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16. Abstract

A model to determine the time to first repair and subsequent rehabilitation of concrete bridge decks exposed to chloride deicer salts that recognizes and incorporates the statistical nature of factors affecting the corrosion process is developed. The model expands on an existing deterministic model by using statistical computing techniques, including resampling techniques such as the parametric and simple bootstrap. Emphasis was placed on the diffusion portion of the diffusion-cracking model, but advances can be readily included for the time for corrosion deterioration after corrosion initiation.

Data collected from ten bridge decks built in Virginia between 1981 and 1994 were used to model the surface chloride concentration, apparent diffusion coefficient, and clear cover depth. Several ranges of the chloride corrosion initiation concentration, as determined from the available literature, were investigated. The time to first repair and subsequent rehabilitation predicted by the stochastic model is shorter than the time to first repair and subsequent rehabilitation predicted by the deterministic model. The stochastic model is believed to more accurately reflect the true nature of bridge deck deterioration because it takes into account the fact that data for each of the parameters affecting chloride diffusion and corrosion initiation are not necessarily normally distributed.

The model was validated by comparison of projected service lives of bridge decks built from 1981 to 1994 derived from the model to historical service life data for 129 bridge decks built in Virginia between 1968 and 1972. The time to rehabilitation predicted for the set of bridge decks built between 1981 and 1994 by the stochastic model was approximately 13 years longer than the normalized time to rehabilitation projected for the bridge decks built between 1968 and 1972 using historical data. The time to first repair and rehabilitation predicted by the probabilistic method more closely matches that of historical data than the time to first repair and rehabilitation predicted by the average value solution. The additional service life expected for the set of bridges built between 1981 and 1994 over those constructed from 1968 to 1972 can be attributed to the decrease in w/c ratio from 0.47 to 0.45 and slight increase in as-built cover depth from approximately 50 mm (2 in.). to 63.5 to 76 mm (2.5 to 3.0 in.)

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FINAL CONTRACT REPORT

A MODEL TO PREDICT THE IMPACT OF SPECIFICATION CHANGES ON THE CHLORIDE-INDUCED CORROSION SERVICE LIFE OF VIRGINIA BRIDGE DECKS

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ABSTRACT

A model to determine the time to first repair and subsequent rehabilitation of concrete bridge decks exposed to chloride deicer salts that recognizes and incorporates the statistical nature of factors affecting the corrosion process is developed. The model expands on an existing deterministic model by using statistical computing techniques, including resampling techniques such as the parametric and simple bootstrap. Emphasis was placed on the diffusion portion of the diffusion-cracking model, but advances can be readily included for the time for corrosion deterioration after corrosion initiation.

Data collected from 10 bridge decks built in Virginia between 1981 and 1994 were used to model the surface chloride concentration, apparent diffusion coefficient, and clear cover depth. Several ranges of the chloride corrosion initiation concentration, as determined from the available literature, were investigated. The time to first repair and subsequent rehabilitation predicted by the stochastic model is shorter than the time to first repair and subsequent rehabilitation predicted by the deterministic model. The stochastic model is believed to more accurately reflect the true nature of bridge deck deterioration because it takes into account the fact that data for each of the parameters affecting chloride diffusion and corrosion initiation are not necessarily normally distributed.

The model was validated by comparison of projected service lives of bridge decks built from 1981 to 1994 derived from the model to historical service life data for 129 bridge decks built in Virginia between 1968 and 1972. The time to rehabilitation predicted for the set of bridge decks built between 1981 and 1994 by the stochastic model was approximately 13 years longer than the normalized time to rehabilitation projected for the bridge decks built between 1968 and 1972 using historical data. The increase in time to rehabilitation for the newer set of bridge decks was attributed to a reduction in the specified maximum water/cement ratio and increase in clear cover depth between the two time periods.

The study shows the time to first repair and rehabilitation predicted by the probabilistic method more closely matches that of historical data than the time to first repair and rehabilitation predicted by the average value solution. The additional service life expected for the set of bridges built between 1981 and 1994 over those constructed from 1968 to 1972 can be attributed to the decrease in w/c ratio from 0.47 to 0.45 and slight increase in as-built cover depth from approximately 50 mm (2 in) to 63.5 to 76 mm (2.5 to 3.0 in).

