel, leading to cracking, staining, and spalling of the concrete cover, which in turn can lead to unserviceable structures that may be compromised in respect of safety, stability, and aesthetics (Alexander, 2018). As per the ASCE 2021 report card, there are more than 617,000 bridges across the United States, 42% of which are at least 50 years old, and 46,154, or 7.5%, are considered structurally deficient. It is thus imperative that highway agencies assess the remaining service life of different bridge elements to help maintain these bridges and ensure adequate serviceability and safety of their existing bridges. Florida has fared above the national average with 65% of its bridges in good condition compared to 45% nationally. In further improving the current state of its bridges, one of the initiatives that FDOT is pursuing is the adoption of an accurate and reliable remaining service life model. Such a model is necessary for the asset management of existing bridges and for its utilization in various phases of the design and construction process of new bridges and other infrastructure components to last 100+ years.

Developing accurate maintenance or rehabilitation schedules and plans for concrete structures to maintain their performance at the lowest cost is a challenge for engineers and asset owners. These schedules and plans are determined based on multiple uncertain variables, including the predictive models of remaining service life. According to the AASHTO LRFD, service life for bridges can be defined as the period of time that the bridge is expected to be in operation. This definition can be extended to other Reinforced Concrete (RC) structures. Moreover, it leads to the need for a good understanding and representative models that capture the different deterioration mechanisms affecting the bridge condition and affect its operation. Service life models for RC structures are mathematical models that quantify the condition, or the deterioration level, of structures at different times during their service life.

Among the different deterioration mechanisms, corrosion is one of the significant mechanisms that can have detrimental impact on the condition of RC elements and structures. Over the years, several service life prediction models have been developed for RC structures considering different corrosion mechanisms. Corrosion can occur in the RC structures due to diffusion of chloride or carbonation. Although both mechanisms are observed, chloride-induced corrosion is predominant, particularly in the marine environment. Typically, carbonation-induced corrosion is less likely to occur unless the structure is built with lower-quality concrete or insufficient concrete cover (Bentz and Thomas, 2001).

Being one of the most common ions, the presence of chlorides in the environment is the primary source of chloride-induced corrosion. Chlorides are also present in the concrete mix components, including the aggregate, cement, and admixtures. However, seawater, groundwater contamination, and de-icing salts (typically calcium chloride) are the most problematic chloride sources. There is a passive oxide layer around the steel in concrete, which protects it from corrosion and grows with time. Chloride ions that accumulate on the surface, known as surface chloride concentration, can reach the steel reinforcement through diffusion and initiate corrosion. Surface chloride can accumulate up to 0.5 inches inside the concrete surface due to moisture (Fanous and Wu, 2005). Sohanghpurwala (2006) reported that the accumulation of chloride ions on the first 0.25 inch of the concrete depends on the exposure conditions, i.e., they can wash away due to the rain, snow, and other sources flowing over the surface. However, the chloride ions accumulated at a depth of more than 0.25 is not affected by exposure. Several service life models have been developed and used to determine the durability of concrete structures against corrosion. The objective of this report is to review the state of practice and state of the art of the remaining service life models of the reinforced concrete bridges. Based on the review of the available models, a service life model of concrete structures for chloride induced corrosion will be proposed for adoption by the Florida Department of Transportation (FDOT). This report will also provide suggestion on the necessary input parameters of the suggested model.

2 LITERATURE REVIEW

Service life models can be empirical or mechanistic models. Empirical models are derived by fitting an equation, typically a polynomial function, to capture the overall trend of the observed measurements. The fitted functions are not selected based on an engineering model preceding the observations. It is instead an iterative process depending on the judgment of the modeler to choose a function that captures the observations trend. Alternatively, mechanistic models start by using fundamental engineering models that describe a deterioration mechanism, such as fatigue or corrosion, and then use these models to describe the change in deterioration over time. In addition to that general classification, these models can be deterministic or probabilistic. A deterministic model describes the condition or deterioration level by a single state or value at any given point in time. These models could include a description of the uncertainty in this prediction, but it describes the condition over time using a unique trend. On the other hand, probabilistic, also referred to as stochastic, models describe the distribution of possible conditions or deterioration levels over time. Commonly, three models have been adopted to model the service life of RC structures, these are:

a) Empirical-Deterministic models (Empirical models for short)
b) Empirical-Stochastic models (Stochastic models for short)
c) Mechanistic-Deterministic models (Mechanistic models for short)

2.1 Empirical Models

Empirical service life models are well-suited for network-level assessment. At the network level the conditions and their trends over time vary significantly, and an average model that provides an overall trend is more feasible. Bolukbasi, et al. used 2601 bridge rating data from National Bridge Inventory (NBI) to develop a regression-based condition rating equation with bridge age *t* as the variable as sown in equation (1) (Bolukbasi et al., 2004).

$$C(t) = a_0 + a_1 t + a_2 t^2 + a_3 t^3 \tag{1}$$

here, *C(t)* is the condition rating of the bridge at age *t*, and a_0, a_1, and a_3 are the regression coefficients. They set the minimum rating value of three as the time when the bridge would require maintenance.

Stukhart, et al. have evaluated several linear, piecewise linear, and nonlinear equations to predict the condition rating of the bridges using the NBI data for Texas bridges or from expert opinion (Stukhart et al., 1990). Condition Rating (*CR*) for the deck, superstructure, and substructure were determined using piecewise linear equations shown in equation (2). The model includes break points t_1 and t_2 to be 25 and 45 years. These points corresponding to the time when corrosion initiates, and the time when deterioration starts accelerating towards the limit state.

$$CR = \begin{cases} a_0 + a_1 t, & if\ t \leq t_1 \\ a_0 + a_1 t_1 + a_2(t - t_1), & if\ t_1 \leq t \leq t_2 \\ a_0 + a_1 t_1 + a_2(t_2 - t_1) + a_3(t - t_2) & if\ t > t_2 \end{cases} \tag{2}$$

2.2 Stochastic Models

Researchers have also used stochastic Markov models to describe the deterioration of bridge structures (Gao et al., 2019). In the Markov chain approach, the bridge condition is divided into several discrete states. Thus, the condition state probability vector (X_t) of the bridge at year t can be represented as

$$X_t = [X_{1t}, \ldots\ldots, X_{nt}] \tag{3}$$

here, X_{nt} is the probability that the bridge stays in state i after year t. The deterioration of the bridge can be represented using the change in the state probability vector. Transition Probability Matrix (TPM) is applicable for this purpose.

$$P_t = \begin{pmatrix} P_{11t} & \cdots & P_{1nt} \\ \vdots & \ddots & \vdots \\ 0 & \cdots & 1 \end{pmatrix} \tag{4}$$

P_t is the transition probability matrix of year t, and P_{ijt} is the probability of bridge condition deterioration from i to j in year t. The last of entry of one in equation (4), indicates the bridge condition at state n, i.e., the bridge cannot deteriorate any further. Without any maintenance, the bridge deterioration can be estimated using equation (5)

$$X_{t+1} = X_t P_t \tag{5}$$

The main advantage of the stochastic method is that it can consider the uncertainty in the deterioration of the concrete structures, which the empirical-deterministic model cannot consider. However, the Markov chain model does not account for the historical records of performance and conditions of an individual bridge.

2.3 Mechanistic Models

Although regression-based empirical or stochastic models are easier to implement, they are rarely used in the design phase. Primarily, there are two strategies for structural design. The first strategy provides the necessary means to the structure so that it never reaches the limit states under environmental or service load. This strategy is achieved by selecting one or a combination of the following:

a) Selecting a durable material that can withstand the deterioration experienced during the design period
b) Adopting protection measures, such as using coatings in reinforcement or steel members
c) Adjusting the dimensions of the structures to reduce the rate of deterioration, such as using concrete cover and reinforcement size
d) Selecting shorter service life spans for the members that have a faster deterioration rate so that their deterioration does not govern the service life, such as joints and bearings

The second strategy is the removal of the vulnerabilities that cause deterioration by adopting appropriate design methods, such as the removal of vulnerable joints or using stainless steel.

Under the first strategy, there are three design approaches:

1. Full Probabilistic – This method explicitly calculates the reliability indices of a specific limit state based on the deterioration models.
2. Partial Factor – This method uses load and resistance factors to evaluate a particular limit state based on a given reliability index.
3. Deemed-to-Satisfy – This method provides a set of requirements, which, if followed, will result in a structural design life higher than the specified reliability.

The most used design approach is the deemed-to-satisfy method due to the unavailability of reliable deterioration models and the identification of precise limit states. However, a notable exception is the availability of the chloride and carbonation ingress mechanistic models for concrete structures.

Mechanistic models are mainly diffusion-based models. Past studies have considered three distinct time periods, or stages, for predicting the service life of RC bridges experiencing corrosion-induced damages, as shown in Figure 2-1. These stages are:

1) Time to corrosion initiation, t_1- It is the time required for the chloride ions built up at the surface to reach the passive protective layer around the reinforcement.
2) Time from initiation to cracking, t_2- It is the time when corrosion is formed, and damage starts accumulating in the concrete to start forming cracks and spalling.
3) Time for corrosion damage to reach a limit state, t_3.

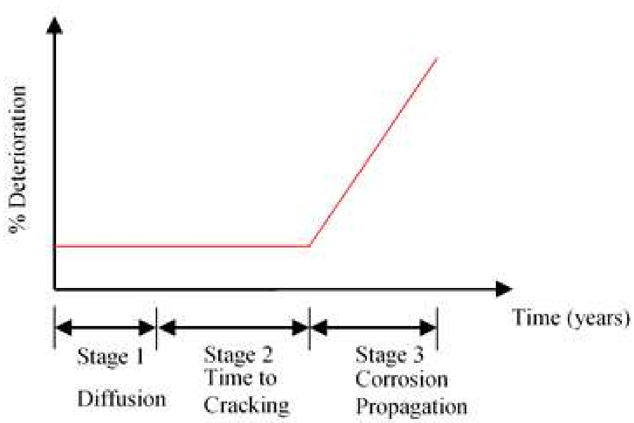


Figure 2-1. Different stages of corrosion-induced damage in RC structures

2.3.1 Time to Corrosion Initiation, t_1

Generally, the corrosion process in the RC structure is modeled using Fick's second law of diffusion. It is assumed that the corrosion at the rebar initiates when the chloride concentration exceeds the threshold concentration. Fick's law of diffusion can be represented by the following equation:

$$\frac{\partial C}{\partial t} = D_a \frac{\partial^2 C}{\partial x^2} \tag{6}$$

here, C is the chloride concentration, D_a is the apparent diffusion coefficient, x is the depth from the exposed surface, and t is the time. Past studies have used the error function for the solution of the above equation, which is represented by equation (7):

$$C(x,t) = C_s \left[1 - erf\left(\frac{x}{\sqrt{4D_a t}}\right)\right] \tag{7}$$

here, C is the chloride concentration at time t and depth x, C_s is the chloride concentration at the surface, and *erf* is the statistical error function. According to Hu et al. (2013) equation (8) can be used to determine the time to corrosion initiation t_1 with a known concrete cover, x_d and chloride threshold concentration C_t (Hu et al., 2013),

$$t_1 = \frac{x_d \left[erf^{-1}\left(1 - \frac{C_t}{C_s}\right)\right]^{-2}}{4D_a} \tag{8}$$

2.3.2 Time from Initiation to Cracking, t_2

Liu (1996) proposed a method of corrosion-induced crack initiation in the concrete based on the accumulated rust volume around rebar. According to Liu (1996), there is a porous space (W_p) between the concrete and the rebar. As the corrosion initiates, this porous space gets filled by the "corrosion products" (W_T), which is the accumulated volume of rust. The maximum amount of corrosion products that can be accommodated in the porous space without any crack initiation is denoted by W_{crit}. As soon as the amount of accumulated corrosion, W_T, exceeds the critical value, W_{crit}, expansive pressure induces a crack on the surrounding concrete, as shown in Figure 2-2.

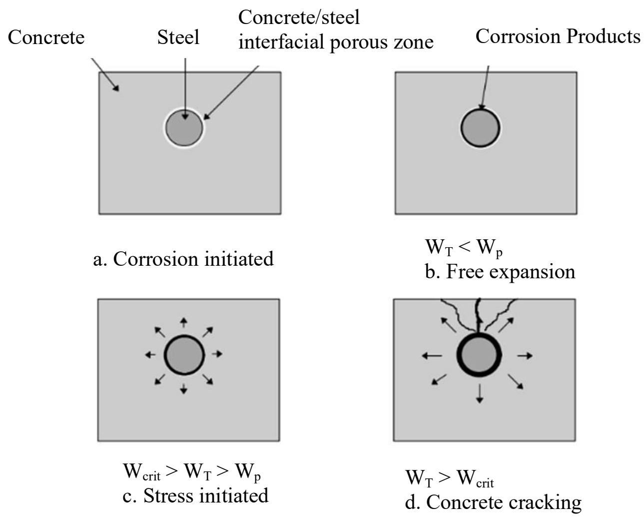


Figure 2-2. Corrosion initiation and cracking process (source: Liu, 1996)

Using equation (9), the critical amount of rust required to initiate a crack can be determined.

$$W_{crit} = \rho_{rust}\left[\pi D\left\{\left(\frac{x_d f_t}{E_{ef}}\left(\frac{a^2+b^2}{b^2-a^2}\right)+v_c\right)+d_0\right\}+\frac{W_{st}}{\rho_{st}}\right] \tag{9}$$

Where, ρ_{rust} is the density of rust (226 lb/ft^3), d_0 is the porous space between reinforcement steel and concrete, ρ_{st} is the steel density, W_{st} is the corroded steel amount, D is the diameter of the steel, a is the inner radius, b is the outer radius of the thick walled cylinder surrounding the rebar, as shown in Figure 2-3, f_t is the tensile strength of concrete, and v_c is the Poisson's ratio of concrete.

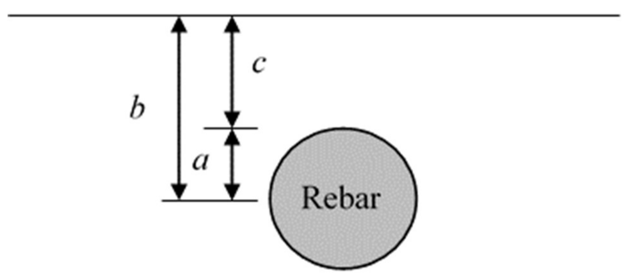


Figure 2-3. Input parameters of thick-walled cylinder around corroded rebar (not drawn to scale)

If rust production rate, k_p, is determined using equation (10)

$$k_p = \frac{0.098}{\alpha}\pi D i_{corr} \tag{10}$$

Then, using equation (11), time from corrosion initiation to concrete cracking can be determined.

$$t_2 = \frac{{W_{crit}}^2}{2k_p} \tag{11}$$

here, i_{corr} is the annual mean corrosion rate (mA/ft^2).

2.3.3 Time Required for Corrosion Damage Propagation to Reach a Limit State, t_3

The estimation of time to reach limit state (t_3) is typically based on regression equations. Andrade & Alonso converted corrosion current to rebar diameter to determine the remaining service life (Andrade and Alonso, 1996). Williamson used deterioration percentages, and Hu, Haider, and Burgueno used crack width (Williamson, 2007; Hu et al., 2013). Williamson defined the 12% deterioration level as the end of functional service life for a bridge deck. They have surveyed seven bridge decks in Virginia to develop the following estimation model:

$$t_3 = 8.61\left(\sqrt{D_r + 1.38} - 1.45\right) - 3.34 \tag{12}$$

here, D_r is the deterioration percentage, the limit is 12%.

Crack width has also been associated with the end of service life. Although some European countries set the crack width limit to be 0.2 mm, some studies have suggested that a crack width below 0.3 mm does not have a significant impact on the ingress of chloride to the concrete (Williamson, 2007). Hu et al. (2013) used equation (13) to determine the crack width at the surface

$$w_s = 0.1916\Delta A_s + 0.164 \tag{13}$$

here, ΔA_s is the average loss of rebar cross sectional area.

Vu et al., (2005) also developed an empirical equation to determine the time required for excessive cracking in concrete based on water cement ratio, concrete cover, corrosion current density, which is shown in equation (14)

$$t_3 = t_2 + k_r \times 0.0114 i_{corr}\left[A\left(\frac{x_d}{wc}\right)^B\right] \tag{14}$$

here, A, and B are constants based on the crack width limit states, x_d is the concrete cover, wc is the water cement ratio, K_r is the loading rate correction factor, i_{corr} is the corrosion current density ($\mu A/cm^2$), t_2 is the crack initiation time defined earlier.

