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# Bridge Service Life Design

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Final Report VTRC 18-R1

VIRGINIA TRANSPORTATION RESEARCH COUNCIL
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**Standard Title Page - Report on Federally Funded Project**

| 1. Report No.: FHWA/VTRC 18-R1 | 2. Government Accession No.: | 3. Recipient's Catalog No.: |
|---|---|---|
| 4. Title and Subtitle: Bridge Service Life Design | | 5. Report Date: June 2018 |
| | | 6. Performing Organization Code: |
| 7. Author(s): Elizabeth Rose Bales, Venkatasaikrishna Chitrapu, and Madeleine M. Flint, Ph.D. | | 8. Performing Organization Report No.: VTRC 18-R1 |
| 9. Performing Organization and Address: The Charles E. Via, Jr. Department of Civil and Environmental Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA | | 10. Work Unit No. (TRAIS): |
| | | 11. Contract or Grant No.: 107645 |
| 12. Sponsoring Agencies' Name and Address: Virginia Department of Transportation, 1401 E. Broad Street, Richmond, VA 23219 | Federal Highway Administration, 400 North 8th Street, Room 750, Richmond, VA 23219-4825 | 13. Type of Report and Period Covered: Final Contract |
| | | 14. Sponsoring Agency Code: |
| 15. Supplementary Notes: | | |

16. Abstract:

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In addition to supporting the implementation of service life design, several investigations identified key assumptions and variables in the service life model and identified critical areas for future characterization. The partial differential equation for apparent chloride diffusion was solved with both an approximate analytical approach and a numerical approach. Delays in the application of deicing salt were investigated using the numerical approach, and a ramp-type function for surface chloride concentration was explored using the analytical approach. Aging coefficients based on curing were also considered. A sensitivity analysis identified the aging coefficient and the surface chloride concentration as the most critical variables.

The study concluded that sufficient data are available to implement the *fib* Model Code for Service Life Design, but that caution in interpreting results is warranted because of the high uncertainty associated with the most critical variables. According to the results of the service life analyses, the regional climatic variability and differences in mix design across Virginia indicate that a "one-size-fits-all" approach to bridge deck specifications may not be appropriate. The use of corrosion-resistant steel and low-cracking concrete mixtures can provide a substantial (greater than 100 years) bridge deck service life.

| 17 Key Words: service life design, bridge deck, chloride diffusion, *fib* bulletin 34, numerical model, aging coefficient, life-cycle cost, NT Build 492 | 18. Distribution Statement: No restrictions. This document is available to the public through NTIS, Springfield, VA 22161. | | |
|---|---|---|---|
| 19. Security Classif. (of this report): Unclassified | 20. Security Classif. (of this page): Unclassified | 21. No. of Pages: 32 | 22. Price: |

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# FINAL REPORT

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In Cooperation with the U.S. Department of Transportation
Federal Highway Administration

Virginia Transportation Research Council
(A partnership of the Virginia Department of Transportation
and the University of Virginia since 1948)

Charlottesville, Virginia

VTRC 18-R1
June 2018

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## ABSTRACT

High costs and traffic disruption associated with the deterioration of reinforced concrete bridge decks because of corrosion have sparked renewed interest in service life design. Reinforced concrete bridge decks are exposed to chlorides from deicing salts; when the chlorides reach the steel reinforcement, they initiate corrosion. This study supports the adoption of the methodology described in *fib* Bulletin 34, *Model Code for Service Life Design*, for reinforced concrete bridge decks in Virginia. Concrete mixture properties and environmental exposure conditions were characterized. Values particular to regions within Virginia and suggested default values were identified and organized in a database to support the development of service life design guidelines. The predicted service life for eight bridge decks using low-cracking concrete and corrosion-resistant reinforcement (VDOT Reinforcement Class I, MMFX, ASTM 1035) was evaluated. The service life model was also implemented in a life-cycle cost analysis for a case study bridge, which found superior reliability of corrosion-resistant reinforcement from a life-cycle perspective.

In addition to supporting the implementation of service life design, several investigations identified key assumptions and variables in the service life model and identified critical areas for future characterization. The partial differential equation for apparent chloride diffusion was solved with both an approximate analytical approach and a numerical approach. Delays in the application of deicing salt were investigated using the numerical approach, and a ramp-type function for surface chloride concentration was explored using the analytical approach. Aging coefficients based on curing were also considered. A sensitivity analysis identified the aging coefficient and the surface chloride concentration as the most critical variables.

The study concluded that sufficient data are available to implement the *fib* Model Code for Service Life Design, but that caution in interpreting results is warranted because of the high uncertainty associated with the most critical variables. According to the results of the service life analyses, the regional climatic variability and differences in mix design across Virginia indicate that a “one-size-fits-all” approach to bridge deck specifications may not be appropriate. The use of corrosion-resistant steel and low-cracking concrete mixtures can provide a substantial (greater than 100 years) bridge deck service life.

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## INTRODUCTION

The average age of bridges in Virginia is 48 years, with 25% of bridges older than 60 years (Virginia Department of Transportation [VDOT], 2015). Many of these bridges were designed to last 50 years before needing major repairs or replacement. Bridge decks deteriorate from traffic loads but also from the corrosion damage caused by chloride-laden environments. Deicing salts, seawater, and atmospheric chlorides contribute to corrosion of the reinforcing steel in bridge decks. The direct cost of corrosion for highway bridges nationwide was estimated at $8.3 billion annually in 2002 (Koch et al., 2002). Maintaining, repairing, or replacing corrosion-damaged decks also contributes to substantial traffic disruption and user costs.

These cost and maintenance issues necessitate a more active approach to the durability design of bridge decks, specifically quantitative and probabilistic approaches that can characterize the service life of a particular design. A bridge component's service life represents its expected lifetime requiring only routine maintenance operations; no major repairs are necessary during a service life. With upkeep costs for existing structures rising, service life goals for new designs are changing. At the federal level, the goal of designing a bridge deck with a service life of 100 years is being promoted.

Bridges constructed in Virginia before 2007 are expected to have a service life of 50 years; bridges constructed since 2007 are expected to have a service life of 75 years (VDOT, 2015). These service lifetimes are a byproduct of other design limit states; service life design was not explicitly completed. Since 2007, VDOT has implemented new concrete mixtures and reinforcing steel types in an attempt to reach an even greater service life of 100 years; however, the service life impact of both the new mixtures and types of reinforcing steel has not yet been quantified. It is therefore necessary both to evaluate whether the new material technologies can support a 100-year service life and to implement a limit state-based durability design method for new bridge decks in Virginia.

## PURPOSE AND SCOPE

The purpose of this study was to provide a basis of knowledge to allow VDOT to implement a fully probabilistic service life design against deicing chloride-induced corrosion for new bridge decks using low-cracking concrete. Based on the recommendations of the Strategic Highway Research Program 2 (SHRP 2) Project R19A, the *fib* Bulletin 34 Model Code for Service Life Design (*fib*, 2006) was selected. Three objectives of the study were (1) to perform necessary characterization activities to support implementation; (2) to evaluate the potential significance of choices made in service life design implementation; and (3) to perform a preliminary assessment of the service life offered by decks with low-cracking concrete and corrosion-resistant reinforcement.

Characterization activities generated or gathered all necessary data to implement service life design. The scope of the characterization study included material testing to determine concrete mixture design parameters, including initial chloride concentration, diffusion coefficient, and aging coefficient; environmental characterization to obtain the annual temperature and surface chloride concentration through the gathering of meteorological data to implement the *fib* approach; a literature review to compile relevant surveyed data; and determination of all other necessary parameters, including critical chloride content to initiate corrosion, through the literature review.

Evaluation activities explored various assumptions and choices made in the implementation of service life design. The scope of this portion of the study included evaluation of possible modifications to the *fib* methodology, such as use of a numerical approach for modeling chloride diffusion; consideration of delays in the time to first application of deicing salts; use of a ramp-type function for surface chloride concentration; and use of aging coefficients derived from rapid chloride migration (RCM) tests. Sensitivity assessments were used to identify the most critical parameters.

Assessment activities implemented the suggested approach for the eight bridges included in the study and a case study bridge. The case study bridge was used to explore practical questions related to application of the *fib* methodology in Virginia, including the importance of regional environmental variation and life-cycle cost considerations.

## METHODS

### Overview

To achieve the study objectives, six tasks were performed:

1. concrete material testing to characterize the required probabilistic distributions of concrete properties used in the *fib* methodology

2. environmental characterization from both the literature and meteorological stations across Virginia

3. a literature review to determine all other necessary model parameters

4. evaluation of possible modifications to the *fib* methodology, including using a numerical approach for chloride diffusion, considering a delay in first exposure to deicing salts, using a ramp-type function for surface chloride concentration, and using curing-based aging coefficients

5. multivariate sensitivity assessments to identify the most critical model parameters and convergence studies to determine the number of required simulations

6. implementation of the *fib* approach for all bridges involved in the study and a more extensive analysis of a case study bridge, which included assessment of the importance of regional environmental variation and life-cycle cost analysis.