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INTRODUCTION

According to the Federal Highway Administration, approximately 30 percent of the nation's bridges are either structurally deficient or functionally obsolete. It is estimated that approximately $90 billion will be required to rehabilitate or replace these bridges (Federal Highway Administration 2000a). In Virginia approximately 28 percent of the bridges on interstates, principal arteries, and major highways are considered structurally deficient or functionally obsolete (Federal Highway Administration 2000b). For many of these bridges, the concrete decks will need to be rehabilitated before other components of the bridge, and prior to the previous predicted design service life of 50 years. Chloride-induced corrosion of the reinforcing steel is known to be a major cause of premature rehabilitation of bridge structures.

Several methods have been used to protect the reinforcing steel from chloride corrosion attack in concrete bridge decks. The methods include low-permeability concrete to slow the ingress of chlorides, polymer overlays, and deck sealers; increased concrete cover depth; cathodic protection; and alternative reinforcement. The use of epoxy-coated reinforcement (ECR) is particularly prevalent in the United States (Babaei and Hawkins 1988). Previous work has demonstrated that the epoxy coating on ECR will begin to debond from the steel reinforcement in bridge decks in Virginia in as little as 4 years and most likely by 12 to 15 years (Weyers et al. 1998).

A better service life model would assist bridge engineers in two ways. First, the remaining time to first repair and subsequent rehabilitation could be estimated with greater accuracy for a given bridge or set of bridges. Second, the effectiveness of the various protection methods could be compared and evaluated.

Service life models have been used for many years (Gannon 1998). One common service life model for the chloride-induced corrosion of reinforcing steel in concrete involves two time periods and is presented in Figure 1. The first is the time for chloride diffusion to a concentration to initiate corrosion. The second is the time for corrosion damage to the end of functional service life (Weyers et al. 1993). Based upon a survey of engineers in several state departments of transportation conducted under a Strategic Highway Research Program (SHRP) study, the end of functional service life is reached when approximately 12 percent of the worst span lane of a bridge deck has deteriorated (Fitch et al. 1995). The time to first repair is reached when 2.5 percent of the worst span lane of a bridge deck has deteriorated (Weyers et al. 1993).

An apparent diffusion process, based on Fick's second law, can be used to model the time for chloride to reach and initiate corrosion at first repair and rehabilitation reinforcing steel depths. When solved for the condition of constant surface chloride and a one-dimensional infinite depth, Fick's second law takes the following form (Crank 1975):

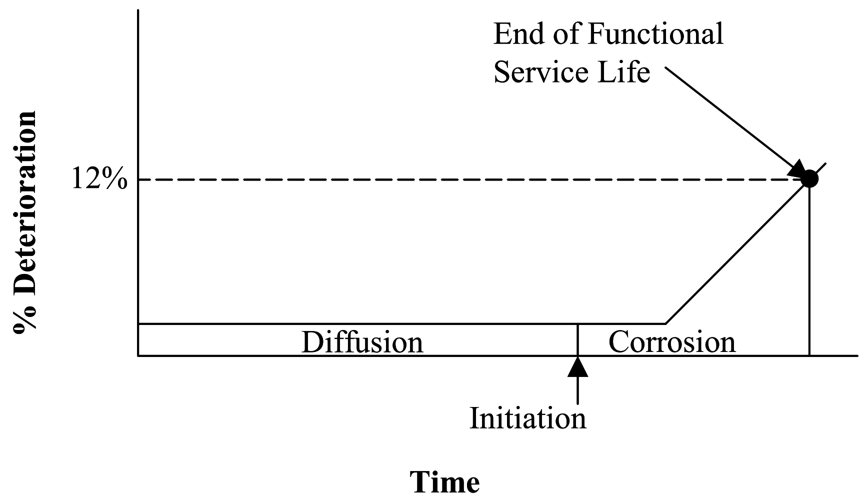


Figure 1 – Service Life Model

$$C_{(x,t)} = C_o(1 - erf\frac{x}{2\sqrt{D_c t}}) \tag{1}$$

Where:

	$C_{(x,t)}$	=	chloride concentration at depth and time,
	C_o	=	surface chloride concentration,
	D_c	=	apparent diffusion coefficient,
	t	=	time for diffusion,
	x	=	concrete cover depth, and
	erf	=	statistical error function.

When $C_{(x,t)}$ is set equal to the chloride corrosion initiation concentration, and Equation 1 is solved for t, the time for diffusion of chloride ions to the chloride corrosion initiation concentration can be determined. However, for a given bridge deck, the values of $C_{(x,t)}$, C_o, D_c, and x are random variables, each with its own statistical distribution, mean, and variance. A solution to Equation 1 for the time for diffusion should include the probabilistic nature of the input variables.