After the corrosion damage propagates to the limit state, the crack will reach the surface; and at that time, the determination of the crack width is most critical. Crack typically takes the shortest path from the rebar to the surface, and its width increases from the sources of initiation to the surface, similar to the shape of a trapezoid, as shown in Figure 2-4 (Zhao et al., 2012). To determine the crack width, Chun-Qing et al. (2006) et al. has proposed an analytical model, which requires the determination of a stiffness reduction factor. Determination of the stiffness reduction factor is not a straightforward process, it requires information regarding the concrete material properties. Furthermore, the model was developed considering plane stress criteria, although the plane strain solution is more applicable to the corrosion crack propagation in concrete. Zhao et al. (2012) and Chernin et al. (2012) used regression or empirical equations to predict the crack width, but the success was limited. The models either underestimated or significantly overestimated the crack widths. The reason for the over-prediction of the crack width is the usage of simplified models for corrosion initiation, propagation, and growth (Chernin et al., 2012). The primary disadvantage of these mechanistic models is their deterministic approach, i.e., inability to consider the inherent uncertainties involved in the service life prediction.

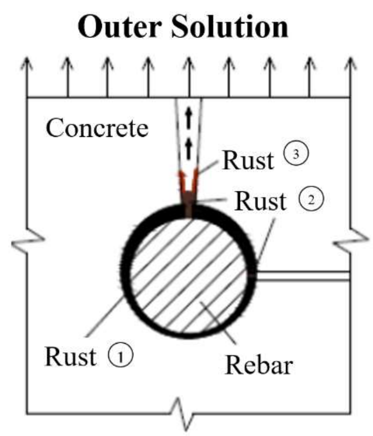


Figure 2-4. Crack shape from rebar to surface (source: Zhao et al., 2012)

3 SERVICE LIFE MODELING TOOLS

Several commercial software tools have been developed to implement and fit service life models and provide predictions of remaining service life. This section summarizes the information related the available commercial programs to identify the gaps or limitations in the current knowledge base.

3.1 Life-365

Remaining service life modeling of RC structures in Life-365 is carried out using 4 steps:

1. Predicting the time for corrosion initiation, which is referred to as the initiation period, t_i;
2. Time required for corrosion to reach an unacceptable level is referred to as the propagation period, t_p
3. Determining the repair schedule after the first repair
4. Estimate life-cycle costs based on the initial and future repair costs.

Similar to the mechanistic models found in the literature, Life-365 also calculates the corrosion initiation time based on the Fickian diffusion model presented in equation (6). In the software, the apparent diffusion coefficient ($D(t)$) is dependent on both time and temperature. Equation (15) represents the relation of diffusion coefficient with time (t)

$$D(t) = D_{ref}\left(\frac{t_{ref}}{t}\right)^{m} \tag{15}$$

where, m is the decay constant, D_{ref} is the reference diffusion constant at reference time (t_{ref}) of 28 days. The software selects the value of m and D_{ref} based on the user input of mixture properties, i.e., water-cement ratio and type and proportion of cement used. The value of $D(t)$, calculated by equation (15), is only valid for up to 25 years; beyond that it remains constant. In addition to time impacts, the software accounts for temperature impacts using equation (16):

$$D(T) = D_{ref} \times e^{\left[\frac{U}{R}\left(\frac{1}{T_{ref}}-\frac{1}{T}\right)\right]} \tag{16}$$

here, U is the activation energy (35000J/mol), R is the gas constant, T is the absolute temperature, T_{ref} is the absolute reference temperature of 293 K (68°F).

Another important parameter for Life-365 is the maximum surface chloride concentration, C_s. The software calculates this value based on the user input about structure type, geographic location, and exposure. There are seven exposure conditions in Life-365. They are:

a) Marine splash zone (in tidal range or within 3.3 ft (1m) of the high-tide level)
b) Marine spray zone (more than 3.3 ft (1m) above high tide level but occasional exposure to salt water)
c) Within 0.5 mile (800 m) of the ocean

d) Within 0.93 mile (1500 m) of the ocean
e) Parking garages
f) Rural Highway bridges
g) Urban Highway bridges

Figure 3-1 shows the adoption of different surface chloride concentration zones in North America in the Life-365 software.

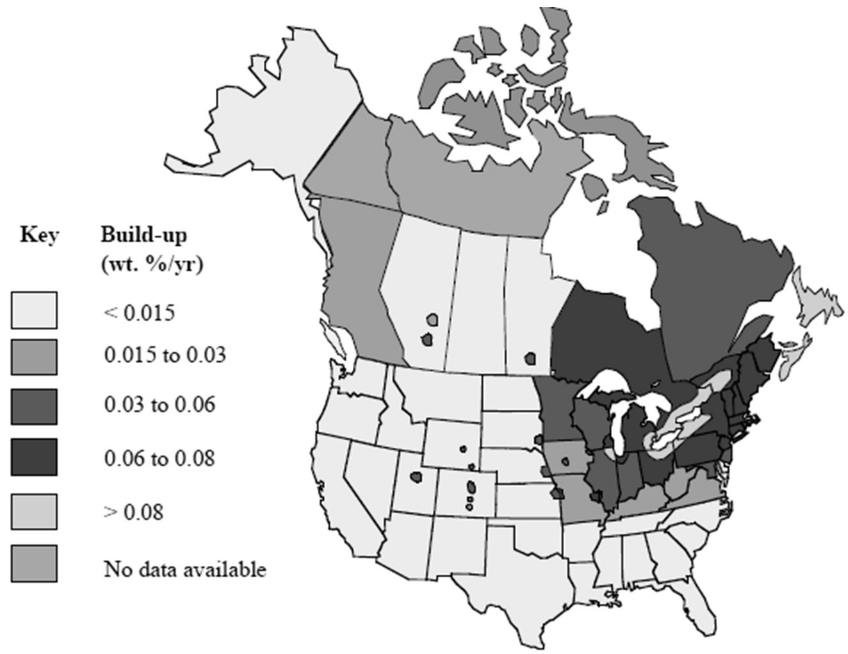


Figure 3-1. Surface chloride concentration zones in North America (source: Life-365)

For structures situated in the marine conditions, the software has the following default values shown in Table 3-1:

Table 3-1. Surface Chloride Build-up Rate in Different Locations

Location	Build up Rate (% per year)	Maximum (%)
Marine Splash Zone	Instantaneous	0.8
Marine Spray Zone	0.1	1.0
Within 0.5 mile of Ocean	0.04	0.6
Within 0.93 mile of Ocean	0.02	0.6

The surface chloride build-up rate can vary between 0.004 to 0.1% per year, and it is a function of moisture and proximity to ocean breezes. Therefore, it is important and recommended that the user checks the default value against the available local data. The software also allows users to set their C_s value or determine the surface chloride concentration according to ASTM C1556. Following the procedure outlined in ASTM C1556, the software allows the user to provide inputs regarding the variations of chloride concentration with depth. The surface chloride concentration and apparent diffusion coefficient are determined through the minimization of the following objective function:

$$\sum (C_x^t - C(x,t))^2 \tag{17}$$

here, C_x^t is the experimental surface chloride concentration data, and $C(x, t)$ is determined based on equation (7).

Life-365 also considers the presence of fly ash, slag, and silica fume in the concrete mixture. It changes the apparent diffusion coefficient and decay rate (m) based on the percent of silica fume, fly ash, or slag present in the mixture.

A chloride concentration threshold (C_t) value of 0.05% by mass of concrete is adopted in Life-365. This simplistic adoption of the threshold value is based on the comprehensive literature review by Glass & Buenfeld (2000). It is important to note that such a simplistic approach may not provide an accurate estimate of the remaining service life as the chloride threshold may depend on the microstructural composition of the concrete, such as porosity, pore structure, other ions, admixtures, type of cement, temperature, and moisture.

The primary advantage of Life-365 is that it provides a mechanistic-based service life prediction of the concrete structure considering several factors, such as the effect of various mix properties, corrosion inhibitors, and admixtures. However, in some instances, the assumptions are too simplified to yield an accurate result (Hu et al., 2013). For example, a corrosion propagation period of only six years is not always realistic. The developers of Life-365 recommend verifying the default values adopted by the software with the local values for accurate estimation of the remaining service life of RC structures, at least for the critical inputs (Bentz and Thomas, 2001).

Several researchers have compared the Life-365 predicted values with those observed in the field. Pack et al. (2010) studied 11 concrete bridges located in the coastal area to identify the effect of two inputs mentioned above: apparent diffusion coefficient, $D(t)$, and surface chloride concentration, C_s. They have developed a model of chloride transport within concrete structures and compared the results with Life-365. From the results, it was observed that for the ordinary Portland cement (OPC) concrete, the diffusion coefficient predicted by the Life-365 was around 3.32×10^{-12} m^2/s, and their developed model predicted the value to be around 5.81×10^{-12} m^2/s, which is closer to the in situ value (ranged from 4.32×10^{-12} to 7.89×10^{-12} m^2/s). This is due to the fact Life-365 overestimates the diffusion decay rate. Furthermore, they studied two types of concrete mixes with exposure times of around 22.54 and 11.36 years. During the analysis, surface chloride concentration values changed from initial values of 1.38% and 2.01% to 2.54% and 2.69% for the two mixes, respectively. The in situ chloride surface concentrations were measured to be around 2.36% and 2.54%. The chloride surface concentration of Life-365 was constant, around 3.9%.

Stipanovic Oslakovic et al. (2010) also evaluated a coastal bridge in Croatia using Life-365. The bridge was in service for more than 25 years at the time of their study. High salinity, high wind, and high temperature accelerated the penetration of chlorine inside the concrete structure, leading to corrosion and microcracking. It was expected that surface chloride concentration will decrease as the elevation above sea level increases. However, they have observed that due to high wind, chloride surface concentration can be very high, even 40-60 m above sea level. This type of microclimatic difference can pose a challenge to define any uniform parameter for the entire structure. Stipanovic Oslakovic et al. also compared the experimental surface chloride concentration (C_s) profile with Life-365 prediction (Stipanovic et al., 2010). Similar to Pack et al. (2010), they observed that reducing the surface chloride concentration matches well with the field values. They also reported the diffusion coefficient overestimation by Life-365. In addition, they performed the parametric study by varying the diffusion coefficient and concrete cover to obtain a range of times of corrosion initiation, which could be helpful for rehabilitation purposes.

3.2 STADIUM Software Model

Stadium is a multi-ionic transport model where the governing equation is based on the Nernst-Planck equation. Unlike Life-365, it can be applied to unsaturated media with variable temperatures. The governing equation considers the electrical coupling, chemical activity between ion fluxes, and transport of ions due to water content gradient and temperature effects. The governing equation of Stadium is shown in equation (18).

$$\frac{\partial(wc_i)}{\partial t} - \nabla\cdot\left(D_i w\nabla c_i + \frac{D_i z_i F}{RT} wc_i\nabla\psi + D_i wc_i\nabla(\ln(\gamma_i)) + \frac{D_i c_i \ln(\gamma_i c_i)}{T}\nabla T + c_i D_w \nabla w\right) = 0 \quad (18)$$

here c_i is the concentration, w is the water content, D_i is the diffusion coefficient, z_i is the valence number of ionic groups, F is the Faraday number, ψ is the electro diffusion constant, T is the temperature, γ_i is the activity coefficients, D_w is the water diffusivity. The ions that were considered in the Stadium are: OH^-, Na^+, K^+, SO_4^{2-}, Ca^{2+}, $Al(OH)^-_4$, Mg^{2+}, and Cl^-.
There are a total of 11 variables in the system of equations: eight ionic concentration, electro diffusion constant, water content, and temperature. The Newton-Raphson method is used to solve these 11 non-linear equations simultaneously. For spatial discretization, the Galerkin-based finite element approach is used, while for time discretization implicit Euler method is used. The software uses a chemical equilibrium module to check for the equilibrium of concentration at each node of the finite element mesh.

The necessary inputs of the stadium software are as follows: material density, paste content, diffusion coefficients, water diffusivity, total porosity, capillary porosity, initial values of ion concentration, the volumetric water content in the pores, electrical potential, initial amount of solid phases, equilibrium constants, boundary conditions for ion concentration, and temperature (Zatar, 2014).

The primary advantage of Stadium software is that it considers not only the chloride ion transportation due to diffusion but also the movement of chloride ions due to the interaction of multiple ions, water movement in the unsaturated media, and temperature. However, the Stadium software calculates the material degradation based on the mass transfer alone; it does not consider the concrete microstructure or its properties. Furthermore, the software does not consider any corrosion propagation period, which may lead to inaccurate estimation of the remaining service life of RC bridges.

3.3 fib Bulletin 34

fib Bulletin 34 Model Code for service life design performs a full probabilistic analysis of the remaining service life of the uncracked concrete structures subjected to chloride-induced corrosion damages (Schiessl, et al., 2006). Similar to Life-365, it also uses Fick's 2nd law for chloride diffusion into the concrete structure. However, it uses a full probabilistic design approach to account for the uncertainties associated with the input parameters and conditions. According to fib, the approximate solution of Fick's equation to estimate the chloride concentration at any location x inside the concrete at time t is determined using equation (19)

$$C(x=a,t) = C_0 + \left(C_{s,\Delta x} - C_0\right)\left[1 - erf\left(\frac{x_d - \Delta x}{2\sqrt{D_{app,c}t}}\right)\right] \tag{19}$$

where, C_0 is the initial chloride concentration, which is the result of chloride present in aggregates, cement, and water during the construction phase, $C_{s,\Delta x}$ is the surface chloride concentration at a depth Δx from the surface, *erf* is the error function, x_d is the depth of the reinforcing steel and $D_{app,c}$ is the apparent diffusion coefficient.

The apparent diffusion coefficient is determined using equations (20), (21), and (22)

$$D_{app,c} = k_e D_{RCM,0} k_t A(t) \tag{20}$$

$$k_e = e^{\left[b_e\left(\frac{1}{T_{ref}} - \frac{1}{T_{real}}\right)\right]} \tag{21}$$

$$A(t) = \left(\frac{t_0}{t}\right)^{\alpha} \tag{22}$$

here; k_e is the environmental transfer function, k_t is the transfer parameter taken as one, and $A(t)$ is the aging function, T_{ref} is the reference temperature and T_{real} is the ambient temperature, t_0 is the time reference point of 28 days expressed in years, and α is the aging factor.

As it is a probabilistic model, instead of constant values, the inputs are provided in the form of mean values, standard deviation, and probability distributions. Failure is defined when the chloride concentration ($C_{s,\Delta x}$) exceeds critical chloride concentration (C_{crit}). Using the random sampling of the input parameters, the software uses Monte Carlo simulation to estimate the distribution of chloride concentration based on equation (19). According to fib bulletin 34, the acceptable probability of failure is 10%, which is represented by the reliability index of 1.3. It means if the reliability of the particular run is below 1.3, then it is assumed that the bridge will fail due to chloride-induced corrosion. The final probability of failure is calculated based on the results of the full set of analyses, which typically consists of several thousands of analyses runs.

Several state Departments of Transportations (DOTs) have used fib Bulletin 34 to determine the remaining service life of bridges. Virginia and Pennsylvania DOT have already developed fib-Bulletin-34-based model, and Oregon DOT is also in the process of developing its own tool.

FHWA has also developed a fib based probabilistic model for predicting the service life model of bridges as a part of the SHRP2 program (Pease et al., 2019).

3.3.1 Virginia

Virginia DOT (VDOT) uses region-specific inputs for calculating the remaining service life of corrosion-induced structures. Apart from fib Bulletin 34, the VDOT model relied on the work of Williamson et al. (2007) and Bales et al. (2018). Williamson et al. divided Virginia into six regions based on the exposure conditions, which was later adopted by Bales et al., as shown in Figure 3-2. The severity of winter varies significantly from one region to the other. Furthermore, the regions also have different maintenance schedules and procedures based on their environment. For example, in the Tidewater region, the application rate of de-icing salt is around 0.4 tons/mile, while the value increases to 7.75 tons/mile in the northern region. In addition, Williamson et al. (2007) also showed that the concrete cover depth of the bridges varies by region and concrete mix properties, particularly the water-cement ratio. Thus, VDOT adopted region-specific values for the temperature, surface chloride concentration, and concrete cover due to the variations in climate and maintenance procedures among the regions. Apart from regions, Williamson et al. (2007) also performed statistical analysis to show significant variations in chloride migration coefficients based on the concrete mix types. fib Bulletin 34 and Ji et al. also demonstrated the variation in critical chloride concentration and aging coefficients based on the concrete and rebar types (Schiessl et al., 2006; Ji et al., 2005). Therefore, VDOT uses material-specific values for the critical chloride concentration, aging coefficient, and chloride migration coefficient, along with other region-specific values.