*fib* Bulletin 34, *Model Code for Service Life Design*, offers fully probabilistic methods for service life design that aim to balance accuracy and ease of use (*fib*, 2006). In an approximate solution to one-dimensional Fickian diffusion, chloride concentration at a depth, $x$, and time, $t$, is predicted by Equation 1:

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

where

$C_0$ is the initial chloride concentration
$C_{s,\Delta x}$ is the surface chloride concentration at depth $\Delta x$
$\Delta x$ is the depth of the so-called "transfer function," also known as the convection zone
*erf* represents the error function
$a$ is the depth of the reinforcing steel
$D_{app,C}$ is the apparent diffusion coefficient.

In the fully probabilistic approach, each of the parameters on the right side of Equation 1 with the exception of time is assumed to be a random variable. One variable, $D_{app,C}$, is determined by a function of more basic random variables; $C_{s,\Delta x}$ may also be determined by a function of basic random variables.

Failure is defined as $C(x = a, t) > C_{crit}$, where $C_{crit}$ is the total chloride content that leads to depassivation of the reinforcing steel (*fib*, 2006), also known as the critical chloride concentration. Because the *fib* approach is probabilistic, satisfactory performance is evaluated in accordance with a target reliability index ($\beta$) or a target probability of corrosion initiation (probability of "failure," $p_f$), with 10% probability of failure suggested as the limit for acceptable performance.

VDOT laboratories characterized the concrete parameters: initial chloride concentration and the chloride migration coefficient as obtained from the NT Build 492 test (NordTest, 1999). The chloride migration coefficients were used to determine the apparent diffusion coefficient and to explore the aging coefficient. Survey data were collected, and historical temperature, snowfall, and precipitation data were analyzed to characterize exposure conditions in Virginia. Exposure conditions include surface chloride concentration and ambient air temperature. Additional parameters were obtained from the literature and/or from *fib*.

Potential modifications to the *fib* approach were also evaluated. Instead of using the error function approximation given in Equation 1, the partial differential equation was solved directly using a finite difference approach, here called the *fib*-Numerical model. Using the *fib*-Numerical model it was possible to include a delay to the first application of deicing salts or the use of a ramping surface chloride concentration function. A reliability-based multivariate sensitivity assessment was also performed on the *fib*-recommended equation to compute surface chlorides. Using the sensitivity analysis as guidance, the researchers made a number of comparisons using different methods or sources to obtain distributions for the *fib* parameters.

The service life of each bridge deck included in the study was assessed. For a case study bridge, a hypothetical regional analysis and a life-cycle cost analysis were completed. The life-cycle cost analysis compared the cost of plain reinforcing steel and corrosion-resistant reinforcing steel over the predicted service life.

## Concrete Characterization

The testing done to quantify the concrete mixture characteristics, initial chloride concentration, and the diffusion coefficient was carried out by VDOT laboratories. It was initially intended that 17 bridges undergo testing; however, testing was completed for 8 bridges. Location and mixture data for the 8 bridges are provided in Table 1.

**Table 1. Bridge Decks Included in the Study and Their Mixtures Properties**

| Bridge | Location | Concrete Mixture Description | | |
|---|---|---|---|---|
| | | **Composition (lb/yd$^3$)** | **w/c** | **Class** |
| Bridge 1 - Richmond District | Eastern Piedmont | Cement (Type II): 395; Slag: 263; Water: 296 | 0.45 | A4 - Modified Lightweight |
| Bridge 2 - Richmond District | Eastern Piedmont | Cement (Type II): 476; Fly Ash: 159; Water: 285.75 | 0.45 | A4 |
| Bridge 3 - Richmond District | Eastern Piedmont | Cement (Type II): 460; Fly Ash: 145; Water: 250 | 0.41 | A4 - Low Cracking With HRWR/ Retarder |
| Bridge 4 - Bristol District | Southwestern Mountain | Cement (Type I/II): 520; Fly Ash: 127; Water: 272 | 0.42 | A4 - Modified |
| Bridge 5 - Bristol District | Southwestern Mountain | Cement (Type I/II): 504; Fly Ash: 126; Water: 265 | 0.42 | A4 - Modified |
| Bridge 15 - Richmond District | Eastern Piedmont | Cement (Type II): 460; Fly Ash: 145; Water: 260 | 0.43 | A4 - Low Cracking With HRWR |
| Bridge 16 - Richmond District | Eastern Piedmont | Cement (Type II): 460; Fly Ash: 145; Water: 260 | 0.43 | A4 - Modified Lightweight |
| Bridge 17 - Lynchburg | Western Piedmont | Cement (Type II): 460; Fly Ash: 145; Water: 260 | 0.43 | A4 - Low Cracking |

w/c = water/cement ratio; HRWR = high-range water reducer.

Specimen collection and testing were completed as shown in Table 2.

**Table 2. Experimental Design for Material Testing**

| Bridge | No. of Specimens | | | | |
|---|---|---|---|---|---|
| | NT Build 492 | | | | Initial Chloride Content Titration |
| | 14 days | 28 days | 56 days | 90 days | |
| Bridge 1 - Richmond District | 3 | 3 | 3 | -- | -- |
| Bridge 2 - Richmond District | 6 | 6 | 6 | -- | -- |
| Bridge 3 - Richmond District | 6 | 6 | 6 | -- | -- |
| Bridge 4 - Bristol District | 3 | 3 | 3 | -- | 3 |
| Bridge 5 - Bristol District | 6 | 6 | 6 | 6 | 3 |
| Bridge 15 - Richmond District | 3 | 3 | 3 | 3 | 3 |
| Bridge 16 - Richmond District | 6 | 6 | 6 | -- | 6 |
| Bridge 17 - Lynchburg | -- | 9 | -- | -- | -- |

## Initial Chloride Concentration

Chloride titrations were completed by VDOT for four decks to test for the initial, or background, chloride concentrations, $C_0$ in Equation 1. The chloride titration test yields the total, bound, and free chlorides in the concrete sample. Additional surveyed data were obtained from Williamson (2007).

## Chloride Migration Coefficient and Apparent Diffusion Coefficient

To determine the reference chloride migration coefficient of the bridge deck concrete, $D_{RCM,0}$, the samples underwent the NT Build 492 test at VDOT laboratories (NordTest, 1999). NT Build 492 is similar to the modified ASTM C1202 protocol used by VDOT for RCM testing. Samples from all bridges were tested at 28 days of age. Additional tests at 14, 56, and 90 days of age were also performed for some bridges, as described in Table 2. At least three cylinders were tested at each age. The RCM coefficient is used to determine the apparent diffusion coefficient in accordance with Equation 2:

$$D_{app,C} = k_e D_{RCM,0} k_t A(t) \qquad \text{(Eq. 2)}$$

where

$k_e$ is the environmental transfer parameter (see Eq. 3)
$k_t$ is the transfer parameter, taken as 1 (*fib*, 2006)
*A(t)* is the aging sub-function (described in the next section).

The environmental transfer parameter, $k_e$, corrects for any variation between the laboratory temperature and the exposure temperature. It is given by Equation 3:

$$k_e = \exp\left[ b_e \left( \frac{1}{T_{ref}} - \frac{1}{T_{real}} \right) \right] \qquad \text{(Eq. 3)}$$

where

$b_e$ is a regression variable
$T_{ref}$ is the standard test temperature
$T_{real}$ is the temperature of the structural element or the ambient air.

## Aging Coefficient

Various processes such as concrete curing and chloride binding cause the apparent diffusion coefficient to change over time. These processes are accounted for by *A(t)*, the aging sub-function in Equation 4:

$$A(t) = \left(\frac{t_0}{t}\right)^{\alpha} \qquad \text{(Eq. 4)}$$

where

$\alpha$ is the aging coefficient
$t_0$ is the reference point of time, 28 days (in years).

The particular aging sub-function used here is based on a regression completed on hundreds of published chloride profiles from existing reinforced concrete structures (*fib*, 2006). Further guidelines from *fib* (2015) detail the process of determining the aging coefficient (Balázs et al., 2013).

An alternative approach, in which the RCM test results at different days of age were used to find a curing-based aging coefficient, was also explored. A nonlinear regression analysis on Equation 5 using the $D_{RCM}$ data was performed using the curve fitting toolbox in MATLAB (cftool). The regression was forced through the mean value of $D_{RCM,0}$ in accordance with *fib* (2015). The curing-based aging coefficients were then compared to the values provided by *fib* for similar mixtures.

## Exposure Characterization

A division of Virginia into six climate regions was used, as shown in Figure 1.