The time for corrosion damage to the end of functional service life is likely a random variable as well and depends on the corrosion rate, concrete cover depth, reinforcing steel bar spacing, and size (Liu and Weyers 1998). However, little is known about the possible distribution of the time for corrosion damage; therefore, in this research, the value is taken as a point estimate. A concurrent research project seeks to determine the time for corrosion damage to the end of functional service life for both bare and epoxy-coated reinforcement, and results from the study can be readily incorporated into this work (Brown 2000).

Because the input parameters in the service life model are random variables, a statistical tool is required to determine the service life. One tool commonly used to solve statistical problems is Monte Carlo simulation. Monte Carlo is a general class of repeated sampling methods where a value is randomly sampled from theoretical distributions, for example $C_{(x,t)}$, C_o, D_c, and x. Then, a mathematical model is solved for the desired response. The entire process is repeated a sufficient number of times to define a distribution of the response. In this case, the solution to Equation 1 for the time for diffusion is added to the time for corrosion damage to end of functional service life a sufficient number of times to define a distribution of the service life. A related resampling method called bootstrapping uses the same repetitive sampling procedure but uses data to define the parameters for the distributions or samples directly from the existing data. Two types of the bootstrap, the parametric and simple bootstrap, were used in this research.

For a given bridge deck, values of C_o and D_c can be determined from chloride concentration profiles measured through the depth of the deck, and values of x can be measured using a rebar locator. It is best to measure the chloride contents directly over the reinforcing steel to at least partially or wholly account for the presence of the reinforcing steel (Kranc et al. 2001). Here it is noted that Equation 1 is the solution for a one-dimensional analysis for infinite depth, whereas the presence of the reinforcing steel has been shown to significantly influence the rate of chloride increase at bar locations (Kranc et al. 2001). The sample populations obtained from field data can then be used as the basis for the simulations. The distribution of the corrosion initiation concentration of chloride, $C_{(x,t)}$, is unknown. A range of approximately 0.6 to

5.5 kg/m^3 (1.0 to 9.2 lb/cy) has been commonly reported in the literature (Glass and Buenfeld 1997; Vassie 1984; Stratfull et al. 1975; Matsushima et al. 1998).

A more in-depth discussion of the topics in this section has been presented elsewhere (Kirkpatrick 2001).

PURPOSE AND SCOPE

Service life models for chloride-induced corrosion of steel-reinforced concrete structures consist of a chloride diffusion period to corrosion initiation followed by a corrosion period from initiation to cracking and spalling of the cover concrete. Deterministic models typically use mean parameter values for the chloride corrosion initiation concentration, surface chloride concentrations, apparent chloride diffusion constant, steel cover depth, and the corrosion time period. However, for a given structural component, or structural component(s) within a system, all of these model parameters are random variables.

The primary objective of this research was to utilize information from this and previous phases of the ongoing study on ECR to apply a model for the projection of service life and time to first repair of bridge decks in Virginia, with and without ECR, using statistical computing techniques.

A secondary objective was to validate the model using historical service life data for 129 bridge decks built in Virginia between 1968 and 1972.

The scope of the study includes data from a geographically representative sample of Virginia's bridge decks. Apparent chloride diffusion coefficients, apparent surface chloride contents, and cover depths were determined using field data from 10 bridge decks. A possible distribution of the chloride corrosion initiation concentration was determined from the literature.

METHODS AND MATERIALS

This study utilizes data from bridge decks in Virginia to incorporate probabilistic considerations into a service life model. The data to be used in this research project were collected from 10 geographically diverse bridge decks in Virginia. Figure 2 shows the location of the bridges in Virginia that were selected for the project. The bridge decks were chosen at random from a larger group of structures meeting geographic and specification requirements and were to be used in a research project to determine the effectiveness of ECR.