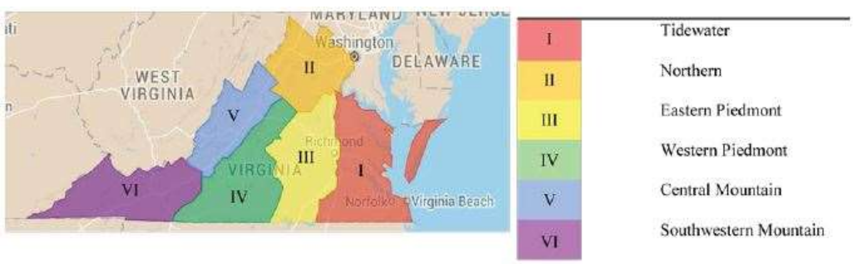


Figure 3-2. Different regions of Virginia for remaining service life model (courtesy: Bales et al., 2018)

One of the important aspects of the remaining service life analysis of bridges with chloride-induced corrosion is the availability of the data. VDOT developed a database of input parameters from historical data, past literature, or field testing (Pease et al., 2019; Cady and Weyers, 1983). For example, region-specific values of the surface chloride concentration were obtained from the Bales et al. study, while the input parameters for the chloride migration coefficients and initial chloride concentration were obtained from field testing according to the Nordtest NT build 492, and Nordtest NT build 208 or ASTM C1152 (ASTM C1152, 2020; NT Build 208, 1996; NT Build 492, 1999; Bales et al., 2018). Region and material-specific input parameters used by VDOT are given in Table 3-2, Table 3-3, and Table 3-4.

3.3.2 SHRP2 R-19A

Similar to VDOT, the remaining service life developed by the SHRP2 program also has different inputs based on the exposure conditions. It considers four exposure conditions: atmospheric zone with severe deicing salts, splash zone with moderate deicing zone, submerged zone, and zone without deicing salts (Langlois and Bartholomew, 2015). The input parameters were obtained from the past literature, Duracrete/DARTS investigation, and various other sources (Engelund rt al., 2000; Gehlen and Kapteina, 2004). The input parameters adopted by the SHRP2 program are provided in Table 3-2.

3.3.3 Pennsylvania

Pennsylvania DOT has also sponsored the development of a fib-Bulletin-34-based full-probabilistic design tool for determining the remaining service life of concrete bridges. The model is called ProCIM (Naito et al., 2016). During the development of ProCIM, Naito et al. (2016) considered chloride migration coefficients for different concrete mixes. For other input parameters, fib-recommended default values were adopted. Two exposure conditions were considered: splash road environment: which correlates to the concrete structure being located within 1.5 m of the chloride source, and spray road environment, which correlates to the concrete structure being located more than 1.5 m of the chloride source. The inputs adopted in ProCIM are given in Table 3-2. According to fib*,* failure occurs when the chloride concentration at a certain depth exceeds the critical chloride concentration. From the results, it was observed that almost all the mixes considered in the study failed. The failure may have occurred due to the usage of uncoated rebar in the simulation, which typically accelerates the failure. Using material-specific input values would have yielded more reasonable results.

Table 3-2. Input Parameters for fib-Bulletin-34-based Models

Variable Name	Unit	Mean (μ)			Standard Deviation			Distribution		
		SHRP 2	VDOT	ProCIM (PennDOT)	SHRP 2	VDOT	ProCIM (PennDOT)	SHRP 2	VDOT	ProCIM (PennDOT)
Critical Concentration, C_{crit}	Mass of % Binder	0.6	Table 3-4	0.6	0.15	Table 3-4	0.15	Beta	Table 3-4	Beta
Initial Chloride Content, C_o	Mass of % Binder	0.1	0.034	0.1	NA	0.021	NA	Deterministic	Normal	Deterministic
Surface Chloride Concentration, C_s	Mass of % Binder	1.75	Table 3-3	3	0.5µ	Table 3-3	1.5	Log-normal	Table 3-3	Log-normal
Concrete cover, x_d	Inch	2	Table 3-3	2.5	0.24	Table 3-3	0.25	Normal	Table 3-3	Log-normal
Transfer Function, Δx	Inch	0	0.5	0	NA	NA	NA	Deterministic	Deterministic	Deterministic
Ageing factor, α	NA	0.65	Table 3-4	0.45	0.15	Table 3-4	0.2	Beta	Table 3-4	Beta
Temperature, T_{real}	°F	58.3	Table 3-3	55.9	15.7	Table 3-3	15.7	Normal	Table 3-3	Normal
Chloride Migration Coefficient	$\times 10^{-9}$ in^2/s	15.5	Table 3-4	7.567	0.2µ	Table 3-4	0.182	Normal	Table 3-4	Normal

Table 3-3. VDOT Region Specific Parameters

Regions	Ambient Temperature, T_{real} (F)		Surface Chloride Concentration, Cs (% of wt. cl./wt. binder)		Cover Depth, *a* (inch)	
	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
Central Mountain	53.6	14.6	1.13	0.72	2.4	0.3
Eastern Piedmont	56.3	14.4	0.78	0.18	2.3	0.3
Northern	57.4	14.7	0.99	0.43	2.1	0.3
Southwestern Mountain	53.1	14.1	1.57	0.73	2.3	0.3
Tidewater	59.1	14.3	0.42	0.27	2.2	0.3
Western Piedmont	54.8	15.1	1.33	0.54	2.1	0.2
Distribution	Normal	Normal	Log-normal	Log-normal	Log-normal	Log-normal
Source	Bales (2018)	Bales (2018)	Bales (2018)	Bales (2018)	Williamson (2007)	Williamson (2007)

Table 3-4. VDOT Material Specific Inputs

Variable Name	Material	Distribution	Mean	Standard Deviation
Critical Concentration, C_{crit} (wt. cl./wt. binder)	Carbon Steel	Beta	0.65	0.15
	MMFX	Log-normal	1.08	0.443
	Class I CRR	Deterministic	2.4	NA
	Class II CRR	Deterministic	6	NA
	Class III CRR	Deterministic	3.6	NA
Aging factor, α	PCC	Beta	0.3	0.12
	PCC w/ Fly Ash	Beta	0.6	0.15
	PCC w/ Slag	Beta	0.45	0.2
Chloride Migration Coefficient ($\times 10^{-9}$ in^2/s)	PCC	Normal	1.55	0.31
	PCC w/ Fly Ash	Normal	27.9	4.2
	PCC w/ Slag	Normal	10.2	2.94

3.3.4 AASHTO Guide Specification for Service Life of Highway Bridges (NCHRP 12-108)

AASHTO has also adopted fib Bulletin 34 based probabilistically calibrated Deemed-to-Satisfy model for corrosion in concrete structures. AASHTO guide specification considers four environments for concrete corrosion: nonaggressive, buried, deicing, and marine. Marine environments were further subdivided into atmospheric, submerged, tidal, or splash/spray zone, which is defined in Figure 3-3.

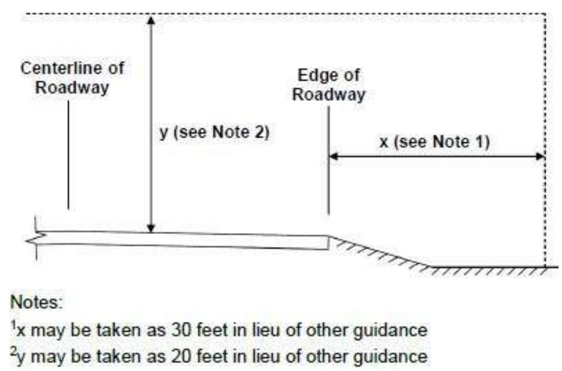


Figure 3-3. Splash/spray zone according to AASHTO Guide Specification (source: Murphy et. al., 2020)

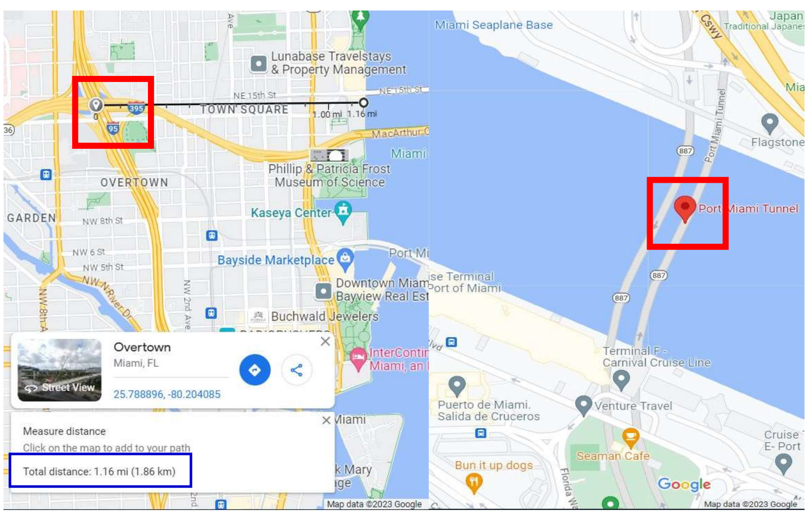
3.3.5 Florida

Florida Department of Transportation (FDOT) and State Materials Office (SMO) have done several service life modeling analyses for concrete bridges and tunnels. Both deterministic and probabilistic service life modeling were conducted. Jacobs Engineering (JE) performed service life modeling of the Port of Miami Tunnel (POMT) for FDOT (Sagues et al., 2012). The probabilistic analysis was conducted using Duracrete/DARTS. For deterministic analysis, a constant propagation period of 10 years was assumed. The expert panel also agreed with this assumption, given other variabilities in the analysis. Furthermore, the expert panel also suggested that the global anode and coatings of the inners of the liner may not be ideal for corrosion control due to the presence of stray current and enhanced moisture retention of the coatings. FDOT also conducted service life estimation of the super and substructure of the I-395 signature bridge on Miami using fib Bulletin 34 and Life-365, respectively (Canjura, 2020; Pease B. J., 2021). The service life analysis showed that the superstructure has a 90% probability of surviving the 90-year design period, and the substructure has a design life of 465 years at 50% reliability. These reports also mentioned the current service life models' inability to consider cracks and repair. Based on these three studies by FDOT around Miami, the chloride diffusion coefficients at 28 days and surface chloride concentration are shown in Table 3-5. It is important to note that surface chloride concentration as % weight of cement of the I-395 arch bridge foundation is determined based on the concrete mix proportion of 1:2:4 and concrete density of 4000 lb/yd^3.

Table 3-5: Chloride Diffusion Coefficient Around Miami

Project	Year	Component	Type	Distance from shore (mi)	Chloride Diffusion Coefficient, D_{28} (m^2/s)	Surface Chloride Concentration (%wt. of cement)
I-395 Arch Bridge	2021	Arch	Superstructure	1.16	7.8×10^{-12} (max allowed, not field data)	1 (analysis assumption)
I-395 Arch Bridge	2020	Foundation	Substructure	1.16	5.24×10^{-13}	3.5 (maximum value assumed)
POMT	2012	Concrete shell surface	Substructure	0	3.96×10^{-12}	2.8

Figure 3-4 shows the locations of the I-395 signature bridge and POMT and their distance from shoreline.



(a) I-395 Signature Bridge (b) Port Miami Tunnel

Figure 3-4: Project locations around Miami and distance from shoreline (source: Google Maps)

Similar to AASHTO Guide specification, Florida DOT also has an environmental exposure criterion It considers three environmental exposure conditions-extremely aggressive, moderately aggressive and slightly aggressive.

Currently, Florida DOT does not use full probabilistic analysis. Thus, it does not require the distribution of the input parameters. As AASHTO guide specification uses probabilistic analysis for chloride ingress, it requires mean, standard deviation, and distribution type for the input parameters similar to SHRP 2 and VDOT approaches. Input parameters adopted by AASHTO guide specification for various environmental exposure conditions are shown in Table 3-6.
It is important to note that temperature is an important factor in concrete corrosion, particularly for determining the rate of diffusion, as shown in Eq. (21). Thus, the AASHTO guide considers two mean annual temperatures for the marine environments: one for typical coastal conditions (east or west coast) in the US, 64°F; another for hot marine environments of Florida or Hawaii, 77°F.

Table 3-6. Input Parameters for Various Environmental Exposures (source: Murphy, et al., 2020)

Input Parameter		Temperature			Surface Chloride Concentration			Depth of Convection Zone				
Exposure Zone		Dist.	Mean (°F)	SD (°F)	Dist.	Mean (%[1])	SD (%)	Dist.	Mean (in)	SD (in)	a (in)	b (in)
Deicing	Underside of Deck	Normal	50	3.6	Log-Normal	1	0.5	0	0	0	0	0
	Atmospheric	Normal	50	3.6	Log-Normal	2	1	0	0	0	0	0
	Indirect Deicing	Normal	50	3.6	Log-Normal	2	1	Beta	0.35	0.22	0	2
	Direct Deicing-Low	Normal	50	3.6	Log-Normal	3	1.5	Beta	0.35	0.22	0	2
	Direct Deicing-High	Normal	50	3.6	Log-Normal	4	2	Beta	0.35	0.22	0	2
Marine	Atmospheric	Normal	64 [2]	3.6	Lognormal	2	1	0	0	0	0	0
	Submerged	Normal	64 [2]	3.6	Lognormal	3	1.5	0	0	0	0	0
	Tidal or Spray	Normal	64 [2]	3.6	Lognormal	4	2	Beta	0.35	0.22	0	2
Buried	Buried	Normal	64		Lognormal	0.6	0.3	0	0	0	0	0
Non-aggressive	Interior	Normal	64		Lognormal	0.7	0.3	0	0	0	0	0
	Other Exterior	Normal	64		Lognormal	1	0.5	0	0	0	0	0

Notes: [1]% are given by mass of total cementitious materials, [2]taken as 77 for hot marine environments
Dist. = Distribution, SD = Standard Deviation

3.3.6 Carbon-Induced Corrosion According to fib Bulletin 34

fib Bulletin 34 also considers a full probabilistic approach for modeling carbonation induced corrosion of concrete structure. The depth of carbonation induced corrosion (x_c) during service life t can be determined using equation (23)

$$x_c(t) = W \times \sqrt{2k_e k_c \left(k_t R_{AAC,0}^{-1} + \varepsilon_t\right) C_s} \times \sqrt{t} \tag{23}$$

where; $R_{AAC,0}^{-1}$ is the inverse effective carbonation resistance of concrete, C_s is the CO_2 concentration of the ambient air, ε_t is an error term, k_t is the regression parameter, k_e is the environmental function, k_c is the execution transfer parameter, W is the time-dependent weather function. fib Bulletin 34 uses analytical equations to determine the environmental and weather function. For example, k_e is dependent on relative humidity (RH)

$$k_e = \left(\frac{1 - \left(\frac{RH_{real}}{100}\right)^{f_e}}{1 - \left(\frac{RH_{ref}}{100}\right)^{f_e}} \right)^{g_e} \tag{24}$$

In equation. (24), f_e and g_e are exponents. The transfer function k_c can be determined using equation (25)

$$k_c = \left(\frac{t_c}{7}\right)^{b_c} \tag{25}$$

where t_c is the time of curing, and b_c is a regression parameter. The weather function W is defined according to Eq. (26)

$$W(t) = \left(\frac{t_{ref}}{t}\right)^{w/2} = \left(\frac{t_{ref}}{t}\right)^{\left(P_{SR}\frac{t_w}{365}\right)^{b_w}/2} \tag{26}$$

where w is the weather exponent, b_w exponent from regression, P_{SR} probability of driving rain, and t_w is the time of wetness.

As mentioned previously, many service models do not consider carbon-induced corrosion. Between chloride and carbon induced corrosion, design requirements for chloride induced corrosion will always govern (Bentz and Thomas, 2001; Pease et al., 2019).

3.4 Validation and Residual Service Life of Existing Structures

Typically, service life modeling is done during the design and construction phases. However, it is equally important to regularly perform the testing of in-service structures to validate the model prediction and monitor the deterioration process. Samples should be taken from the existing structure to determine surface chloride concentration and apparent diffusion coefficient, i.e., environmental load and material resistance. These quantities can be used to predict the residual service life of the existing structures. ASTM C823 can be used to determine the number of samples required to be considered statistically significant for a particular component in a hardened concrete structure (ASTM C823, 2012).

Table 3-7 shows the parameters, associated test standards, and recommended frequency intervals required to validate the service life model prediction. The recommendation on testing frequency by FM 5-516 test is obtained from Life-365 and SHRP2.