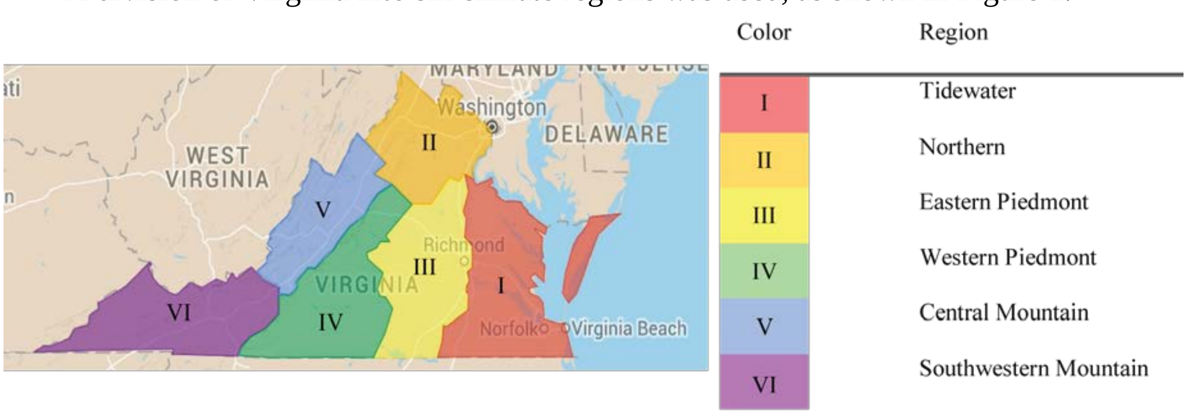


**Figure 1. Regional Division of Virginia for the Purpose of Exposure Characterization**

This separation is supported by and used in the work of Williamson (2007). The exposure elements to be classified in this work included regional temperature and surface chloride concentration for the six climate and exposure regions of Virginia: Tidewater, Northern, Eastern Piedmont, Western Piedmont, Central Mountain, and Southwestern Mountain.

## Temperature

The annual mean and standard deviation of temperature were quantified for each region. The National Climate Data Center (NCDC) maintains historical weather and climate data and was the source for the temperature data. Weather stations in each region were accessed to gather temperature data for the past 30 years (1985-2015). Bales (2016) lists the weather stations used for temperature characterization.

## Surface Chloride Concentration

Two methods were used to find surface chloride concentrations for the six Virginia climate regions. The first method was a collection of historical surface chloride concentration test results. The second method followed the *fib* method for calculating surface chloride concentration based on precipitation and the volume of deicing salts applied. Additional details of this application of the methodology is available in Bales (2016). When comparing the *fib* method surface chloride concentrations to the historical data results, the units must first be made similar. To convert the historical data to percent mass binder from percent volume concrete, a binder content of 300 kg/m$^3$ was assumed.

### *Historical Test Data Collection*

Several previous investigations studied the surface chloride concentration of bridges in Virginia (Kirkpatrick, 2001; Williamson, 2007). Their findings were compiled to establish the mean surface chloride concentration of bridges within the six climate regions of Virginia. The locations of the bridges where surface chloride concentrations were reported are shown in Figure 2.


**Figure 2. Location of Bridges for Which Surface Chloride Concentration Data Were Processed**

*fib Method for Computing Surface Chloride Concentration*

*fib* Bulletin 34 details a method for calculating, $C_{s,\Delta x}$, a representative surface chloride concentration for a structure subject to deicing salts in the case that surveyed data are not available (*fib*, 2006). According to *fib*, the concentration of chloride-contaminated water because of deicing salts is found with Equation 5:

$$C_{oR} = \frac{n c_{s,i}}{h_{s,i}} \qquad \text{(Eq. 5)}$$

where

$C_{oR}$ is the average chloride concentration
$n$ is the average number of salting events per year
$c_{s,i}$ is the average amount of chloride spread per event
$h_{s,i}$ is the amount of water from rain and snow during the events.

The average amount of chloride spread was quantified by Williamson (2007) for winter seasons in 2000-2001, 2001-2002, and 2002-2003. During these seasons, snowfall was observed as early as October and as late as April; therefore, the winter season herein is defined as October through April. The chloride spread per event must be in units of grams per square meter for use in the *fib* method. To convert the amounts from Williamson (2007) to be used in Equation 6, it was assumed that bridges in Virginia, on average, have a lane width of 12 ft.

To evaluate the number of salting events, daily snowfall, daily precipitation, and average daily temperature records from NCDC were accessed. A list of weather stations used is provided in Bales (2016). These weather stations were accessed for October 1, 2000, through April 30, 2001. A salting event was defined as a day where there was snowfall and the daily average temperature was between 20°F and 32°F. It is VDOT procedure to apply deicing salts only when the temperature is above 20°F because of their ineffectiveness when the temperature is less than 20°F (Williamson et al., 2007).

Equivalent precipitation from snowfall and precipitation amounts recorded during the salting events was summed for the winter season. This value was used for $h_{s,i}$, precipitation from rain and snow. To quantify the amount of precipitation occurring with each salting event, precipitation accumulated during salting events was quantified. Since snowfall data are collected as a depth of snowfall, they must be converted to an equivalent rainfall amount to be used in Equation 6. The conversion between snowfall and equivalent precipitation used the method of Kyle and Wesley (1997), which is based on air temperature.

*fib* instructs that once the chloride adsorption isotherm for the type of cement and the concrete composition is known, the chloride saturation content, or total chloride content, in the concrete at the surface, can be calculated from $C_{oR}$. The methods expressed in Tang and Sandberg (1996) are to be carried out to calculate the chloride saturation content, where $C_{s,0}$ is equal to the surface chloride concentration at the depth of the convection zone, $C_{s,\Delta x}$. Tang and Sandberg (1996), however, do not detail a repeatable methodology for this calculation. For this reason, the example given by *fib* was followed to find the surface chloride concentration, $C_{s,\Delta x}$.

The binder content was assumed to 300 kg/m$^3$, and the water to cement ratio was assumed to be 0.50. From there, $C_{s,\Delta x}$ was determined based on the correlation provided by *fib* between $C_{s,0}$ and $C_{s,\Delta x}$.

## Determination of Additional Parameters

Sources for probabilistic distributions of additional required parameters were determined from *fib* and/or from a literature review of related work previously carried out under the Virginia Transportation Research Council (VTRC). The parameters determined through the literature review were $\Delta x$, depth of the convection zone (Cady and Weyers, 1983); $b_e$, the temperature regression coefficient (*fib*, 2006); the critical chloride concentration, $C_{crit}$, of plain and corrosion-resistant reinforcement (Ji et al., 2005); and variability in the concrete cover depth, *a* (*fib*, 2006).

## Evaluation of Modifications to the *fib* Methodology

### Numerical Solution Method Comparison

According to prior research (e.g., Flint et al., 2014; Justnes and Geiker, 2012; Luping and Gulikers, 2007), there is some controversy regarding the appropriate solution of the diffusion equation when some term is time-dependent. In real structures, the apparent chloride diffusion coefficient does change over time (because of curing and binding), which suggests the implementation of an approach that directly characterizes the evolution of this variable. Numerical approaches such as finite difference methods can account for this evolution. In such an approach, both distance, *x*, and time, *t*, are discretized as shown in Figure 3. In the literature, it is unclear whether such an approach is compatible with the aging sub-function as described in Equation 5. Because the entire history of the diffusion coefficient would be incorporated into the numerical solution, faster predicted ingress of chlorides would be expected.

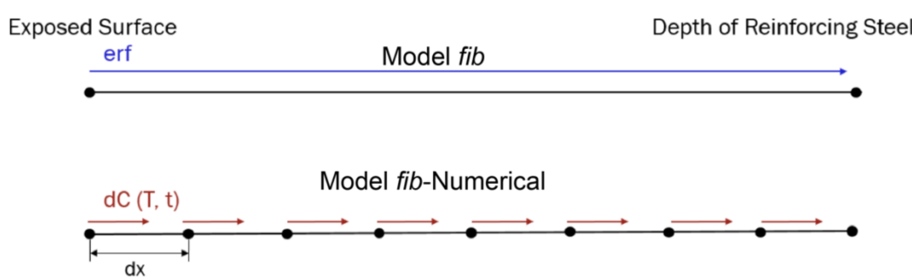

**Figure 3. Graphical Comparison of Solution Methods**

To explore this issue, studies were conducted with the *fib*-Numerical model, which was based on the *fib* definition of variables but used a finite difference solution for one-dimensional Fickian diffusion, as opposed to the *fib* error function solution. The governing partial differential equation was Equation 6:

$$\frac{\partial C}{\partial t} = \frac{\partial}{\partial x}\left\{ D_{app,C} \frac{\partial C}{\partial x} \right\} \qquad \text{(Eq. 6)}$$

where

$C$ represents the chloride concentration
$x$ is the depth, and $t$ is time
$D_{app,C}$ is the apparent diffusion coefficient with the same definition as in *fib* (2006).

The differential equation was solved using the MATLAB "pdepe" function with the initial condition as the initial chloride content $C_0$. The initial chloride content was assumed to be constant throughout the depth. The boundary conditions were as follows: at the "surface" (taken as the depth of the convection zone, $\Delta x$), the concentration of chlorides was equal to the surface chloride concentration, $C_{s,\Delta x}$, and a "no flux" condition was prescribed at the boundary farthest away from the exposed surface. A mesh convergence study was performed on this model and is described in Bales (2016). The depth selected was 50 mm beyond the cover depth (i.e., the maximum distance $x$ would be $a$ plus 50 mm).