Table 3-7: Testing for Service Life Model Validation and Condition Monitoring

Parameter	Specification	Title	Frequency		Reference
			Life-365	SHRP2 (*fib* bulletin 34 based)	
Surface Chloride Concentration	ASTM C1152	Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete	year 0, Year 2, then every 5 years	every 10-20 year	(Pease et al., 2019; ASTM C1152, 2020; AASHTO T-260, 2010; NT Build 208, 1996; Bentz and Thomas, 2001)
	Nord Test NT Build 208	Concrete, hardened: Chloride content by Volhard titration	year 0, Year 2, then every 5 years	every 10-20 year	(Pease et al., 2019; ASTM C1152, 2020; AASHTO T-260, 2010; NT Build 208, 1996; Bentz and Thomas, 2001)
	AASHTO T-260	Standard Method of Test for Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials	year 0, Year 2, then every 5 years	every 10-20 year	(Pease et al., 2019; ASTM C1152, 2020; AASHTO T-260, 2010; NT Build 208, 1996; Bentz and Thomas, 2001)
	Florida Method FM 5-516	Determining Low-Levels of Chloride in Concrete and Raw Materials	year 0, Year 2, then every 5 years	every 10-20 year	(Pease et al., 2019; FM 5-516, 2023; Bentz and Thomas, 2001)
Diffusion Coefficient	ASTM C1556	Standard Test Method for Determining the Apparent Chloride Diffusion Coefficient of Cementitious Mixtures by Bulk Diffusion	year 0, Year 1, then every 5 years	every 10-20 year	(ASTM C1556, 2022; NT Build 492, 1999; Pease et al., 2019; Bentz and Thomas, 2001)
	Nord Test NT Build 492	Concrete, mortar and cement-based repair materials: Chloride migration coefficient from non-steady-state migration experiments	year 0, Year 1, then every 5 years	every 10-20 year	(ASTM C1556, 2022; NT Build 492, 1999; Pease et al., 2019; Bentz and Thomas, 2001)

3.5 Service Life Modeling of Repaired Structures

Service life prediction of repaired concrete structures is not straightforward as newly built or unrepaired structures (Taffesea and Sistonen, 2013). Figure 3-5 shows that mass transport of corroding agents between the existing and repaired concrete and reinforcement steel, geometry, and environmental conditions affects the service life prediction of repaired structures.

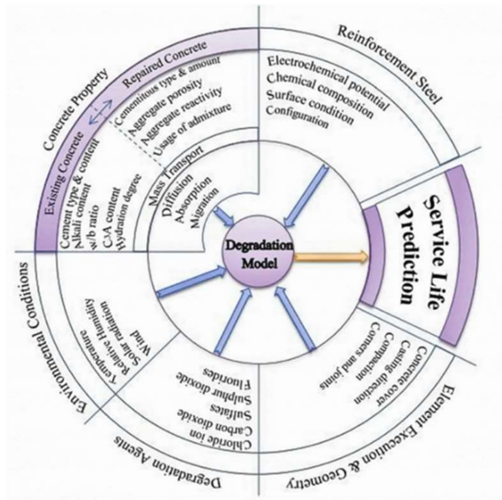


Figure 3-5: Factors affecting the service life of repaired structures (source: Taffesea and Sistonen, 2013)

To determine the service life extension, it is important to perform a service life modeling of repaired concrete structures. However, the commonly available models, such as Life-365 or fib Bulletin 34, cannot accurately estimate the service life extension of the repaired concrete structures. In fact, there exist very few numbers of service life prediction models that can be used for repaired structures (Taffesea and Sistonen, 2013; Song et al., 2009; Dao et al., 2010). In Life-365 and fib Bulletin 34, one-dimensional chloride ion diffusion from surface to rebar is a space-dependent problem. However, according to Song et al., in repaired concrete structures, chloride ion diffusion is both a space and time-dependent problem (Song et al., 2009). Figure 3-6 (a) shows that immediately after the repair, there is no chloride ion present in the repaired material. However, chloride concentration is present at the surface and in the old concrete. With time, chloride ion penetrates from the surface to the new material. Also, redistribution of chloride ions occurs from the old material to the newly repaired material. Thus, the chloride ion distribution becomes a space-dependent problem due to the differences in diffusion coefficients, and a time-dependent problem due to the difference in the aging of the materials. The more the number of repairs, the more complicated it becomes.

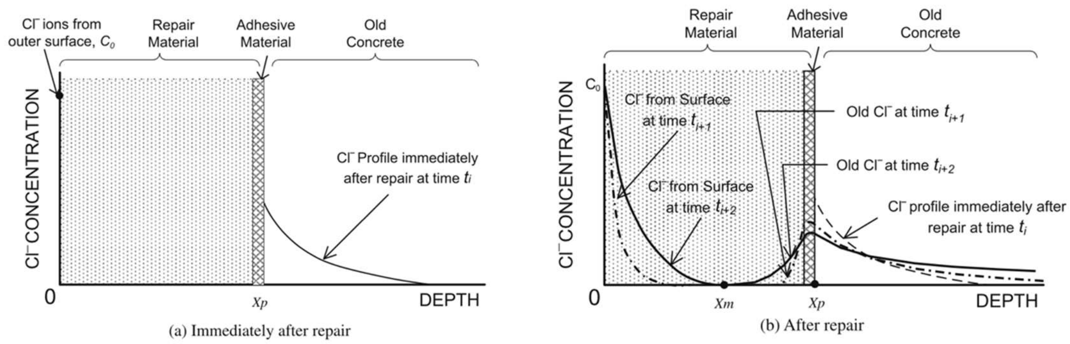


Figure 3-6: Penetration and redistribution of the chloride ions into the repaired material (source: Song et al., 2009)

Song et al. (2009) solved the problem of chloride ion diffusion through materials of different diffusion coefficients using the finite difference method, as shown in Eq. (27)

$$\frac{c_{i,j+1}-c_{i,j}}{\Delta t}=\frac{1}{2}\left(\frac{\left[D_{i+\frac{1}{2}}(c_{i+1}-c_i)-D_{i-\frac{1}{2}}(c_i-c_{i-1})\right]_{j+1}}{(\Delta x)^2}+\frac{\left[D_{i+\frac{1}{2}}(c_{i+1}-c_i)-D_{i-\frac{1}{2}}(c_i-c_{i-1})\right]_{j}}{(\Delta x)^2}\right) \tag{27}$$

here, D and c are the diffusion coefficients and chloride concentration.

Figure 3-7 shows the effect of properties of repaired material on the service life extension. For example, the critical chloride threshold for this particular structure was 1.2. The structure reaches the critical chloride threshold 25 years after construction without any repair work. Based on the developed service life model, Song et al. demonstrated that the service life of the existing structure can be extended to 100 years with two or four repairs with materials having water cement ratio of 0.3 and 0.4 respectively (Song et al., 2009).

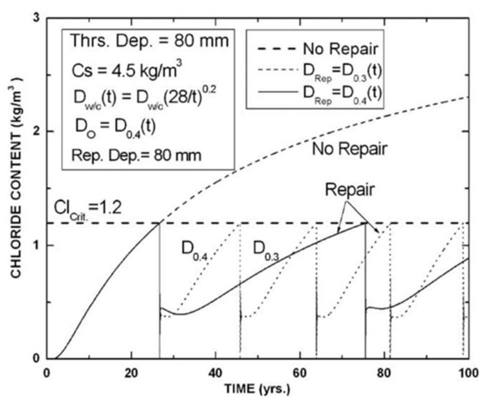


Figure 3-7: Service life extension due to repair (source: Song et al., 2009)

Although promising, the above model developed by Song et al. cannot be used, as it is not publicly available, and the development of any model is out of the scope of this project.

Life-365 and fib models cannot accurately provide the service life estimation of the repaired structure as these models were not developed to consider the variation between the diffusion coefficient and aging of existing and repaired concrete.

3.6 Limitations

One of the limitations of concrete corrosion models is their inability to consider the presence of cracks in-service life estimation. Life-365 and fib Bulletin 34 are developed based on the assumption of uncracked concrete. However, fib developers recognize it as an important factor and suggest crack width should be kept below a certain threshold to ensure the longer service life of the structures (Schiessl, et al., 2006). ISO 16204:2012 specification on service life design of concrete structures recommends that the width of the crack between eight mils to16 mils (0.2mm–0.4mm) do not affect the reinforcement corrosion under normal conditions (ISO-16204, 2012). Although significant variations are observed in the literature regarding the effect of cracks on corrosion initiation, it is widely accepted that the frequency and width of cracks affect chloride diffusivity (Matthews, 2014). It is particularly true for the concrete mixes with low diffusion coefficients, where a crack width of 0.2 mm can increase the diffusion coefficient by more than 15 times (Djerbi et al., 2008). More research is needed on this topic-incorporation of cracks (before or after repair) in service life prediction models. One solution can be the introduction of a factor to alter the diffusion coefficient based on the severity of the cracks. At this stage, AASHTO emphasizes following appropriate design standards and local construction experiences to reduce cracks in concrete (Murphy, et al., 2020).

In addition, similar to Life-365, fib Bulletin 34 also does not consider chloride ingress in concrete due to capillary action. The fib calculated reliability also does not have any meaning in terms of the distribution of corrosion. The spatial distribution of the material properties should be reviewed to define a meaningful correlation between reliability and the distribution of corrosion. fib also does not have any provision to consider the effect of admixture or concrete properties directly during the analysis phase. However, the user can modify the input parameters prior to the analysis to consider their effect. It is advised that users should validate and update the suggested values based on the local conditions.

Despite their limitations, Life-365 and fib Bulletin 34 are mechanistic models that consider many important factors and their interactions in service life prediction. Furthermore, they are widely available and easy to use. Thus, these two softwares will be further reviewed in the next chapter.

4 SENSITIVITY ANALYSIS

In this chapter, a sensitivity analysis was performed using both the models to identify the important input parameters. Due to the inherent differences between Life-365 and fib methods, i.e., deterministic versus probabilistic, the sensitivity analysis performed is slightly different.

4.1 Life-365

To perform a sensitivity analysis using Life-365, a base model must first be selected. The input for the base model is as follows:

- Location: Miami, FL
- Structural type: 30-inch circular column
- Surface chloride concentration, $C_S = 20$ lb/yd^3
- Diffusion coefficient, $D_{28} = 5.24\times10^{-13}$ m^2/s
- Cover, $c = 2$ inch
- Diffusion decay index, $m = 0.51$

The above inputs for the base model were obtained from the FDOT SMO-sponsored report on a major bridge in South Florida (Canjura, 2020; Pease B. J., 2021). Based on these studies, several parameters such as a chloride threshold value of 1.0 lb/yd^3, propagation period of 10 years, and maximum surface chloride concentration build-up time of 1 year were assumed to be constant. Only one input variable is changed with time while the others constant to identify the effect of C_s, D_{28}, c, and m.

4.1.1 Effect of Surface Chloride Concentration, C_s

The input of surface chloride concentration varied from 5 lb/yd^3 to around 23 lb/yd^3 to identify its effect on service life. Figure 4-1 shows an inverse relation of surface chloride concentration with service life while other inputs remain constant. Surface chloride concentration is the environment load variable. As the chloride concentration increases, the service life is expected to decrease. Figure 4-1 shows that the decay rate is 0.878, i.e., one unit increase in the normalized surface chloride concentration decreases the service life by 58.4% while everything else remains constant. Figure 4-1 also shows that the service life of the concrete structure is 256.6 years in the absence of any normalized surface chloride concentration with other inputs unchanged. It is important to note that this study represents the results of the sensitivity analysis in normalized units to have a meaningful comparison between the input parameters, as their units vary by several orders of magnitude. Unit normalization for the sensitivity analysis was based on the respective unit of the base model.

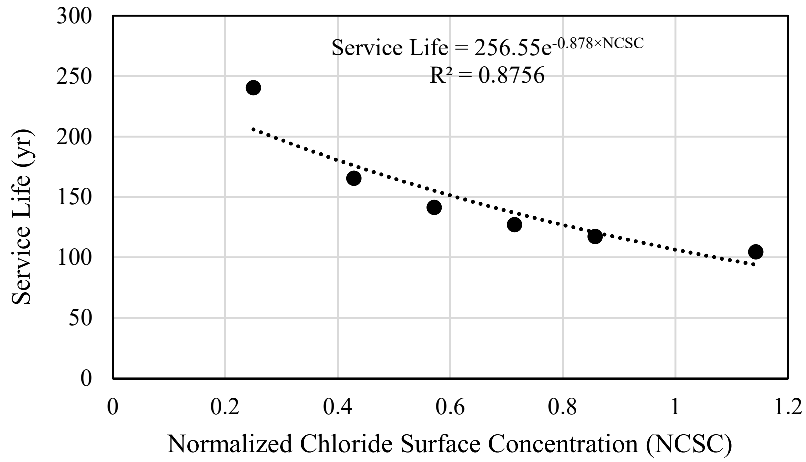


Figure 4-1: Effect of surface chloride concentration on service life

4.1.2 *Effect of Diffusion Coefficient, D_{28}*

Figure 4-2 shows the effect of diffusion coefficient on the service life of concrete structures. Like surface chloride concentration, the diffusion coefficient also has an inverse relation with service life while other inputs remain constant. With a one unit increase in normalized diffusion coefficient, service life decreases by around eight percent, while all other inputs remain constant. Figure 4-2 also shows that according to the Life-365 analysis, the service life of a concrete structure is around 60.4 years with extremely low to negligible normalized diffusion coefficient if other variables remain constant.

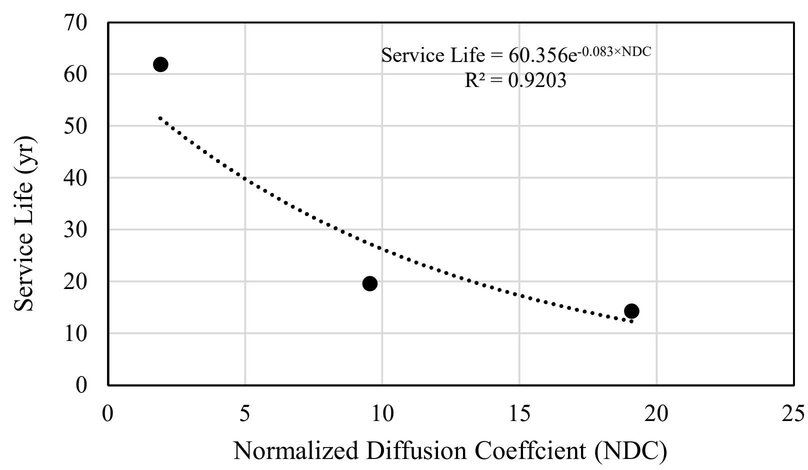


Figure 4-2: Effect of diffusion coefficient on service life

4.1.3 *Effect of Concrete Cover*

Figure 4-3 shows the effect of concrete cover on the service life of the concrete structures. It shows that a one unit increase in the concrete cover compared to the concrete cover in the base model increases the service life by almost 258.6% if other variables are constant. It also shows that with a negligible normalized cover, the service life of the concrete structure against chloride-induced corrosion is around 31 years, with other variables constant. Unlike the other two input parameters, service life has a positive relation with concrete cover, i.e., increasing cover increases service life.

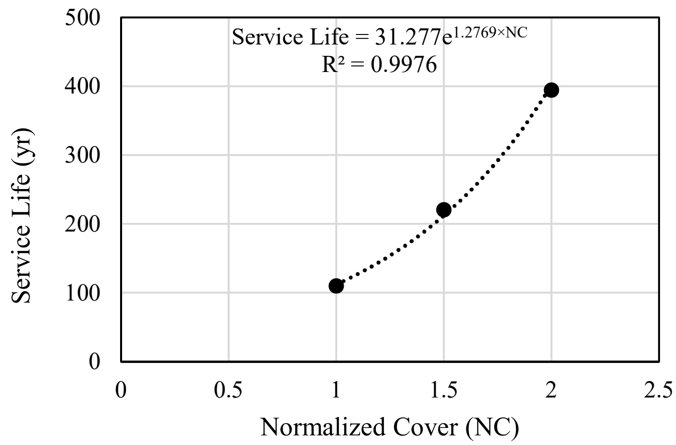


Figure 4-3: Effect of concrete cover on service life

4.1.4 Effect of Diffusion Decay Index

Based on the project literature review, one important parameter that affects the service life prediction by Life-365 is the diffusion decay index which is a function of the concrete mixture. Life-365 calculates the diffusion decay index based on the percentages of fly ash (FA) and slag cement (SG) in the mixture using Eq (28)

$$m = 0.2 + 0.4 \times \left(\frac{\%FA}{50} + \frac{\%SG}{70}\right) \tag{28}$$

The base model for the major bridge in South Florida, used 38.4% fly ash. Using Eq. (28), the diffusion decay index, m, for the base model was 0.51. For the sensitivity analysis, three values of diffusion decay index were considered: 0.4, 0.51, and 0.6, which corresponds to normalized values of 0.78, 1 and 1.18. Similar to the concrete cover, the diffusion decay index also has a positive relation with service life. Figure 4-4 shows that according to Life-365, a one unit increase in the normalized diffusion decay index increases the service life by 491.8% if other variables remain constant. In addition, when the normalized diffusion decay index is close to zero, the service life is approximately 19 years with other inputs remaining unchanged.