To verify that the *fib* and *fib*-Numerical solutions were working correctly, their results were verified against a chloride profile taken from a 17-year-old bridge in Giles, Virginia (Williamson, 2007). The methods and results of this validation are provided in Bales (2016).

To compare the performance of the solution methods, both approaches were used to evaluate a case study bridge located in Lynchburg (this bridge is similar to Bridge 17). The input variables describing the case study bridge are provided in Table 3.

The distributions used for the random variables were either fit to data from the listed source or chosen at the instruction of the listed source. The type of reinforcing steel used was multiphased martensitic formable steel (MMFX), which is a low carbon, chromium steel (VDOT Reinforcement Class I, ASTM-1035).

**Table 3. Input Variables for Case Study Bridge**

| Variable | Description | Distribution | Justification for Distribution | Mean | Standard Deviation | Source of Data |
|---|---|---|---|---|---|---|
| $a$ | Cover depth (mm) | Lognormal | *fib* (2006) | 50 | 0.12 | VDOT |
| $b_e$ | Regression variable | Normal | *fib* (2006) | 4800 | 0.15 | *fib* |
| $\alpha$ | Aging factor | Beta | *fib* (2006) | 0.3 | 0.25 | *fib* |
| $D_{RMC,0}$ | Rapid migration coefficient ($mm^2$/year) | -- | *fib* (2006) | 17.49 | .13 | VDOT |
| $t_o$ | Reference time, years | -- | -- | 17 | -- | *fib* |
| $T$ | Yearly average temperature (K) | Normal | *fib* (2006) | 285.2 | 7.98 | NCDC |
| $T_{ref}$ | Reference temperature (K) | -- | -- | 293 | -- | *fib* |
| $C_0$ | Initial chloride concentration (% mass concrete) | Normal | Fit to data from this study | 0.3 | 0.027 | VDOT |
| $C_{s,\Delta x}$ | Surface chloride concentration (% mass concrete) | Lognormal | Fit to data of Ji et al. (2005) | 5.57 | 0.41 | Ji et al. (2005) |

## Variation in the Time to First Chloride Exposure (*fib*-Numerical Model)

The original *fib* service life model solves for the probability of failure on the condition that the bridge deck is exposed to chlorides immediately from the start of service. To understand what effect a delay to first chloride exposure has on the probability of corrosion initiation, the *fib*-Numerical model was run for one bridge at 5,000 simulations for each monthly increment of delay in time of application of deicing salts. The effect of a 12-month delay to first chloride exposure was studied for all bridges.

### Ramp-Type Function for Surface Chlorides (*fib* Model)

Previous studies of Virginia bridge decks have shown that taking surface chlorides as a ramp-type function as opposed to a constant value throughout the service life leads to a marginal increase in the time to initiate corrosion. Phurkhao and Kassir (2005) modified data of Virginia bridge decks from Weyers et al. (1994) to model the surface chlorides as a ramp-type concentration for different diffusion coefficients and cover depths. According to Williamson (2007), for a typical Virginia bridge deck, the differences in service life when using the ramp function are expected to be in the range of 5% to 15%.

### Curing-Based Aging Coefficients (*fib* Model)

The *fib* model was run using aging coefficients derived from RCM tests at multiple days of curing as opposed to *fib*-defined coefficients.

## Sensitivity Analysis

### Multivariate Analysis Using the First Order Reliability Method (FORM)

*Analysis of the Limit State Function for Depassivation*

Given the varying quantities of data available to characterize each of the random variables in Equation 1, a sensitivity assessment is desirable to identify the most critical variables. Previous studies identified the aging coefficient (Boddy et al., 1999) and the surface chloride concentration (Saassouh and Lounis, 2012) as the most important. A comprehensive reliability-based multivariate sensitivity assessment was performed using FORM (Melchers, 1999) on the limit-state function for depassivation of reinforcement. In FORM, the sensitivity is determined using importance factors that reflect the relative influence of each random variable at the most probable point to cause failure of the system. Additional details on the sensitivity assessment are provided in Chitrapu (2017).

*Analysis of the fib Surface Chloride Concentration Model*

The FORM sensitivity assessment was also completed for the variables contained in the *fib*-recommended equation to compute surface chlorides. In this analysis Equation 6 was substituted into Equation 1. Because uncertainties of the basic random variables (i.e., number of salting events, deicing salts applied, amount of precipitation) were not quantified in this study, they were assumed to be normally distributed, with the mean values provided in Bales (2016). The sensitivity assessment was performed three times, with coefficients of variation for each of the variables of 0.2, 0.3, and 0.4.

## Monte Carlo Convergence

Monte Carlo convergence studies were completed on both the error function and numerical approaches for multiple bridges to ensure that the sampling was representative and resulted in a converged probability of failure. The coefficient of variation of the probability of failure was computed according to Equation 7:

$$\delta_{p_f} = \sqrt{\frac{1-p_f}{np_f}} \qquad \text{(Eq. 7)}$$

where $p_f$ is the average probability of failure for a certain number of samples, *n*.

Equation 8 can be rewritten to give the required number of samples to provide a confidence interval (coefficient of variation) on the probability of failure estimate. Here the desired confidence interval was 10%. The initial guess for $p_f$ was obtained using 1000 samples, and then the required *n* was calculated and additional simulations were performed. The probability of failure was considered converged if the coefficient of variation of the average probability of failure for the five runs at a given sample size was less than 10%.

# Implementation of Service Life Prediction

## Implementation Using Plain and Corrosion-Resistant Steel

Service life predictions were carried out using the *fib* methodology for eight bridges, listed in Table 1. A spreadsheet was used to generate Monte Carlo samples of all model parameters included as random variables in the *fib* model given in Equation 1 and the equations used to derive the dependent variables in Equation 1 (i.e., $D_{app,C}$ and $C_s$). Matlab scripts were also developed to implement the numerical approach. The distribution parameters assumed for each bridge are provided in Chitrapu (2017).

## Implementation for Case Study Bridge

Service life predictions were also made for a Lynchburg case study bridge similar to Bridge 17. More details on the case study bridge are provided in Bales (2016); the assumed distribution parameters are provided in Table 3.

### *Regional Comparison*

The *fib* model was run for the case study bridge as if it were to be built in each region across the state to compare the effect the regional exposure conditions have on service life. The model input variables that changed with region were temperature and surface chloride concentration. The data collected in the environmental characterization were used in this assessment.

*Life-Cycle Cost Analysis*

Costs associated with plain reinforcing steel and corrosion-resistant reinforcement (VDOT Reinforcement Class I, MMFX, ASTM 1035) were compared for both solution methods (error function and numerical). Based on the time to end of service life for each reinforcing steel, the equivalent uniform annual cost (EUAC) distribution was calculated (see Eq. 8).

$$EUAC = \frac{NPV}{\left(1 + \frac{1}{(1+r)^t}\right) \Big/ r} \qquad \text{(Eq. 8)}$$

where

*NPV* is net present value
*r* is the discount rate
*t* is the time over which the *EUAC* is being considered.

For this analysis, net present value (NPV) was the initial cost of the reinforcing steel. It was assumed that construction and labor costs associated with each type of reinforcing steel were the same. This analysis was also independent of the type of maintenance strategy chosen. The distribution of cost for both types of reinforcing steel came from VDOT bid tabulations for the year 2015 and is shown in Table 4.

The discount rate, *r*, can be determined from the real discount rate relating to the interest rate on a treasury bond with the same analysis time period (Williamson, 2007). This interest rate, 1.5%, was found using Office of Management and Budget (OMB) data (2015), with interest rates updated yearly. The time, *t*, is the time, in years, of service life as predicted from the service life models, *fib* and *fib*-Numerical, for the respective reinforcing steel type. Because NPV and time of service life are stochastic variables, Monte Carlo simulation was completed to develop a distribution of EUAC for each reinforcing steel type.

**Table 4. Probability Density Function Parameters for Reinforcing Steel Based on 2015 Unit Costs**

| Reinforcing Steel Type | Distribution Type | Mean, $/lb | | Standard Deviation, $/lb | |
|---|---|---|---|---|---|
| | | Sample | Distribution | Sample | Distribution |
| Plain | log-logistic | 2.15 | 2.03 | 2.23 | 3.04 |
| Corrosion resistant (VDOT Reinforcement Class I, MMFX, ASTM 1035) | log-logistic | 3.23 | 3.01 | 1.12 | 1.18 |

# RESULTS

## Concrete Characterization

### Initial Chloride Concentration

Means and standard deviations of initial chloride concentration titration test results, as completed by VDOT, are shown in Table 5. Raw data are provided in Bales (2016) and are stored at VTRC. In the absence of any evidence to the contrary, the initial chloride concentration was assumed to be normally distributed.

**Table 5. VDOT Initial Chloride Concentration Results**

| Bridge | Mean Initial Chloride Concentration (% mass binder) | Std. Dev. of Initial Chloride Concentration (% mass binder) |
|---|---|---|
| Bridge 4: Bristol District | 0.033 | 0.009 |
| Bridge 5: Bristol District | 0.024 | 0.003 |
| Bridge 15: Richmond District | 0.004 | 0.0006 |
| Bridge 16: Richmond District | 0.054 | 0.006 |
| All Samples | 0.034 | 0.021 |

## Chloride Migration Coefficient and Apparent Diffusion Coefficient

Statistics regarding the VDOT results for the NT Build 492 tests at 28 days are shown in Table 6. *fib* instructs that a standard deviation equal to 20% of the mean is to be used for the standard deviation of the chloride migration coefficient. The VDOT results generally had lower values for the coefficient of variation than suggested by *fib* (the exceptions were Bridges 5 and 16).

**Table 6. Statistics of Rapid Chloride Migration Tests at 28 Days**

| Bridge (No. of Samples) | $D_{RCM,0}$ ($m^2/s$) | | $D_{RCM,0}$ ($mm^2/year$) | | Coefficient of Variation |
|---|---|---|---|---|---|
| | Mean | Std. Dev. | Mean | Std. Dev. | |
| Bridge 1 - Richmond District (3) | 4.40E-12 | 7.95E-13 | 139 | 25 | 0.18 |
| Bridge 2 - Richmond District (6) | 1.96E-11 | 2.12E-12 | 620 | 67 | 0.11 |
| Bridge 3 - Richmond District (6) | 1.52E-11 | 1.99E-12 | 479 | 63 | 0.13 |
| Bridge 4 - Bristol District (3) | 2.23E-11 | 8.78E-13 | 703 | 28 | 0.04 |
| Bridge 5 - Bristol District (6) | 1.80E-11 | 6.52E-12 | 567 | 206 | 0.36 |
| Bridge 15 - Richmond District (3) | 9.04E-12 | 1.53E-12 | 285 | 48 | 0.17 |
| Bridge 16 - Richmond District (6) | 6.26E-12 | 3.37E-12 | 197 | 106 | 0.54 |
| Bridge 17 - Lynchburg (9) | 1.48E-11 | 1.88E-12 | 467 | 59 | 0.13 |

## Aging Coefficient

Figures 4 and 5 show plots of the curing age and $D_{RCM,0}$ values along with the curve fit for Bridges 1 and 4. Bridge 1 has Type II cement with 40% slag, whereas Bridge 4 has Type I/II cement with 19.6% fly ash. Additional plots are provided in Chitrapu (2017). Table 7 summarizes the results of the regression analysis for the remaining bridges, with the exception of Bridge 17, for which RCM testing was not performed for ages other than 28 days. R-squared values of the regressions were low, indicating poor fits. The regression values also differed from the diffusion-derived aging coefficients suggested by *fib.*

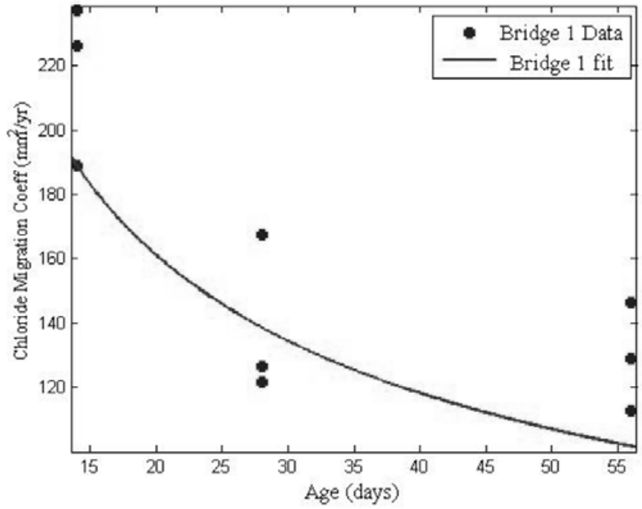


**Figure 4. Curve Fit to Determine Curing-Based Aging Coefficient: Bridge 1**

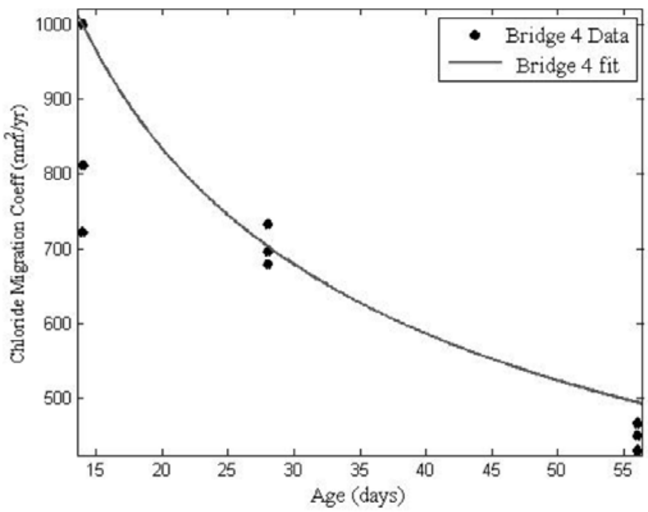


**Figure 5. Curve Fit to Determine Curing-Based Aging Coefficient: Bridge 4**

**Table 7. Curing-Based Aging Coefficients From Nonlinear Regression**

| Bridge (Total No. of Samples) | Mixture Details | Mean $D_{RCM,0}$ at 28 Days ($mm^2$/year) | Curing-Based Aging Coefficient, $\alpha$ | R-Squared | *fib*-Based Aging Coefficient, $\alpha$ |
|---|---|---|---|---|---|
| Bridge 1 (9) | Cement (Type II) + 40% Slag | 139 | 0.446 | 0.54 | 0.45 |
| Bridge 2 (18) | Cement (Type II) + 25% Fly Ash | 620 | 0.240 | 0.56 | 0.6 |
| Bridge 3 (18) | Cement (Type II) + 24% Fly Ash | 479 | 0.870 | 0.85 | 0.6 |
| Bridge 4 (9) | Cement (Type I/II) + 19.6% Fly Ash | 703 | 0.510 | 0.97 | 0.6 |
| Bridge 5 (27) | Cement (Type I/II) + 20% Fly Ash | 567 | 0.318 | 0.51 | 0.6 |
| Bridge 15 (27) | Cement (Type II) + 40% Slag | 285 | 0.662 | 0.37 | 0.45 |
| Bridge 16 (18) | Cement (Type II) + 40% Slag | 197 | 0.766 | 0.34 | 0.45 |

## Exposure Characterizations

### Temperature

The mean and standard deviation of annual average temperature for the regions of Virginia from 1985-2015 are shown in Table 8.

**Table 8. Annual Average Temperature Statistics**

| Region | Mean, Kelvin (F) | Std. Dev., Kelvin (F) |
|---|---|---|
| Tidewater | 288 (59) | 7.9 (18) |
| Northern | 286 (55) | 8.4 (30) |
| Eastern Piedmont | 287 (57) | 8.2 (29) |
| Western Piedmont | 287 (56) | 8.0 (29) |
| Central Mountain | 285 (54) | 8.1 (29) |
| Southwestern Mountain | 284 (53) | 7.9 (28) |

### Surface Chloride Concentration

The average and standard deviation of the collected historical surface chloride concentration surveys and the *fib* method are shown in Table 9. The data used to compute the *fib* results are shown in Table 10. The results for surface chloride concentration obtained using the *fib* approach vary considerably from the historical data but are of the same order of magnitude.

**Table 9. Comparison of Historical and *fib* Method Surface Chloride Concentrations**

| Region | Historic Mean (kg Cl/m$^3$ concrete) | Historic Std. Dev. (kg Cl/m$^3$ concrete) | Historic Mean, Converted (% mass binder) | *fib* Method (% mass binder) | Relative Difference in Means |
|---|---|---|---|---|---|
| Tidewater | 1.26 | 0.81 | 0.42 | 0.25 | 41% |
| Northern | 2.98 | 1.30 | 0.99 | 0.80 | 20% |
| Eastern Piedmont | 2.34 | 0.56 | 0.78 | 0.4 | 49% |
| Western Piedmont | 4.01 | 1.63 | 1.33 | 0.25 | 81% |
| Central Mountain | 3.40 | 2.16 | 0.72 | 0.35 | 69% |
| Southwestern Mountain | 4.73 | 2.19 | 1.57 | 0.30 | 79% |

**Table 10. Results of Analysis of Meteorological Data From Winter 2000-2001 Including Deicing Salt Application and Precipitation Results for *fib* Method for Surface Chloride Concentration Calculation**

| Region | *n*, No. of Salting Events per Year | $C_{s,i}$ average Amount of Chloride Spread per Year (g/m$^2$) (Williamson, 2007) | $h_{s,i}$, Amount of Water From Rain and Snow During Events (mm) | $C_{o,R}$, Average Chloride Concentration of Contaminated Water (g/L) |
|---|---|---|---|---|
| Tidewater | 12 | 38.22 | 220.99 | 0.173 |
| Northern | 29 | 742.22 | 223.36 | 3.323 |
| Eastern Piedmont | 11 | 90.04 | 132.74 | 0.686 |
| Western Piedmont | 12 | 37.37 | 237.67 | 0.157 |
| Central Mountain | 37 | 113.99 | 253.29 | 0.450 |
| Southwestern Mountain | 42 | 116.88 | 284.68 | 0.304 |

It is noted that the Northern Region had the highest volume of chlorides spread per year partially because the deicing salt used had a higher concentration of chlorides than the other regions.