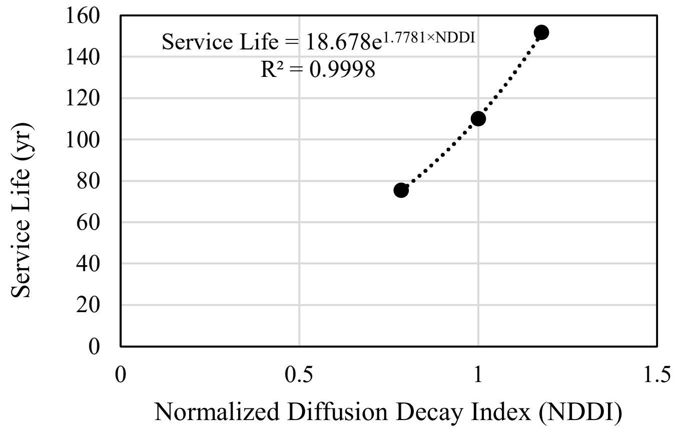
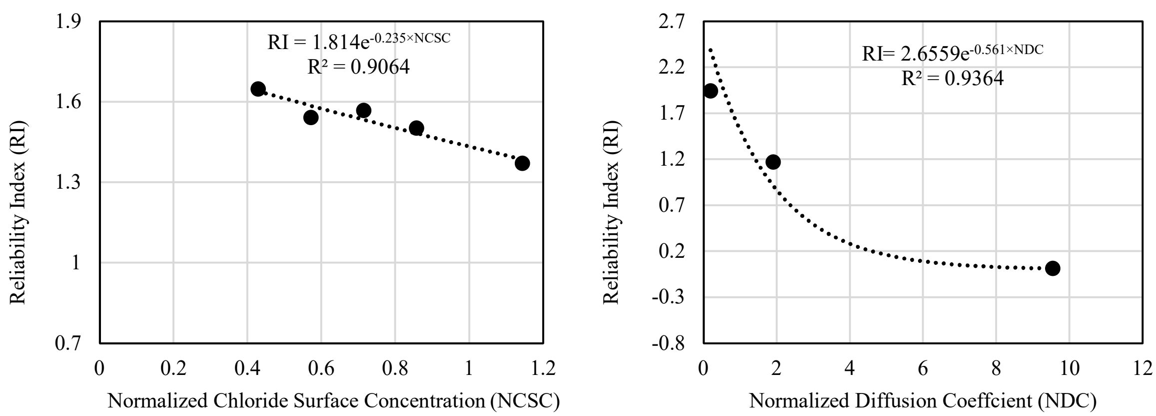


Figure 4-4: Effect of diffusion decay index on service life

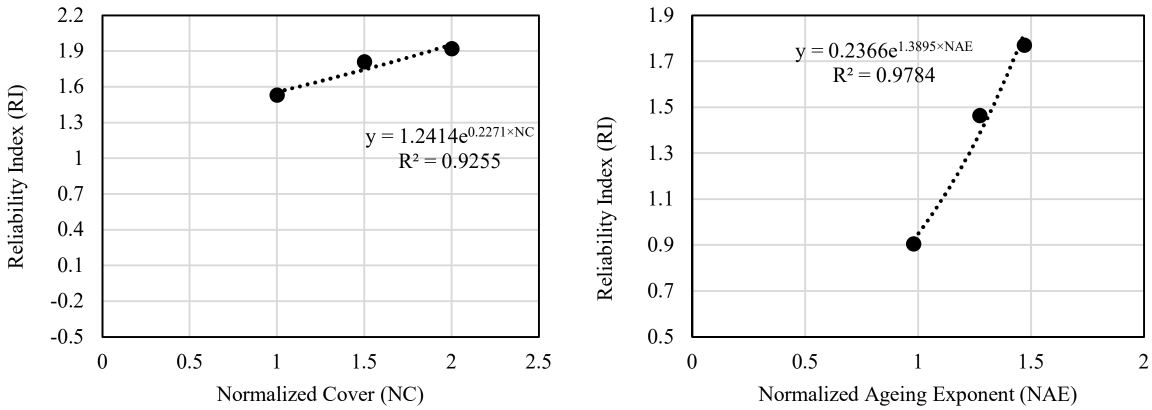
4.2 fib Bulletin 34

Similar to Life-365, a sensitivity analysis was also performed using the fib-34-Bulletin-based probabilistic method. The inputs for the base model were same as used in Life-365 sensitivity analysis. Figure 4-5 shows the effect of different variables on the reliability index while other variables are held constant. In the fib-based model, the output is the reliability index instead of service life. If the reliability index is 1.3 or higher, there is a 90% probability that the structure will survive the design life, which is typically 100 years.



(a) Surface Chloride Concentration **(b) Diffusion Coefficient**

Figure 4-5: Effect of variables on reliability index



(c) Concrete Cover **(d) Aging Exponent**

Figure 4-5: Effect of variables on reliability index

Figure 4-5 (a) shows the effect of changing surface chloride concentration on the reliability index. A unit change in the normalized surface chloride concentration reduces the reliability index by approximately 21% while other variables are held constant. Similar to the normalized surface chloride concentration, Figure 4-5 (b) shows that a unit increase in the normalized diffusion coefficient also decreases the reliability index by 42% while other variables remained constant. However, Figure 4-5(c) and Figure 4-5(d) show that a unit increase in the normalized cover and aging exponent increases the reliability index by 25% and 301%, respectively, while other variables are kept constant.

4.2.1 Design of Experiments (DOE)

The two-sensitivity analyses shown previously using the Life-365 and *fib*-Bulletin-34-based models provide an indication of how these input parameters individually affect the service life of concrete structures due to chloride corrosion. The results show the effect of individual parameters while keeping others constant. This is called One Factor at a Time (OFAT) design which does not show the combined effect of varying multiple parameters. To identify significant parameters and the effect of their respective interaction with each other, it is important to perform a full factorial design of experiment (DOE) analysis. The challenge of performing a DOE analysis with Life-365 is its deterministic output. It is not possible to produce replicable samples by varying inputs using a full factorial design with Life-365. However, the fib-Bulletin-34-based probabilistic model does not have such limitations.

In this study, a full factorial DOE was performed at two levels and five factors. Table 4-1 shows the factors and levels considered for these experiments. DOE is a field of applied statistics that involves planning, performing, and analyzing the results of controlled tests to identify the critical factors that affects the output or response variable.

Table 4-1: Variables and Levels of Considered in DOE

Levels	Variable				
	Surface Chloride Concentration (lb/yd^3)	Diffusion Coefficient, D_{28} ($\times10^{-10}$ in^2/s)	Concrete Cover (inch)	Aging Exponent (α)	Mean Temperature (°F)
-1	6	8.12	2	0.3	67.2
+1	20	124	4	0.65	78.9

The two levels represent the high and low values of each factor. These high- and low-value variations can represent region-specific or material-specific variability. For example, variations in surface chloride concentration, concrete cover, and temperature can occur due to location. In contrast, variations in the aging exponent and diffusion coefficient occur due to differences in materials. It is important to note that these values are chosen to demonstrate maximum reasonable contrast between the high and low levels within the state of Florida. If a factor is not identified as significant for maximum contrast values, it would not be considered important for other interim values. Additional details on how the lower- and upper-level values of each variable in Table 4-1 were selected are given below:

- Mean temperature in Table 4-1 is based on the temperatures from Key West, FL, and Lake City, FL, which represents two opposite environmental conditions in the state.
- Values for surface chloride concentration were obtained from the previous studies conducted by FDOT and NCDOT (Canjura, 2020; Gergely et al., 2006). For inland bridges in North Carolina, the average surface chloride concentration was approximately 6 lb/yd^3. For the coastal region, the value was similar to FDOT's adopted value of 20 lb/yd^3. As surface chloride concentrations in the inland bridges were not available, the value from North Carolina was used as the lower-level value.
- The high and low values of the diffusion coefficient were obtained from FDOT SMO-sponsored studies on a major bridge in South Florida (Canjura, 2020; Pease B. J., 2021).
- The upper and lower values of cover in Table 4-1 were obtained from the FDOT structures design guidelines (FDOT, 2016).
- fib Bulletin 34 and the fib-based SHRP2 model varies the aging exponent between 0.3 and 0.65 based on the concrete type and exposure condition. Thus, this study considers these two values to be the aging exponent's high- and low-level values.

Each analysis was performed twice to account for the variations. Thus, a total of 64 simulations were conducted using fib Bulletin 34-based probabilistic model. The inputs of the factor variables for these 64 simulations, along with the output, i.e., reliability index, are presented in Appendix A.

4.2.2 Statistical Analysis

4.2.2.1 Boxplot of Each Variable by Level

Determination of important input parameters and their interactions using DOE and subsequent statistical analysis were performed using *R* (Lawson, 2014). First, the high and low values of each factor shown in Table 4-1 were represented as coded factors of -1 and +1.

Figure 4-6 shows the boxplots of each variable by level. If any variable did not show variation based on high (+1) and low (-1) level values, then that variable would be removed from the analysis. Almost all the variables show response variations with level. However, most variations in the mean value were observed for diffusion coefficients and aging exponent.

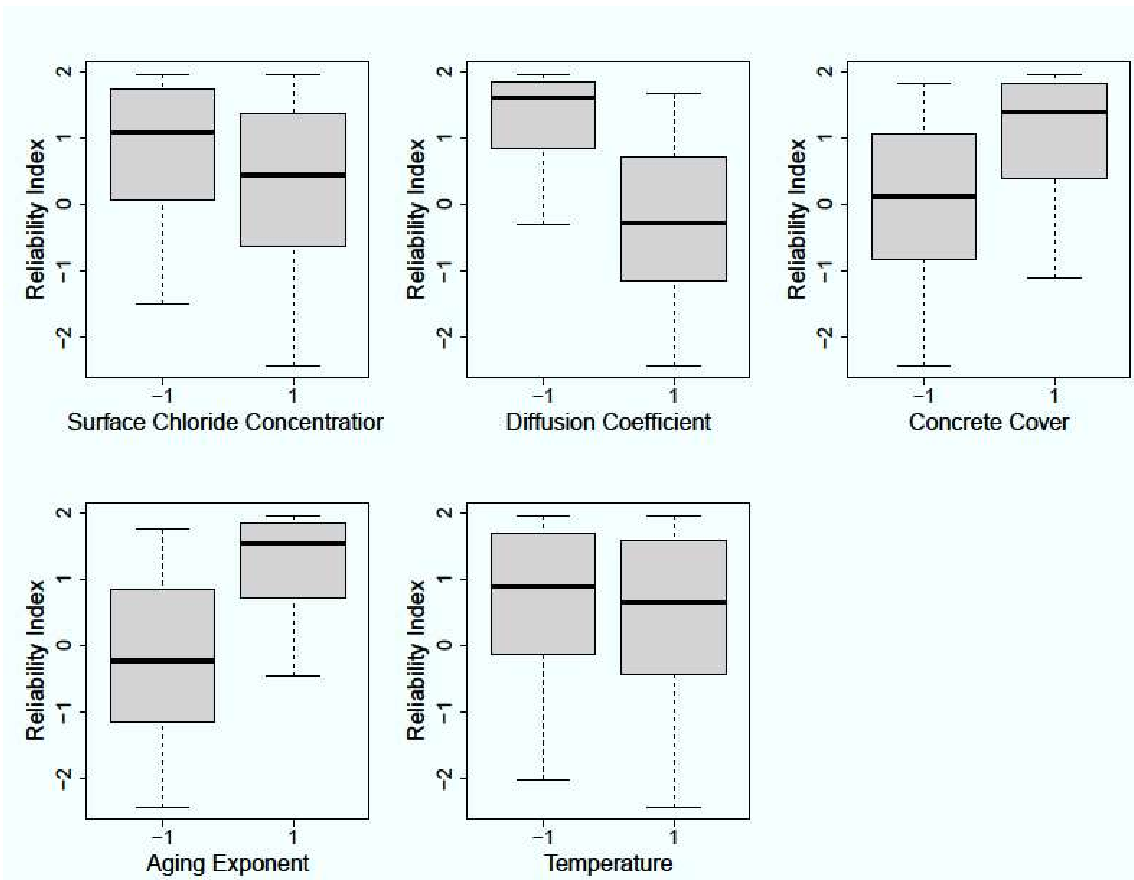


Figure 4-6: Box-plot of factor variables

4.2.2.2 Fit Statistical Models

A total of three Analysis of Variance (ANOVA) models were fitted to the DOE data to determine the significant parameters. First, ANOVA model was fitted with all the terms. It had a total of 32 terms, which included all the main factors and their possible interactions. The main factors are surface chloride concentration, diffusion coefficient, concrete cover, aging exponent, and temperature. Interactions are the combinations of main factors, which can be two-, three-, four-, or five-way. Examples of such interactions are as follows:

- Two-way Interaction: *diffusion coefficient* × *temperature*
- Three-way Interaction: *surface concentration* × *diffusion coefficient* × *temperature*
- Four-way Interaction: *concentration* × *diffusion* × *cover* × *temperature*

All five main factors were found to be significant in the full model with a 95% confidence interval (p-value < 0.05). However, 11 higher-level interaction terms were found to be insignificant. The complete model with the significance level (p-value) of each term is presented in Appendix B.

After that, a second ANOVA model was developed with the significant 21 terms. Figure 4-7 shows the diagnostic plot of the reduced model. These diagnostic plots are important to check the model assumptions. For this study, two assumptions are particularly important:

- Normality of residuals, i.e., errors are normally distributed.
- Random distribution of the residuals, i.e., errors are randomly distributed around zero.

If the residuals are normally distributed, they would follow a straight line in the Normal Q-Q plot, and the histogram of the residuals would follow a normal distribution. The Normal Q-Q plot in Figure 4-7 shows that most points follow the straight red line. However, at both tails or ends of the Normal Q-Q plot, some points highlighted in blue deviate from the straight line. Deviations of these points from the straight line in the Normal Q-Q plot in Figure 4-7 demonstrate a slight violation of the normality assumption. The boxplot also shows the presence of outliers. The outliers are also highlighted with blue circles. In addition, the residual vs. predicted reliability index plot shows that the residuals are more spread with a higher reliability index. In ideal conditions, the residuals should be distributed randomly, i.e., there should be no trend. However, as mentioned earlier, a trend can be observed for this model, and it is shown in a blue dashed line in the residual plot. The presence of this trend indicates a violation of the random distribution of the residuals assumption.

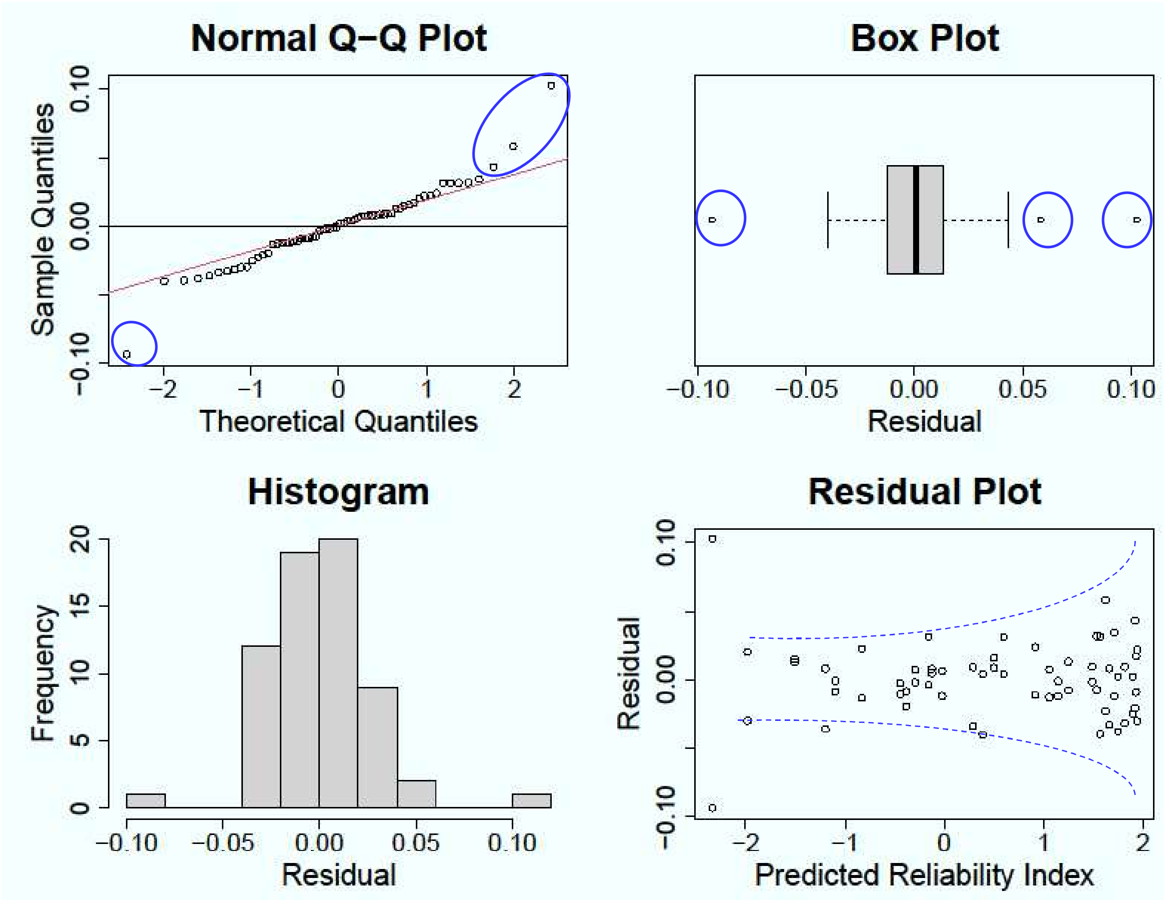


Figure 4-7: Diagnostic plot of the reduced model

4.2.2.3 Data Transformation

Next, the response variable, i.e., the reliability index, was transformed to meet the model assumptions. The reliability index can be either positive or negative based on the probability of failure for a particular set of input parameters. Therefore, this study adopted the Yeo-Johnson power transformation, which can accommodate both positive and negative values of the response variable (Weisberg, 2001). Previously obtained, the 21-term model was fitted with the newly transformed reliability index, and seven interaction terms were found to be non-significant. Removing the non-significant interaction terms leads to a transformed, reduced model with 14 terms, which is also shown in Appendix B. Figure 4-8 does not appear to show any serious violations of model assumptions with the transformed variable. The Q-Q plot and histogram show that residuals are normally distributed, the boxplot shows only a single outlier, and the residual plot shows that transformation has resolved the issue with increasing variance with an increased reliability index.

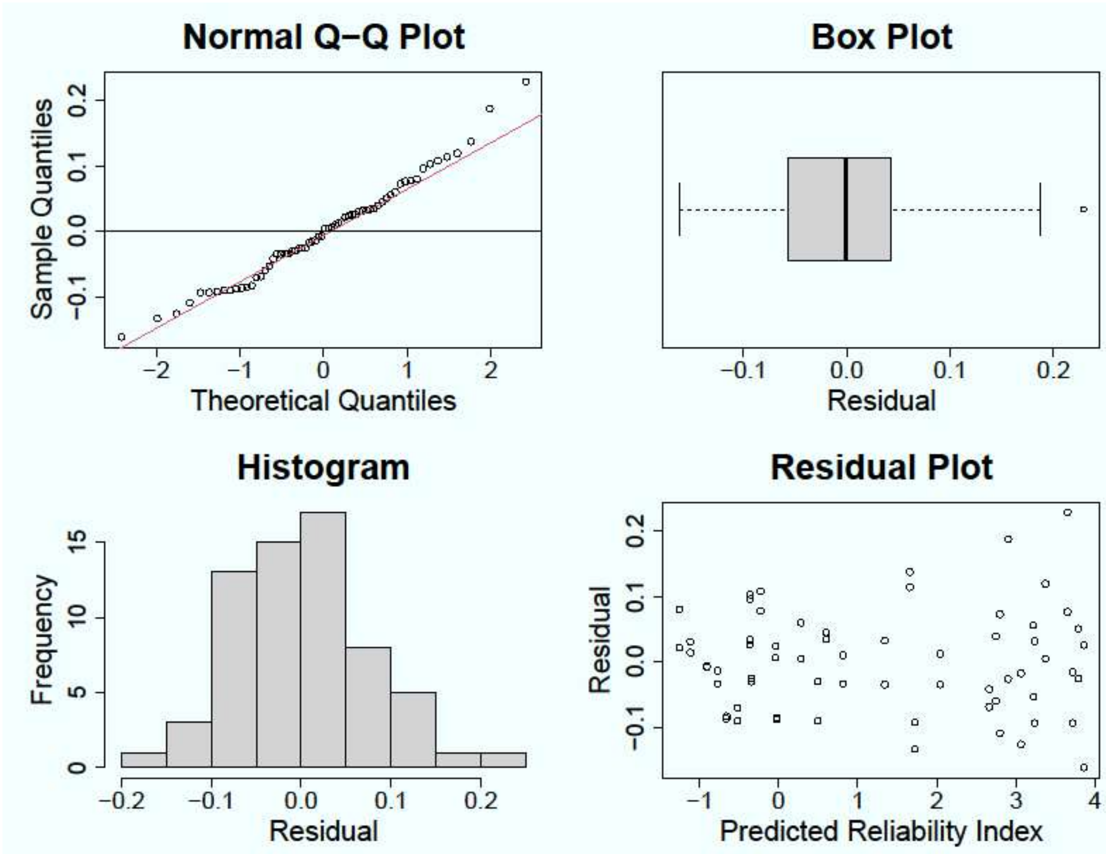
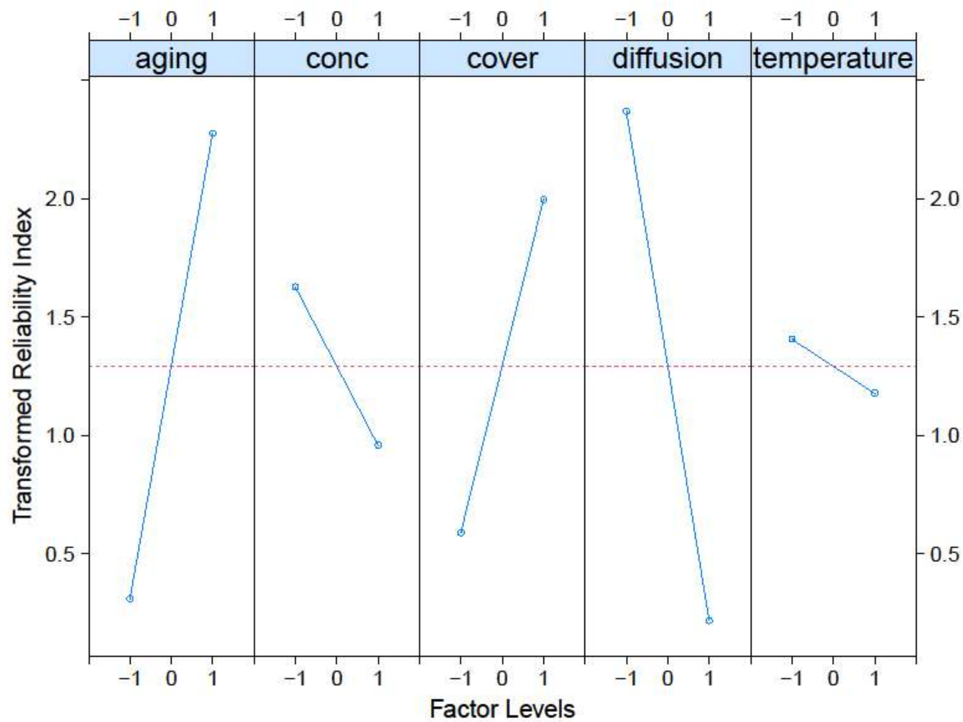


Figure 4-8: Diagnostic plot of the transformed reduced model

4.2.3 Identify Important Parameters

The newly transformed reliability index was used to identify the important variables because it did not violate model assumptions. The main effect magnitude plot is shown in Figure 4-9. The steepness and magnitude change from the low level to the high level helps to identify the important parameters. The steeper and higher the magnitude change from one level to the other for a particular variable, the more important that variable is. Based on the DOE and statistical analysis, Figure 4-9 shows that the diffusion coefficient is the most important factor, followed by the aging exponent, concrete cover, and surface chloride concentration. Theoretically, temperature is also an important factor; however, its main effect was the least significant among the variables considered.



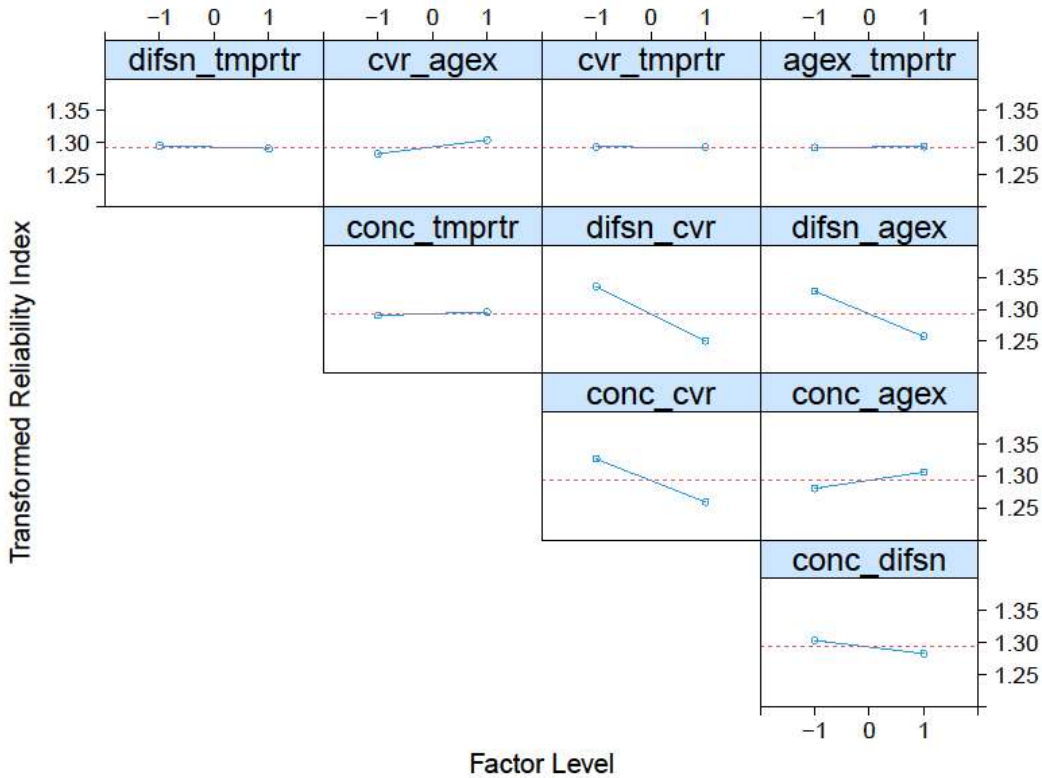
diffusion= diffusion coefficient, conc= surface chloride concentration, aging = aging exponent

Figure 4-9: Main effect plot

Figure 4-10 shows the interaction between the main variables. Based on the magnitude and steepness the following interactions were observed to be significant:

- Diffusion coefficient × concrete cover
- Diffusion coefficient × aging exponent
- Surface chloride concentration × concrete cover

Other interactions were not found to be significant, and temperature was not found to have significant interaction with any other variables.



difsn_tmprtr = diffusion coefficient × temperature, cvr_agex = cover × aging exponent, conc_tmprtr = surface chloride concentration × temperature, difsn_cvr = diffusion coefficient × cover, difsn_agex = diffusion coefficient × aging exponent, cvr_tmprtr = cover × temperature, agex_tmprtr = aging exponent × temperature, conc_cvr = surface chloride concentration × cover, conc_agex = surface chloride concentration × aging exponent, conc_difsn = surface chloride concentration × diffusion coefficient

Figure 4-10: Interaction effect plot

According to the main effect and interaction plots, the following configuration shown in Table 4-2 should maximize the longevity of the concrete structures against chloride induced corrosion:

Table 4-2: Configuration for Maximum Service Life Against Chloride-Induced Concrete Corrosion

Factor	Surface Chloride Concentration	Diffusion Coefficient	Concrete Cover	Aging Exponent	Temperature	Surface Chloride Concentration × Concrete Cover	Diffusion Coefficient × Aging Exponent	Diffusion Coefficient × Concrete Cover
Level	-1	-1	+1	+1	-1	-1	-1	-1

Based on the configurations in Table 4-2 and the values shown in Figure 4-1, the highest durability of the concrete structures against chloride-induced corrosion will be obtained when surface chloride concentration, diffusion coefficient, and temperature are at the lowest level, i.e., 6 lb/yd^3, and 8.12×10^{-10} in^2/s, and 67.2°F, respectively; and concrete cover, and aging exponent are at the highest level, i.e., 4-inch and 0.65. As mentioned earlier, the rationale for performing the DOE and statistical analysis was to identify the important parameters and their order.

However, the outcome of the statistical analysis also shows that it matches the common understanding, which, in a sense, verifies the analysis procedure.

5 SPECIFIC INPUT PARAMETERS

Table 5-1 highlights the interaction among the input parameters based on Figure 4-9 and Figure 4-10 to identify the order of or important parameters. Both diffusion coefficient and cover have the three High interactions. However, magnitude of diffusion coefficient in Figure 4-9 is higher compared to cover. Temperature variation due to different regions of Florida was found to be of lowest significance as it does not have any significant interaction with other variables, and its main effect is also lowest compared to others.

Table 5-1: Interaction of Input Parameters

Factor	Diffusion Coefficient	Surface Concentration	Cover	Aging Exponent	Temperature
Diffusion Coefficient	Highest	Low	High	High	Low
Surface Concentration	Low	High	High	Low	Low
Cover	High	High	Higher	Low	Low
Aging Exponent	High	Low	Low	Higher	Low
Temperature	Low	Low	Low	Low	Low

Based on the above discussion and data, the order of important input parameters from highest to lowest impact are as follows:

1. Diffusion coefficient
2. Concrete cover
3. Aging exponent
4. Surface chloride concentration
5. Temperature

5.1 Material Specific Inputs

Region or material specific inputs for the parameters with higher impact or significance will yield an improved prediction of the service life of the concrete structures. For diffusion coefficient and aging exponent, it is important to have material-specific values (Pease et al., 2019; Schiessl et al., 2006). For example, Virginia Department of Transportation (VDOT) adopted diffusion coefficient values based on the presence of fly ash and slag with Portland Cement Concrete (PCC). *fib* recommends using Nord Test (NT) Build 492 to determine the diffusion coefficient (NT Build 492, 1999). In the absence of test data, *fib* recommends using Table 5-2.

Table 5-2: Quantification of Diffusion Coefficient Based on Concrete Mixtures (source: Schiessl, et al., 2006)

Cement Type	Diffusion Coefficient $\times 10^{-8}$ (in^2/s)					
	w/c Ratio = 0.35	w/c Ratio = 0.4	w/c Ratio = 0.45	w/c Ratio = 0.5	w/c Ratio = 0.55	w/c Ratio = 0.6
CEM I 42.5 R	NA	1.38	1.55	2.45	3.05	3.88
CEM I 42.5 R + Fly Ash	NA	0.868	1.07	1.40	1.69	2.31
CEM I 42.5 R + Silica Fume	0.682	0.744	NA	NA	0.822	NA
CEM III/B 42.5	NA	0.217	0.295	0.434	0.465	0.527

NA = Chloride migration coefficient is not available for the concrete mixtures

The fib model determined the aging exponent based on the statistical analysis of the diffusion coefficient in structures with different concrete types. The statistical analysis was conducted using the data available in the published literature. The recommended aging exponent according to fib is shown in Table 5-3. The quantification of the aging exponent based on concrete mixtures shown in Table 5-3 was adopted by SHRP 2 and VDOT (Pease et al., 2019).