## Determination of Additional Parameters

Table 11 provides all other necessary data to implement the *fib* methodology and the appropriate *fib*-suggested aging coefficient distribution parameters.

**Table 11. Distribution Parameters of All Other Required Random Variables**

| Variable/Material | | Distribution | Mean | Std. Dev. | Lower Limit | Upper Limit | Source |
|---|---|---|---|---|---|---|---|
| Regression Variable, $b_e$ (K) | | Normal | 4800 | 700 | - | - | *fib* (2006) |
| Transfer Function ($\Delta x$) (mm) | | - | 12.7 | - | - | - | Cady and Weyers (1983) |
| Critical Chloride Concentration, $C_{crit}$ (% wt. binder) | Plain Steel | Beta | 0.65 | 0.15 | 0.2 | 2 | *fib* (2006) |
| | MMFX Steel | Lognormal | 1.08 | 0.443 | - | - | Ji et al. (2005) |
| Aging Coefficient, $\alpha$ | PCC | Beta | 0.3 | 0.12 | 0 | 1 | *fib* (2006) |
| | Fly Ash Cement Concrete | Beta | 0.6 | 0.15 | 0 | 1 | |
| | Blast Furnace Slag Cement Concrete | Beta | 0.45 | 0.2 | 0 | 1 | |

## Evaluation of Modifications to the *fib* Methodology

### Numerical Solution Method Comparison

Histograms of the chloride concentration at the depth of the reinforcing MMFX steel (VDOT Reinforcement Class I, ASTM-1035) at 100 years are shown in Figure 6 for the case study bridge. The underlying samples were combined with samples from the critical chloride concentration distributions to determine a time to corrosion initiation and hence a probability of failure. The times to initiation were well represented by a gamma distribution (Bales, 2016). The *fib* model predicts a 4.7% probability of corrosion initiation ($p_f$) over 100 years; i.e., it predicts that 95% of the time, the Lynchburg bridge deck will reach a service life of 100 years before corrosion initiates. The *fib*-Numerical model predicts that 84% of the time, a bridge deck will reach a service life of 100 years.

Results in terms of probability of corrosion initiation ($p_f$) and reliability index ($\beta$) for each of the bridges included in the study are presented in Table 12. Failure probabilities of achieving a 100-year service life that would violate the threshold proposed in *fib* (10%) are also shown. Probabilities of initiation for Bridge 17 vary from those found for the case study bridge because of changes to the underlying distribution parameters. Case study values are provided in Table 3; values for all bridges are provided in Chitrapu (2017) and the implementation spreadsheets. All results are for MMFX steel. It is noted that none of the bridges using the *fib* model had unacceptable performance, and only one bridge had unacceptable performance using the *fib*-Numerical model.

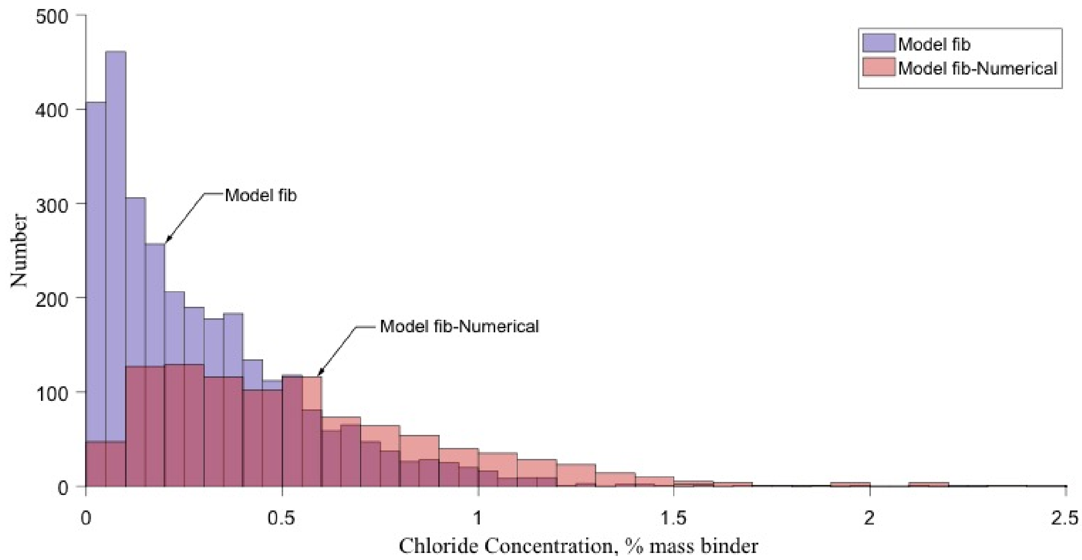


**Figure 6. Histogram of Predicted Service Life for Case Study Bridge**

**Table 12. Comparison of Failure Probabilities Between *fib* and *fib*-Numerical Models Considering Different Scenarios Involving the Time of Deicing Salt Application**

| Bridge | *fib* Model | | | | | | *fib*-Numerical Model | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | No Delay; Constant $C_{s,\Delta x}$ | | No Delay; Ramped $C_{s,\Delta x}$ | | No Delay; Constant $C_{s,\Delta x}$; Curing-Based $\alpha$ | | No Delay; Constant $C_{s,\Delta x}$ | | 12-Month Delay; Constant $C_{s,\Delta x}$ | |
| | $\beta$ | $p_f$ | $\beta$ | $p_f$ | $\beta$ | $p_f$ | $\beta$ | $p_f$ | $\beta$ | $p_f$ |
| Bridge 1 | inf | 0.0% | inf | 0.0% | 2.75 | 0.3% | 2.55 | 0.6% | 2.75 | 0.3% |
| Bridge 2 | 3.09 | 0.1% | inf | 0.0% | 1.98 | 2.4% | 2.30 | 1.1% | 2.49 | 0.6% |
| Bridge 3 | inf | 0.0% | inf | 0.0% | inf | 0.0% | 2.41 | 0.8% | 2.58 | 0.5% |
| Bridge 4 | 1.70 | 4.5% | 1.85 | 3.2% | 1.49 | 6.8% | 0.98 | **16.5%** | 1.22 | **11.1%** |
| Bridge 5 | 1.83 | 3.4% | 1.98 | 2.4% | 1.49 | 6.8% | 1.29 | 9.9% | 1.32 | 9.3% |
| Bridge 15 | 2.75 | 0.3% | inf | 0.0% | 2.41 | 0.8% | 2.03 | 2.1% | 2.12 | 1.7% |
| Bridge 16 | inf | 0.0% | inf | 0.0% | 2.75 | 0.3% | 2.45 | 0.7% | 2.58 | 0.5% |
| Bridge 17 | 2.03 | 2.1% | 2.14 | 0.5% | -- | -- | 1.55 | 6.1% | 1.60 | 5.5% |

inf = infinity.

Values in bold indicate failure probabilities of achieving a 100-year service life that would violate the 10% threshold.

## Variation in the Time to First Chloride Exposure (*fib*-Numerical Model)

Table 12 also compares the results from simulations with a 12-month delay to first chloride exposure to the default no-delay scenario (values at 1-, 2-, 3- . . . months delay were within these limits). Although the relative differences in probability of corrosion initiation were sometimes high, the absolute differences, which are what would matter in an actual assessment, were small. In no case did the consideration of a delay change a bridge from being assessed as inadequate with no delay to adequate with a 12-month delay.

## Ramp-Type Function for Surface Chlorides (*fib* Model)

Table 12 also compares the probability of corrosion initiation that would likely be obtained using a ramp function to that obtained using the default constant (time-invariant) value. It is emphasized that these results are based on the literature review rather than simulation. It was estimated that the effect of incorporating a ramping function for surface chlorides as opposed to using a constant concentration throughout the service life would induce only small variations, with two bridges changing from minimal to no probability of failure.

## Curing-Based Aging Coefficient (*fib* Model)

Table 12 also compares using aging coefficients derived from RCM tests versus the default diffusion-based aging coefficients recommended by *fib*. The curing-based aging coefficients produced marginally higher probabilities of failure.

# Sensitivity Analysis

## Multivariate Analysis Using FORM

### *Analysis on the Limit State Function for Depassivation*

Table 13 presents the sensitivity of each input parameter used in the limit-state function in terms of their importance factors, as obtained from FORM analysis (for Bridge 1). The aging coefficient ($\alpha$) is the most sensitive variable, followed by the critical chloride concentration ($C_{crit}$), and the surface chloride concentration ($C_{s,\Delta x}$). Random variables with negative importance factors can be interpreted as resistances or strengths (i.e., random variables that prevent failure), whereas positive importance factors indicate loads (i.e., random variables that promote failure). The absolute value of the importance factor is related to the underlying uncertainty in that variable.