Table 5-3: Statistical Quantification of Aging Exponent Based on Concrete Mixtures (source: Schiessl, et al., 2006)

Concrete Type	Aging Exponent		
	Distribution	Mean Value	Standard Deviation
Portland Cement Concrete (CEM I)	Beta	0.3	0.12
Portland Cement Concrete + Fly ash ≥ 20%	Beta	0.6	0.15
Blast furnace slag cement concrete (CEM III/B)	Beta	0.45	0.2

5.2 Region Specific Inputs

Region-specific values will be more appropriate for concrete cover and surface chloride concentration as both their main effect and interaction were found to be significant. FDOT already provides concrete cover based on regions (FDOT, 2016). FDOT currently has two regions: extremely aggressive and moderately aggressive environments. These regions were obtained based on the chloride intrusion rate and not surface chloride concentration. Both Life-365 and fib-Bulletin-34-based models require surface chloride concentration as an input. Previous studies have shown that surface chloride concentration can vary by topography and distance from the shoreline (Gergely et al., 2006; Bales et al., 2018). FDOT uses a surface chloride concentration of 20 lb/yd^3 for design purposes. FDOT should determine the surface chloride concentration of the structures in the inland regions of Florida. Based on the findings, FDOT can adopt the following recommendation:

- If the surface chloride concentration is at or below 6 lb/yd^3, FDOT should perform a corrosion durability study using field-obtained surface chloride concentration before adopting region-specific values.

5.3 Recommended Model to Adopt

FDOT is using Life-365 to predict the service life of corrosion-induced concrete structures. It is free, widely accessible, and easy to use. The primary shortcoming of Life-365 is its deterministic nature. It is possible to perform uncertainty analysis with Life-365; however, that is limited to only certain user-defined conditions (Canjura, 2020). In addition, it is time-consuming and not straightforward to perform such uncertainty analysis in Life-365. In comparison the uncertainty of the corrosion-induced service life prediction of concrete structures is done inherently in fib-Bulletin-34-based model. The Microsoft Excel-based fib-Bulletin-34-based model is also widely available through the SHRP2 program (Pease et al., 2019). Like VDOT and SHRP2, FDOT can also develop its own MS Excel spreadsheet to determine the service life model of corrosion-induced structures based on the material and/or regional inputs specific to the State of Florida. However, the development of such a program is beyond this project's scope. As shown previously it is straightforward to identify the critical parameters and perform additional statistical analysis based on the fib-based model. Thus, the authors recommend adoption of fib-Bulletin-34-based model for service life assessment of corrosion-induced concrete structures.

6 SUGGESTED INPUTS AND RECOMMENDATIONS

FDOT needs to provide appropriate inputs to the fib Bulletin 34 service life model to predict the durability of the concrete structures against the chloride-induced corrosion. The suggested values of the input parameters along with the associated recommendations are as follow

6.1 Surface Chloride Concentration

According to fib Bulletin 34, surface chloride concentration (C_s) depends on material properties and geometrical and environmental conditions (Pease et al., 2019; Schiessl et al., 2006). Paredes et al. (2010) studied the surface chloride concentration on concrete mixes in Florida (Paredes et al., 2010). They considered concrete mixes with a w/cm ratio of 0.41 and different amount of fly ash and silica fume. After curing, the samples were left near a coast in Florida to simulate the marine environmental condition. Surface chloride concentration was measured after 6 and 10 years by taking cores from the field-exposed samples.

Table 6-1 shows that surface chloride concentration and diffusion coefficients vary with time, even for same type of mixtures. For example, the average value of C_s after six and 10 years were 23.3 lb/yd^3 and 9.68 lb/yd^3 respectively.

Table 6-1: Surface Chloride Concentration and Diffusion Coefficients of Field Samples (source: Paredes et al., 2010)

Mix ID	Surface Chloride Concentration (C_s) (lb/yd^3)		Diffusion Coefficient (D) ×10^{-12} (m^2/s)	
	6 years	10 Years	6 years	10 Years
CTRL-C1	18.202	7.138	3.406	33.68
REO-C1	14.354	7.192	4.336	15.18
CTRL-G1	12.823	8.57	3.206	7.64
CTRL-P1	36.027	10.781	0.2158	0.1938
CTRL-P2	35.173	14.694	0.7565	0.3196
Mean	23.3158	9.675	2.38406	11.4027
Std Dev	10.1856	2.83677	1.60501	12.4353

Another study from Florida also showed variation of C_s after exposure near the intercoastal water at different time frames (Shill, 2014). Similar to Table 6-1, Table 6-2 also shows variation of surface chloride concentration with time. During the timeframe considered, the average surface chloride concentration varied between 4 and10 lb/yd^3.

Table 6-2: Surface Chloride Concentration of Field Samples (source: Shill, 2014)

Mix ID	Surface Chloride Concentration (C_s) (lb/yd^3)				
	6 months	10 months	12 months	18 months	24 months
DC1a	3.57	9.22	19.82	5.24	6.49
DC2a	5.29	4.79	4.35	7.80	6.49
DC3a	3.02	3.19	10.99	4.11	4.94
DC4a	4.48	13.92	8.39	5.02	6.66
DC5a	3.40	4.82	11.18	5.63	6.08
DC6a	8.29	5.07	12.32	7.89	6.91
DC7a	3.29	5.68	14.66	4.28	6.08
DC8a	3.86	2.85	12.20	4.26	6.62
DC9a	2.12	2.06	3.47	3.57	2.02
DC10a	5.41	2.46	3.61	5.14	4.90
DC10aa	3.89	3.08	10.28	3.83	5.36
DC10ba	3.22	2.90	10.85	3.89	6.81
DC11a	3.15	2.21	10.94	2.04	3.39
Mean	4.08	4.79	10.24	4.82	5.60
Std Dev	1.50	3.24	4.38	1.56	1.42

These surface chloride concentration values obtained from the studies performed in Florida shows a wide variation. Between Table 6-1 and Table 6-2, the average value of C_s varies between four to 23.5 lb/yd^3, and the standard deviation varies between 1.5 to 10 lb/yd^3.

6.1.1 Recommendation

Currently, FDOT is using a surface chloride concentration of 20 lb/yd^3, especially in “extremely aggressive” environments (FDOT, 2016; Canjura, 2020). Using a single value of 20 lb/yd^3 for design may or may not be conservative. Without a field durability study, it is not ideal to recommend any changes to the current state of practice that will account for both the moderate and extremely aggressive environment classification adopted by FDOT (FDOT, 2016).
In the absence of experimental data, FDOT may use the 40 lb/yd^3 and 5 lb/yd^3 as the mean and standard deviation. Using such values will cover the maximum and minimum range of surface chloride concentration observed in Table 6-1 and Table 6-2. The SHRP2 recommended log-normal distribution to be used with fib-Bulletin-34-based (Pease et al., 2019).

However, it is strongly recommended that FDOT perform a field durability study to verify and update the surface chloride concentration using either Nord Test NT 208, ASTM C 1152, or AASHTO T206 under different exposure conditions for different structural components (Pease et al., 2019; ASTM C1152, 2020; AASHTO T-260, 2010; NT Build 208, 1996).

6.2 Temperature

Table 6-3 shows the average of maximum, minimum and mean temperatures along with the standard deviations for different locations in Florida. For each location, temperature data between 1980 to 2022 were obtained from the Florida Climate Center, office of the state climatologist, hosted by Florida State University. Based on the analysis, temperature variations for Florida were found to be least significant. If a single value has to be adopted for the entire state of Florida, it is recommended to use a mean temperature of 77°F and a standard deviation of 10.3°F with fib Bulletin 34. This is the temperature of Miami. The combined average of the mean temperatures shown in Table 6-3 is around 72.4°F. Adopting this value will underestimate the temperature observed in the southern part of the state, like Miami, Key West etc.

Table 6-3: Average Temperatures of Different Locations in Florida Between 1980 to 2022

Location	Temperature (°F)			Standard Deviation (°F)		
	Max	Min	Mean	Max	Min	Mean
Apalachicola	78.3	59.7	69.0	11.3	13.7	12.1
Daytona Beach	80.4	61.8	71.0	12.1	13.3	12.5
Everglades	84.2	65.4	74.8	6.9	9.6	7.8
Ft Lauderdale	83.6	69.6	76.6	6.3	8.7	7.1
Ft Meyers	84.9	66.1	75.5	7.8	9.4	8.3
Tampa	80.4	58.0	69.2	13.1	14.5	13.4
Jacksonville	79.2	57.8	68.5	14.9	15.8	15.0
Key West	82.7	73.2	78.0	10.1	10.3	10.1
Kissimmee	83.6	62.4	73.0	8.7	11.2	9.6
La Belle	85.0	62.1	73.6	8.1	10.4	8.9
Lake City	79.5	57.7	68.6	11.7	13.1	12.0
Live Oak	82.0	57.7	69.9	11.1	13.6	11.8
Miami	84.0	70.0	**77.0**	10.1	10.8	**10.3**
Monticello	78.8	55.3	67.0	12.2	14.0	12.6
Naples	84.9	64.9	74.9	7.3	9.4	8.0
Ocala	82.8	59.8	71.3	9.7	12.4	10.6
Orlando	83.2	63.2	73.2	9.3	10.7	9.7
Pensacola	77.8	60.2	69.0	12.0	13.7	12.5
St Petersburg	81.7	69.4	75.6	8.6	9.1	8.7
Tallahassee	80.2	56.7	68.4	12.2	14.9	13.0
Gainesville	82.1	64.9	73.5	11.7	12.9	12.2
West Palm Beach	83.5	68.4	76.0	6.9	9.2	7.7
Mean	82.0	62.9	72.4	10.1	11.8	10.6

6.2.1 Recommendation

The adoption of 77°F as the mean temperature and 10.3°F as the standard deviation for temperature inputs in the fib-34 model is recommended for service life prediction of the concrete structures against chloride-induced corrosion in Florida. However, as the temperature data is available, FDOT should consider using the closest local temperature data for service life prediction with fib-Bulletin-34-based model. As per recommendation of fib-34 and SHRP 2, normal distribution should be used for temperature (Pease et al., 2019; Schiessl et al., 2006).

6.3 Critical Chloride Content, C_{crit}

Critical chloride content (C_{crit}) is dependent on the reinforcement type. fib, SHRP2, VDOT, and Maine DOT have adopted C_{crit} values based on reinforcement types (Schiessl et al., 2006; Pease et al., 2019). Based on the literature search, SHRP2 has compiled a list of mean and standard deviation values of C_{crit} for different steel types. Table 6-4 summarizes the findings on C_{crit}.

Table 6-4: Critical Chloride Concentration Based on Steel Types

Steel Type	Alloy	Average C_{crit} (% wt. of cementitious material)	Standard Deviation	Minimum average, a	Maximum average, b	Distribution	Remark
Ordinary Mild Steel		0.6	0.15	0.2	2	beta	*fib*-34
MMFX		1.08	0.443			Lognormal	VDOT
Class I CRR		2.4					VDOT
Class II CRR		6					VDOT
Class III CRR		3.6					VDOT
Plain Reinforcement		0.50	0.125	0.2	2	beta	SHRP2
0.25% Plain Reinforcement		1.65	0.4125	0.75	1.9	beta	Maine DOT
Carbon Steel		0.68	0.47	0.13	2.16		Appendix C of SHRP2
Epoxy Coated Carbon Steel		0.63	0.24	0.32	0.9		Appendix C of SHRP2
Galvanized Carbon Steel		1.14	0.77	0.4	3.06		Appendix C of SHRP2
Low Carbon Chromium Steel		1.8	0.92	1	3.7		Appendix C of SHRP2
Stainless Steel-Clad Carbon Steel		5.2		4.42	5.99		Appendix C of SHRP2
Austenitic Steel	304	3.53	0.68	2.5	4.8		Appendix C of SHRP2
	304L	4.78	1.6	2.6	6.5		Appendix C of SHRP2
	316	3.32	0.72	2.5	4.3		Appendix C of SHRP2
	316L	4.88	1.68	2.6	6.9		Appendix C of SHRP2
	316L N	4.79	1.53	3	7.1		Appendix C of SHRP2
Duplex (Austenitic-Ferritic) Stainless Steel	2101L DX	2.37	1.04	0.9	4		Appendix C of SHRP2
	2304	3.7	1.2	2.6	5.9		Appendix C of SHRP2
	2205	3.5	0.06	3.4	3.6		Appendix C of SHRP2

Using the literature-obtained C_{crit} values, SHRP2 also determined the beta distribution parameters as shown in Table 6-5. According to SHRP2, Table 6-5 should only be used as an example to demonstrate the variation of C_{crit} with different steel types. It should not be used to in the design because it requires more robust testing and verification (Pease et al., 2019).

Table 6-5: Beta Distribution Parameters Based on Literature (source: Pease et al., 2019)

Beta Distribution Parameter	Steel Type			
	Carbon Steel	ASTM A1035-CS	304	316
Mean (% wt. of cementitious material)	0.6	1.49	3.66	4.29
Standard Deviation (% wt. of cementitious material)	0.15	0.53	1.03	1.49
Lower Boundary	0.2	0.9	2.5	2.5
Upper Boundary	2	3.8	6.6	7.2
R^2	-	0.98	0.975	0.971

Currently, there is no standard test method available to determine the critical chloride concentration in concrete. However, modified ASTM G109 can be used to determine the critical chloride concentration in concrete.

As mentioned earlier, the C_{crit} values of carbon steel, or other types of steel, are provided to demonstrate the variations observed in the literature. A deterministic, constant C_{crit} value of 0.2% wt. of cementitious material without any standard deviation or distribution may or may not be ideal to use with fib for Florida conditions (Sagues and Kranc, 1998). It is important to mention that C_{crit} is reported as the percent weight of the total cementitious material in the concrete mix, not by the percent weight of the individual mix components. Individual components may vary from one mix to the next. If C_{crit} is based on the individual mix components, it will vary by mix types, and within those mix types, it will also vary by steel types. So, there will be two levels for the C_{crit} value. A lot of experimental studies have to be performed to obtain those values. Thus, C_{crit} is based on the total cementitious material to simplify the adoption and implementation.

6.3.1 Recommendation

Currently, FDOT uses 1.2 lb/yd^3 as the critical chloride content, C_{crit} (Sagues and Kranc, 1998). Three types of inputs are required for any parameter in fib-Bulletin-34-based model. They are mean value, standard deviation and distribution. Thus, it is recommended that FDOT perform an experimental study to determine the fib required input parameters and distribution type of the critical chloride concentration in concrete based on the reinforcement types used in Florida. In the absence of further research, 0.1% to 0.25% by weight of cementitious material can be used as critical chloride content, with the maximum chloride content value not to exceed 1.2 lb/yd^3 for corrosion initiation in carbon steel, as suggested by FDOT.

6.4 Aging Exponent

The aging exponent describes the time-dependent change of chloride diffusion coefficients with time. It is dependent on concrete mix properties. This value is assumed during design based on the concrete mix properties and environmental conditions. Table 6-6 shows the values of the aging exponent recommended by fib. For different mix properties and w/cm ratios, chloride diffusion tests should be conducted at different ages of concrete using either Nord Test NT 492 or ASTM C1556 (ASTM C1556, 2022; NT Build 492, 1999). the aging exponent can be determined by plotting the apparent chloride diffusion against concrete age in a log-log graph. The slope of the graph will be the aging exponent. More details on this can be found on fib Bulletin 34 and 76 (Schiessl, et al., 2006; fib Bulletin 76, 2015).

Table 6-6: Aging Exponent Recommended by fib Bulletin 34

Concrete Type	Aging Exponent		
	Distribution	Mean Value	Standard Deviation
Portland Cement Concrete (CEM I)	Beta	0.3	0.12
Portland Cement Concrete + Fly ash ≥ 20%	Beta	0.6	0.15
Blast furnace slag cement concrete (CEM III/B)	Beta	0.45	0.2

6.4.1 Recommendation

fib recommended values of the aging exponent can be adopted if the concrete mix properties and environmental conditions match the fib conditions. Otherwise, an experimental study should be conducted using either Nord Test NT 492 or ASTM C1556 and following the aging exponent determination procedure outlined in fib Bulletin 34 and 76.

6.5 Aging Time Limit

According to fib Bulletin 34, monumental building structures, bridges should have a design service life of 100 years (Schiessl, et al., 2006). FDOT can adopt this fib recommended default value.

6.6 Chloride Migration Coefficient

Chloride migration coefficient is a function of concrete mix properties. fib recommends values of chloride migration coefficients based on some commonly used concrete mixes, shown in Table 6-7. Chloride migration coefficients for different type of concrete mixtures can be obtained by performing tests using Nord Test NT 492 or ASTM C1556.

Table 6-7: Chloride Migration Coefficients for Different Concrete Mixtures (Schiessl, et al., 2006)

Cement Type	Diffusion Coefficient $\times 10^{-8}$ (in^2/s)					
	w/c Ratio = 0.35	w/c Ratio = 0.4	w/c Ratio = 0.45	w/c Ratio = 0.5	w/c Ratio = 0.55	w/c Ratio = 0.6
CEM I 42.5 R	NA	1.38	1.55	2.45	3.05	3.88
CEM I 42.5 R + Fly Ash	NA	0.868	1.07	1.40	1.69	2.31
CEM I 42.5 R + Silica Fume	0.682	0.744	NA	NA	0.822	NA
CEM III/B 42.5	NA	0.217	0.295	0.434	0.465	0.527

NA = Chloride migration coefficient is not available for the concrete mixtures

6.6.1 Recommendation

It is recommended to obtain the chloride migration coefficients of concrete mixtures by performing tests using Nord Test NT 492 or ASTM C1556.

6.7 Transfer Function

fib Bulletin 34 recommends the following values of transfer function (Δx) based on the exposure conditions, as shown in Table 6-8. These values are also adopted by SHRP2 and Maine DOT (Pease et al., 2019).

Table 6-8: Transfer Function Recommended by fib Bulletin 34

<table>
<tr><th>Exposure</th><th colspan="2">Transfer Function Δx (in)</th><th>Distribution</th></tr>
<tr><td rowspan="4">Splash condition</td><td>mean</td><td>0.35</td><td rowspan="4">beta</td></tr>
<tr><td>standard deviation</td><td>0.22</td></tr>
<tr><td>min value, a</td><td>0.00</td></tr>
<tr><td>max value, b</td><td>1.97</td></tr>
<tr><td>Spray or Submerged condition</td><td colspan="2">0</td><td></td></tr>
</table>

6.7.1 Recommendation

FDOT may adopt the fib Bulletin 34 recommended values of transfer function.

6.8 Initial Chloride Content

Initial chloride content is a function of both chloride ingress from the surface and presence of chloride contaminated aggregates, cementitious materials or water in the concrete mix. In marine environment, the chloride content of fine or coarse aggregates, and water can be considerable. The input can be both probabilistic and deterministic. fib allows a uniform distribution of initial chloride content throughout the cross section. This value should be determined by testing the concrete mixes during the construction phase using ASTM C1152, and the value should be lower than the maximum initial chloride content allowed by the standard specification (ASTM C1152, 2020). Table 6-9 shows the values of initial chloride contents adopted by various fib models (Pease et al., 2019).

Table 6-9: Initial Chloride Content Adopted in Different fib Models

Adopted by	Input Type	Distribution	Mean Value (% wt. of cementitious material)	Standard Deviation (% wt. of cementitious material)
SHRP-2	Probabilistic	Normal	0.1	0.0001
VDOT	Probabilistic	Normal	0.034	0.021
Maine DOT	Probabilistic	Normal	0.04	0.00052

6.8.1 Recommendation

FDOT should determine the initial chloride contents based on the aggregates, cementitious material and water used in the concrete mixes during the construction phase using FM 5-516. Currently, Florida has two values of initial chloride contents based on the environmental exposure conditions. They are 0.7 lb/yd^3 and 0.4 lb/yd^3 for slightly and moderate to extremely aggressive environments, respectively. Thus, in the absence of further experimental study, a value between 0.035% and 0.085% by weight of cementitious material can be used as the mean for the initial chloride content with the maximum limit of 0.4 lb/yd^3, as per the suggestion by FDOT. SHRP 2 recommended normal distribution with a standard deviation of 0.0001% by weight of cementitious material may be used.

6.9 Concrete Cover

Concrete cover should be based on FDOT design specifications (FDOT, 2016).

6.10 Design Service Life

Design service life of the concrete structures should be based on the FDOT requirements.

6.11 Reliability Index

FDOT can adopt the fib Bulletin 34 recommended reliability index of 1.3.

Table 6-10 provides a summary of recommendations and suggested values of the input parameters associated with the fib 34 service life model.

Table 6-10: Recommendations and Suggested Inputs for fib-Bulletin-34-based Model

Input Parameters	Mean	Standard Deviation	Distribution	Beta Distribution Coefficients		Comment
				lower bound, *a*	upper bound, *b*	
Chloride Migration Coefficients						Should be obtained based on the concrete mixes
Temperature (°F)	77	10.3	Normal			Please refer to the above recommendation on Temperature
Aging Exponent						Please refer to the above recommendation provided on the Aging Exponent
Aging Time Limit	100	-	-	-	-	*fib* 34 recommendation
Initial Chloride Content (% wt. of cementitious material)	0.035-0.085	0.0001	Normal			Total value not to exceed 0.4 lb/yd^3; Please refer to the above recommendation provided on the Initial Chloride Content
Surface Chloride Content (lb/yd^3)	40	5	Log-Normal	-	-	Please refer to the above recommendation provided on the Surface Chloride Content
Transfer Function-Splash Zone (in)	0.35	0.22	Beta	0	1.97	fib 34 drop down
Transfer Function-Spray/Submerged Zone (in)	0	0	-	-	-	
Concrete Cover						Based on FDOT Specifications
Critical Chloride Content (% wt. of cementitious material)	0.1-0.25					Total value not to exceed 1.2 lb/yd^3; Please refer to the above recommendation provided on the Critical Chloride Content

Input Parameters	Mean	Standard Deviation	Distribution	Beta Distribution Coefficients		Comment
				lower bound, *a*	upper bound, *b*	
Design Service Life						Based on FDOT requirement
Reliability Index	1.3	-	-	-	-	*fib* 34 recommended value

The above recommendations for input parameters of the fib Bulletin 34 probabilistic model are based on the literature search. FDOT must perform experimental field durability studies to ensure the adopted values represent the actual field conditions. For instance, FDOT should design the concrete structures according to the standard specifications using the recommended and Florida-specific experimental test obtained values (FDOT, 2016; FDOT, 2023). Then, they should observe the long-term corrosion performance to determine if the concrete structures are performing adequately by conducting in-service monitoring, testing, and analysis. The input parameters must be revised based on the field durability study if the structures do not follow the expected design life deterioration.

7 Conclusions and Recommendations

In this study, a literature review was performed to compare the different service life models of corrosion in concrete structures. Based on the findings of the literature review, this study further reviewed Life-365 and fib-Bulletin-34-based probabilistic model. Sensitivity analyses were performed using both models by varying one input parameter at a time. This type of sensitivity analysis provides a general idea of relative variable importance but cannot determine the order of input importance. To illustrate this limitation, a full factorial design of the experiment and subsequent statistical analysis was conducted using fib-Bulletin-34-based model. Based on the analysis results, the following conclusions recommendations are provided:

- Regression based empirical models may be suitable for network-level assessment, but they require frequent calibration, particularly if the new data is out of bounds for which the model was initially calibrated.
- Markov chain based stochastic model can consider the uncertainty related to bridge condition deterioration, but it is difficult or almost impossible to calibrate the stochastic models for the time history of a particular bridge
- Many state DOTs, SHRP2, and AASHTO have adopted fib-Bulletin-34-based full probabilistic model for chloride-induced corrosion in concrete structures. Using probability distributions of the input parameters, it can model the uncertain nature of corrosion-induced damages. However, not considering the crack width and frequency and the adoption of a simpler Fick's law-based 1-D diffusion model can cause variations in the obtained results.
- Both the SHRP2 R19A model and AASHTO guide specification are based on fib-Bulletin-34-based probabilistic model. However, they have inherent differences and may produce different results. AASHTO guide specification for the service life of highway bridges provides calibrated design tables based on the probabilistic analysis for various environmental conditions and materials. Thus, the designer does not need to perform the corrosion analysis, simplifying the design process at the expense of flexibility. The SHRP2 R19A model assumes the designer will perform corrosion deterioration analysis using the inputs based on local conditions. This adds an extra step in the design process but provides flexibility to the designer. However, this does not necessarily mean one model is superior to the other but demonstrates the necessity of calibrating or validating the models based on local conditions.
- This study recommends the fib 34 based model for service life modeling of concrete structures against chloride-induced corrosion due to its inherent ability to consider the associated uncertainties. The risk analysis based on Life-365 is confined to some user-defined scenarios only. Deterministic service life prediction using Life-365 is a better choice if the user is confident about the field observations and knows the input parameters with a significant degree of certainty. Suppose there are material or region-specific variations in the input parameters, and the user is less confident about the field observations, or it is challenging to quantify an exact value of the input parameter. In that case, it is ideal to use a probabilistic analysis which can consider the input variations and uncertainties through appropriate distributions and standard deviations. Thus, this study recommends the fib-Bulletin-34-based model in such cases so that the associated uncertainties can be considered inherently in the analysis.

- The most important to least important input parameters are diffusion coefficient, concrete cover, aging exponent, surface chloride concentration, and temperature.
- A region-specific input variable is not significant for temperature range in Florida.
- Region specific inputs can be used for surface chloride concentration and concrete cover. Florida currently only uses two regions for concrete cover. Further region-specific inputs should be used after a field durability study if the surface chloride concentration is found to be 6lb/yd^3 or lower.
- Material specific input values should be used for diffusion migration coefficient and aging exponent based on the cementitious material type, water cement ratio, and presence of fly ash, slag, and admixtures.
- Recommendations and suggestions on the input parameters of the fib-Bulletin-34-based model were also provided.

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APPENDIX A

DESIGN OF EXPERIMENT

fib-34-Bulletin-based 64 simulations considered in this study for full factorial DOE analysis. They are:

Surface Chloride Concentration (lb/yd^3)	Diffusion Coefficient D_{28} $\times10^{-10}$(in^2/s)	Cover (inch)	Aging Exponent	Temperature (°F)	Reliability Index, β
6	8.12	2	0.3	67.2	0.601
20	8.12	2	0.3	67.2	-0.015
6	124	2	0.3	67.2	-1.187
20	124	2	0.3	67.2	-1.96
6	8.12	4	0.3	67.2	1.708
20	8.12	4	0.3	67.2	1.239
6	124	4	0.3	67.2	-0.127
20	124	4	0.3	67.2	-0.842
6	8.12	2	0.65	67.2	1.784
20	8.12	2	0.65	67.2	1.596
6	124	2	0.65	67.2	0.508
20	124	2	0.65	67.2	-0.117
6	8.12	4	0.65	67.2	1.896
20	8.12	4	0.65	67.2	1.92
6	124	4	0.65	67.2	1.622
20	124	4	0.65	67.2	1.132
6	8.12	2	0.3	78.9	0.346
20	8.12	2	0.3	78.9	-0.29
6	124	2	0.3	78.9	-1.492
20	124	2	0.3	78.9	-2.232
6	8.12	4	0.3	78.9	1.598
20	8.12	4	0.3	78.9	1.044
6	124	4	0.3	78.9	-0.404
20	124	4	0.3	78.9	-1.106
6	8.12	2	0.65	78.9	1.744
20	8.12	2	0.65	78.9	1.497
6	124	2	0.65	78.9	0.256
20	124	2	0.65	78.9	-0.454
6	8.12	4	0.65	78.9	1.899
20	8.12	4	0.65	78.9	1.96
6	124	4	0.65	78.9	1.524
20	124	4	0.65	78.9	0.935
6	8.12	2	0.3	67.2	0.628

Surface Chloride Concentration (lb/yd^3)	Diffusion Coefficient D_{28} $\times 10^{-10}$(in^2/s)	Cover (inch)	Aging Exponent	Temperature (°F)	Reliability Index, β
20	8.12	2	0.3	67.2	-0.033
6	124	2	0.3	67.2	-1.231
20	124	2	0.3	67.2	-2.01
6	8.12	4	0.3	67.2	1.748
20	8.12	4	0.3	67.2	1.26
6	124	4	0.3	67.2	-0.162
20	124	4	0.3	67.2	-0.806
6	8.12	2	0.65	67.2	1.825
20	8.12	2	0.65	67.2	1.525
6	124	2	0.65	67.2	0.515
20	124	2	0.65	67.2	-0.12
6	8.12	4	0.65	67.2	1.96
20	8.12	4	0.65	67.2	1.946
6	124	4	0.65	67.2	1.663
20	124	4	0.65	67.2	1.143
6	8.12	2	0.3	78.9	0.39
20	8.12	2	0.3	78.9	-0.299
6	124	2	0.3	78.9	-1.494
20	124	2	0.3	78.9	-2.428
6	8.12	4	0.3	78.9	1.679
20	8.12	4	0.3	78.9	1.064
6	124	4	0.3	78.9	-0.393
20	124	4	0.3	78.9	-1.098
6	8.12	2	0.65	78.9	1.698
20	8.12	2	0.65	78.9	1.486
6	124	2	0.65	78.9	0.299
20	124	2	0.65	78.9	-0.446
6	8.12	4	0.65	78.9	1.872
20	8.12	4	0.65	78.9	1.908
6	124	4	0.65	78.9	1.563
20	124	4	0.65	78.9	0.9

APPENDIX B

MODEL STATISTICS

Full Model

Serial	term	df	sumsq	meansq	statistic	p.value
1	concentration*	1	4.273006	4.273006	3363.041	5.45E-34
2	diffusion*	1	37.87325	37.87325	29807.89	4.28E-49
3	cover*	1	15.3615	15.3615	12090.17	7.79E-43
4	aging*	1	32.69409	32.69409	25731.67	4.49E-48
5	temperature*	1	0.57817	0.57817	455.0449	1.74E-20
6	concentration:diffusion*	1	0.455794	0.455794	358.7294	5.97E-19
7	concentration:cover*	1	0.093254	0.093254	73.39485	8.64E-10
8	concentration:aging*	1	0.375616	0.375616	295.6259	1.01E-17
9	concentration:temperature*	1	0.008813	0.008813	6.935831	0.012905
10	diffusion:cover*	1	0.748009	0.748009	588.7153	3.57E-22
11	diffusion:aging*	1	3.083097	3.083097	2426.531	9.56E-32
12	diffusion:temperature*	1	0.068841	0.068841	54.18056	2.27E-08
13	cover:aging*	1	0.735521	0.735521	578.8866	4.61E-22
14	cover:temperature*	1	0.029455	0.029455	23.18247	3.40E-05
15	aging:temperature*	1	0.048896	0.048896	38.48348	6.03E-07
16	concentration:diffusion:cover*	1	0.005948	0.005948	4.681543	0.038062
17	concentration:diffusion:aging*	1	0.128971	0.128971	101.5056	1.89E-11
18	concentration:diffusion:temperature	1	0.00335	0.00335	2.636214	0.114264
19	concentration:cover:aging	1	0.005094	0.005094	4.009506	0.053779
20	concentration:cover:temperature	1	9.75E-05	9.75E-05	0.076749	0.783534
21	concentration:aging:temperature	1	0.000186	0.000186	0.146107	0.70481
22	diffusion:cover:aging*	1	1.196563	1.196563	941.7465	2.63E-25
23	diffusion:cover:temperature	1	1.56E-08	1.56E-08	1.23E-05	0.997224
24	diffusion:aging:temperature*	1	0.006868	0.006868	5.405622	0.026573
25	cover:aging:temperature	1	5.08E-05	5.08E-05	0.039955	0.842834
26	concentration:diffusion:cover:aging*	1	0.005347	0.005347	4.20853	0.048481
27	concentration:diffusion:cover:temperature	1	0.000276	0.000276	0.217531	0.644087
28	concentration:diffusion:aging:temperature*	1	0.00646	0.00646	5.08441	0.031117
29	concentration:cover:aging:temperature	1	0.000159	0.000159	0.125447	0.725523
30	diffusion:cover:aging:temperature	1	0.004573	0.004573	3.59926	0.066861
31	concentration:diffusion:cover:aging:temperature	1	0.001838	0.001838	1.446795	0.237867
32	Residuals	32	0.040659	0.001271	NA	NA

Transformed Reduced Model

	term	estimate	std.error	statistic	p.value
1	(Intercept)	1.293189	0.010918	118.4485	6.46E-63
2	conc	-0.33408	0.010918	-30.5997	4.84E-34
3	difsn	-1.07392	0.010918	-98.3645	6.73E-59
4	cvr	0.702689	0.010918	64.36216	9.23E-50
5	agex	0.981695	0.010918	89.91748	5.84E-57
6	tmprtr	-0.11403	0.010918	-10.4449	3.64E-14
7	conc_cvr	-0.03371	0.010918	-3.08787	0.003286
8	difsn_cvr	-0.04253	0.010918	-3.89585	0.000291
9	difsn_agex	-0.0354	0.010918	-3.2424	0.002113
10	difsn_cvr_agex	0.37603	0.010918	34.44212	1.70E-36
11	conc_difsn_cvr	-0.05659	0.010918	-5.18349	3.93E-06
12	conc_difsn_agex	-0.17404	0.010918	-15.9409	3.06E-21
13	difsn_agex_tmprtr	-0.04536	0.010918	-4.15509	0.000127
14	conc_difsn_cvr_agex	-0.104	0.010918	-9.52572	8.03E-13