These results indicate that a change in some aspects of the materials (aging and critical chloride content) has the greatest potential to change the probability of failure. Results for the other bridges were similar, with the second and third variable sometimes trading places. These results are in general agreement with the literature.

**Table 13. Sensitivities of Input Parameters in the Limit-State Function (Bridge 1)**

| Random Variable | Importance |
|---|---|
| $\alpha$ | -0.62 |
| $C_{crit}$ | -0.53 |
| $C_{s,\Delta x}$ | 0.46 |
| $T_{real}$ | 0.28 |
| *a* (cover depth) | -0.17 |
| $D_{RCM,0}$ | 0.11 |
| $C_0$ | 0.04 |
| $b_e$ | -0.002 |

*Analysis of the fib Surface Chloride Concentration Model*

The results of the sensitivity analysis of the surface chloride model are shown in Table 14. The number of salting events per year, *n*, was the most sensitive of the three variables. It is noted that the magnitude of these importance factors is small, as they were of less importance than most of the other random variables provided in Table 13.

**Table 14. Sensitivities of Input Parameters in the *fib*-Defined Equation to Compute Surface Chlorides (Bridge 1)**

| Random Variable | Importance |
|---|---|
| $n$ | 0.044 |
| $C_{R,i}$ | 0.012 |
| $h_{s,i}$ | 0.005 |

## Monte Carlo Convergence

Table 15 compiles results comparing the nominal number of simulations required based on an initial guess for the probability of corrosion initiation (from 1,000 simulations) to the number of simulations required to actually achieve convergence (i.e., to achieve a 10% confidence interval on the true probability of corrosion initiation). Because the initial estimate is imprecise, it may take more simulations than would be expected to converge on the true value. This table can be used to find the number of simulations required to achieve true convergence (right column) based on the order of magnitude of an initial guess (left column). However, given a threshold of acceptable performance of 10% probability of corrosion initiation, 5,000 to 10,000 simulations will generally be sufficient.

**Table 15. Monte Carlo Convergence Results for 10% Confidence Interval on Probability of Corrosion Initiation**

| Estimated $p_f$ (1,000 Simulations) | Nominal No. of Simulations Required | No. of Simulations to Achieve Convergence |
|---|---|---|
| 0.1% | 100,000 | 150,000 |
| 1.0% | 10,000 | 15,000 |
| 10.0% | 1,000 | 1,500 |

# Implementation of Service Life Prediction

## Implementation Using Plain and Corrosion-Resistant Steel

Results using the *fib* model to compare plain steel and MMFX steel are shown in Table 16. As expected, the probability of failure was much higher using plain steel and was unacceptable in the case of Bridge 4. The relatively high $p_f$ values for Bridges 4 and 5 (from both models) are likely due to the high 28-day RCM values of 704 mm$^2$/year and 568 mm$^2$/year, respectively. In addition, both bridges are in the Southwestern Mountain region, which has the highest surface chloride concentration among all regions.

Results using the *fib*-Numerical model were consistent in predicting higher failure probabilities with plain steel reinforcement. As expected, the numerical solution also predicted a higher probability of corrosion initiation than the error function model in all cases.

**Table 16. Comparison of Probabilities of Corrosion Initiation Using MMFX (VDOT Reinforcement Class I, ASTM 1035) and Plain Steel Reinforcement**

| Bridge | ***fib* Model** | | | | ***fib*-Numerical Model** | | | |
|---|---|---|---|---|---|---|---|---|
| | **MMFX** | | **Plain Steel** | | **MMFX** | | **Plain Steel** | |
| | $\beta$ | $p_f$ | $\beta$ | $p_f$ | $\beta$ | $p_f$ | $\beta$ | $p_f$ |
| Bridge 1 | inf | 0.0% | 2.58 | 0.5% | 2.55 | 0.6% | 1.97 | 2.5% |
| Bridge 2 | 3.09 | 0.1% | 2.51 | 0.6% | 2.30 | 1.1% | 1.71 | 4.3% |
| Bridge 3 | inf | 0.0% | 2.40 | 0.8% | 2.41 | 0.8% | 1.81 | 3.5% |
| Bridge 4 | 1.70 | 4.5% | 1.09 | **13.7%** | 0.98 | **16.5%** | 0.42 | **33.9%** |
| Bridge 5 | 1.83 | 3.4% | 1.37 | 8.5% | 1.29 | 9.9% | 0.72 | **23.6%** |
| Bridge 15 | 2.75 | 0.3% | 2.08 | 1.9% | 2.03 | 2.1% | 1.33 | 9.2% |
| Bridge 16 | inf | 0.0% | 2.51 | 0.6% | 2.45 | 0.7% | 1.68 | 4.7% |
| Bridge 17 | 2.03 | 2.1% | 1.65 | 5.0% | 1.55 | 6.1% | 0.98 | **16.5%** |

inf = infinity.
Values in bold indicate failure probabilities of achieving a 100-year service life that would violate the 10% threshold.

## Implementation for Case Study Bridge

As previously stated, the details of the case study bridge varied from Bridge 17; the estimated probability of corrosion initiation using the *fib* model was 4.7% with MMFX (VDOT Reinforcement Class I, ASTM 1035) and 14.3% with plain steel. For the case study bridge, the performance using plain steel was unacceptable.

### *Regional Comparison*

Table 17 provides estimates of the probabilities of corrosion initiation from the *fib* model for the case study bridge as built in each of the regions. The variables changed for each region were the temperature and the historical surface chloride concentrations. When the probability of corrosion initiation results from each region are ranked from highest to lowest, the ranking is more consistent with the historical surface chloride concentrations (Southwestern Mountain highest; Tidewater lowest) than with the temperature (Tidewater highest; Southwestern Mountain lowest). This finding further confirms the results of the sensitivity analysis, as surface chloride concentration affected the estimated probability of corrosion initiation more than the temperature.

**Table 17. Probability of Corrosion Initiation for Case Study Bridge If It Were Built in Each Region of Virginia**

| **Region** | **Probability of Corrosion Initiation, $p_f$** |
|---|---|
| Tidewater | 0.4% |
| Northern | 0.5% |
| Eastern Piedmont | 4.7% |
| Western Piedmont | 2.2% |
| Central Mountain | 4.7% |
| Southwestern Mountain | 7.1% |

*Life-Cycle Cost Analysis*

A gamma distribution was found to fit the results of the Monte Carlo simulation to obtain the Equivalent Uniform Annual Cost (EUAC) for both types of reinforcing steel, based on the service life predictions of the *fib* model. The resulting probability density functions are shown in Figure 7 (distribution parameters are provided by Bales [2016]). The sample mean EUACs were 0.0647 and 0.0582 for MMFX and plain steel, respectively. It is emphasized that potential changes to maintenance costs were not included in the analysis.

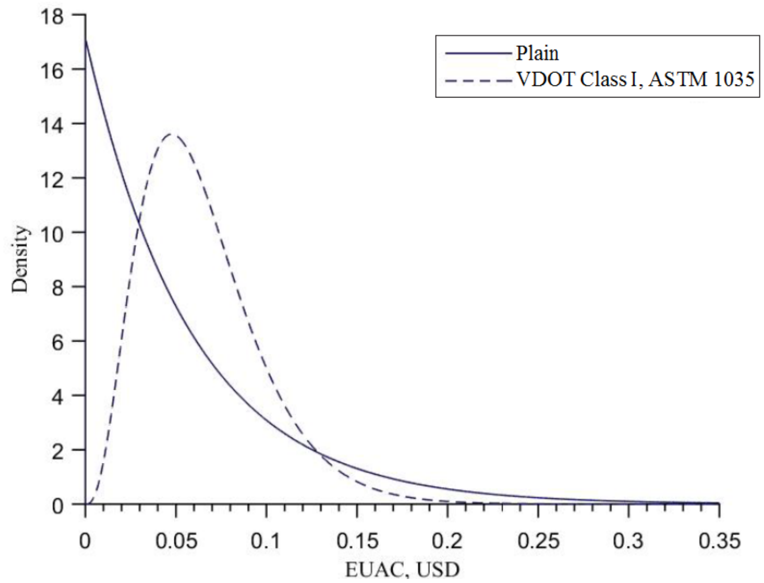


**Figure 7. Probability Density of Equivalent Uniform Annual Cost (EUAC) of Reinforcing Steel Based on *fib* Model**

# DISCUSSION

## Characterization

### Concrete Characterization

The standard deviations of initial chloride concentrations for different bridges were variable: the initial chloride concentration of the concrete from Richmond was an order of magnitude smaller than the results from the other bridges. Additional testing of concrete mixtures from different concrete suppliers used in Virginia and from different regions of the state would provide a useful database for service life modeling. However, given the low importance of this variable as identified by the sensitivity analysis, further characterization may not be required.

Completion of additional NT Build 492 tests for new bridge decks in Virginia would eventually provide a database of $D_{RCM,0}$ values for different suppliers and mixtures that could be used as a reference for service life prediction. That is, with sufficient samples, it would be possible to predict service life without performing the NT Build 492 test on samples for new bridge decks. Additional testing would also confirm the high consistency of RCM values found in this study, which would validate the use of a lower coefficient of variation than recommended by *fib*. In the meantime, it is suggested that the more conservative coefficient of variation between the sample value and the *fib* value of 0.2 be used. Given that there was no clear trend in the results with the number of samples, the use of three test cylinders at 28 days appears to be appropriate.

The commentary to Section B2.2.2.2 in *fib* Bulletin 34 alludes to the fact that the statistical quantification of the aging coefficient somehow takes into account factors other than curing, such as binding properties of the cement. *fib* Bulletin 76 (2015) provides two methods for quantifying the aging coefficient. The first requires taking cores from a large number of existing bridges and performing a regression on the RCM test results. The second can be used for new concretes but requires 1 to 2 years of diffusion testing. Given the high importance factor of the aging coefficient, determination of aging coefficients specific to Virginia should be given high priority if further work is to be done to implement service life prediction.

**Exposure Characterizations**

The regional description of annual temperature provided in this report should be adequate for implementation of the *fib* methodology. One limitation of the use of past meteorological data is that they do not capture any potential future changes in the annual temperature distribution.

There were significant differences between the historical surface chloride concentrations and those predicted using the *fib* methodology. There are several possible sources for the differences, including the following:

1. Implementing the *fib* model required an assumption of binder content for the historic bridges. Limiting the collection of historical data to bridges for which the binder content is known would provide a better basis for comparison.

2. VDOT deicing procedures likely changed over the service lives of the surveyed bridges and have definitely changed since the deicing salt application data were collected. In applying the *fib* methodology, assumptions were made for the temperature ranges for which deicing events were defined. A review of current procedures as adopted by the various districts would improve the accuracy of this part of the model.

3. Only a limited number of years were used to estimate both deicing salt application and the number of snow days. The collection of additional deicing application and meteorological data would provide a better statistical description.

4. The *fib* instructions for back calculating surface chloride concentration from the chloride concentration of contaminated solution are ambiguous. A different interpretation of these instructions could significantly change the results. Conducting ponding tests on low-cracking concrete would provide Virginia-specific functions for converting these values.

Although it is theoretically possible to implement the *fib* methodology for computing surface chloride concentrations, it is not currently advised because of the aforementioned limitations. As surface chloride concentration has a high importance factor, collection of additional deicing salt application data and performance of the ponding test would be expected to significantly increase the predictive capacity of the *fib* approach.

### Other Required Parameters

Ample data were available to characterize other required parameters, such as the temperature regression coefficient, $b_e$, and the critical chloride concentrations. Although many cover measurements of Virginia bridge decks have been taken over the years, a probabilistic distribution describing as-built cover depths as opposed to nominal values is not available. Arguments can be made that as-built cover depths would be biased high (contractor is conservative) or that they are likely biased low (it is difficult to control the cover depth). In the absence of additional information, using the nominal value as the mean and taking the coefficient of variation suggested by *fib* are recommended.

## Evaluation

The differences in the probability of corrosion initiation between the *fib* and *fib*-Numerical models illustrate the importance of properly accounting for the evolution of the apparent diffusion coefficient. Given that the *fib* aging sub-function and coefficient were obtained through regression on existing structures, it does not make sense to solve for chloride diffusion numerically using this sub-function. The aging coefficient takes into account multiple processes: a numerical approach should directly account for these processes, including chloride binding and curing. The numerical solution is, however, useful when other parameters are expected to vary with time (e.g., the surface chloride concentration or the temperature). Because the numerical solution for a delay in time to first chloride exposure did not find a significant impact, it is not currently recommended for considering delays. Similarly, given the negligible difference introduced by the use of the ramp-type function for surface chloride concentration, using a constant value is recommended. However, if a bridge is on the borderline of acceptable and unacceptable performance (e.g., 10.2%), it is likely that consideration of either delay or ramping would result in satisfactory predicted performance.

The importance factors computed in the sensitivity assessment capture both the characteristics of the limit state function and the distributions of each random variable. If a variable has a large standard deviation, it may have a high importance factor, due not to a particular sensitivity to that variable in the limit state function but rather to its large uncertainty. The opposite is true for variables with very small standard deviations, such as the RCM results for some mixtures. A critical finding of the sensitivity analysis was that the variables with the most conceptual uncertainty (aging coefficient and surface chloride concentration) were also the most influential. This finding further motivates additional data collection and testing.

Aging coefficients obtained from the RCM tests, although varying from the *fib*-recommended coefficients, were well within the expected range. However, the fits were highly uncertain. Because of the theoretical mismatch between coefficients based on short-term RCM data and long-term predictions of apparent diffusion, it is recommended that *fib*-specified aging coefficients be used until this parameter can be better quantified in Virginia.

## CONCLUSIONS

- *Sufficient data are available to implement the fib Bulletin 34 Model Code for Service Life Design in Virginia, with suggested default parameters obtained from the literature, fib, the results of this study, and future testing. Predictions of service life should be viewed with caution because of the high uncertainty associated with critical model parameters, including the aging coefficient and surface chloride coefficient.*

- *Chloride migration coefficients varied both with mix design (and supplier) and the spread of the test results. For this reason, it is suggested that test mixtures, rather than regional values, be used for each project.*

- *In the absence of Virginia-specific data, default aging coefficient values from fib are more appropriate than values obtained from the NT Build 492 test results, which account only for curing and are not appropriate for long timespans.*

- *Given changes in deicing salt application protocols and the limited data used to implement the fib approach for computing surface chloride concentration, the use of surface chloride concentrations obtained from surveys of Virginia bridge decks is suggested. Region-specific values for both surface chloride concentration and temperature can be used.*

- *At this time it is not appropriate to modify the fib model. Specifically, it is not supported to solve the diffusion partial differential equation numerically; to incorporate a ramp-type surface chloride concentration function; or to consider the delay to the first application of deicing salts.*

- *The use of low-cracking concrete and corrosion-resistant reinforcing bar, in combination with appropriate cover depths, can achieve the target 100-year service life according to the fib model. The use of these technologies was also found to be beneficial from a life-cycle cost perspective.*

## RECOMMENDATIONS

1. *VDOT's Structure and Bridge Division should work with VTRC to assess the NT Build 492 test method in relation to Virginia Test Method 112 and evaluate the reason behind the high variability in chloride migration across different regions in Virginia.* Results of the assessments of the bridges support the use of low-cracking concrete and corrosion-resistant reinforcement to achieve a 100-year service life. Since VDOT specifications do not require NT Build 492 testing, VDOT's Structure and Bridge Division should work with VTRC to consider evaluating and correlating the performance of concrete mixtures related to the chloride migration as determined by both current specifications (Virginia Test Method 112)

and the NT Build 492 test for the current concrete mixture specifications. In addition, the finding of high variability across regions in the consistency of the chloride migration coefficient results suggests that project-specific specifications and quality control / quality assurance procedures may be necessary to achieve the intended service life.

## BENEFITS AND IMPLEMENTATION

The results of service life assessments of the case study bridge and the eight other bridges indicated that the combination of low-cracking concrete and corrosion-resistant reinforcement (here, VDOT Reinforcement Class I, MMFX, ASTM 1035) can meet the demand of a 100-year service life according to the *fib* model. Given that the low-cracking concrete mixtures address precisely this concern, there is reason to believe that the service life estimates obtained using the *fib* methodology may be accurate.

The implementation of service life design by VDOT has the potential to increase, or guarantee, the longevity of bridge decks in Virginia. Bridge decks that meet their design targets reduce both life-cycle costs and disruptions to motorists. Although modern materials used by VDOT (including low-cracking concrete and corrosion-resistant steel) have been assumed to offer service in excess of 100 years, a fully probabilistic, quantitative approach such as that described in this study can ensure this level of performance. If it is found to be impractical to implement the fully probabilistic approach for all new structures, a next step would be to develop regional guidelines based on the data produced by this study and subsequent material characterization efforts.

The use of low-cracking concrete and corrosion-resistant reinforcing bar, in combination with appropriate cover depths, can achieve the target 100-year service life according to the *fib* model. The use of these technologies was also found to be beneficial from a life-cycle cost perspective. As a consequence, the implementation of the *fib* Bulletin 34 Model Code for Service Life Design in Virginia is not necessary. Given that VDOT is using low-cracking concrete and corrosion-resistant reinforcing bar, no further implementation is needed.

## ACKNOWLEDGMENTS

This research was sponsored by the Strategic Highway Research Program 2, Project R19A. The researchers thank Prasad Nallapaneni, Soundar Balakumaran, Kyle Haber, Michael Brown, Mike Bartholomew, Anne-Marie Langlois, and the numerous VDOT and VTRC personnel involved in the material testing. They also thank the Virginia Tech graduate research assistant committee members: Carin Roberts-Wollman, Ioannis Koutromanos, and Sean McGinnis. Finally, they thank Abraham Salomon for his editing services.

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