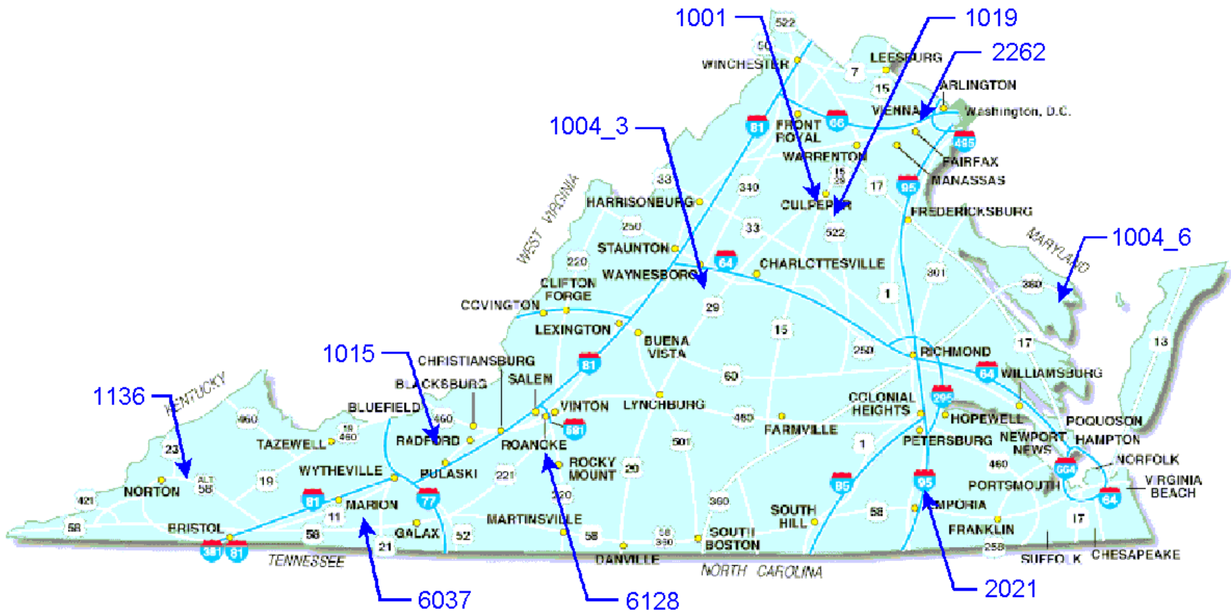


Figure 2 – Location of Bridge Decks Used in This Study

The bridge decks chosen in the study ranged in age from 4 to 18 years (at the time of data collection). Eight of the 10 bridge decks were constructed with ECR, and two were constructed with bare reinforcement. The district, structure number, age, and reinforcement type of the bridges are summarized in Table 1. The decks were all constructed under the same specification. The specified water/cement ratio for the decks is a maximum of 0.45, and the specified cover depth is 63.5 to 76 mm (2.5 to 3.0 in). The previous specification included a maximum water/cement ratio of 0.47 and a cover depth of approximately 50 mm (2 in). Both specifications required a 28-day compressive strength of 27.6 MPa (4,000 psi). The incorporation of ECR into the specification occurred at approximately the same time as the specified reduction in water/cement (w/c) ratio.

Table 1 – Summary of Bridge Decks

District	Structure Number	Year Built	Age at Sampling (Years)	Reinforcement Type
7 - Culpeper	1001	1992	7	ECR
6 - Fredericksburg	1004	1993	6	ECR
3 - Lynchburg	1004	1983	16	ECR
2 - Salem	1015	1987	12	ECR
7 - Culpeper	1019	1990	9	ECR
1 - Bristol	1136	1995	4	ECR
5 - Suffolk	2021	1981	18	ECR
9 - Northern Virginia	2262	1985	14	ECR
1 - Bristol	6037	1983	16	Bare Steel
2 - Salem	6128	1981	18	Bare Steel

Data

The data for use in this project were collected during this and previous studies of the effectiveness of ECR. The field investigations included cover depth measurements, field drilled cores, and powder samples for chloride content analysis. The data used in this study are summarized elsewhere (Kirkpatrick 2001).

Cover Depth

Approximately forty cover depth measurements were taken on each span of the bridge decks using a Profometer 3 electromagnetic cover depth meter. Typically, the bridge decks consisted of three spans. A total of 120 measurements were taken from each bridge (Pyc, 1998). Because of restrictions in traffic control and safety considerations, all of the measurements were taken from the right traffic lane. Also, experience has shown that the worst span lane is typically in the right traffic lane.

Time of rehabilitation of a corroding bridge deck has been associated with damage as cracks, spalls, delaminations, and patches over approximately 12% of the surface area of the worst span-lane (Fitch 1995). Because reinforcing that is closer to the surface of the concrete will be the first to suffer chloride-induced corrosion, the worst span lane coincides with the span lane with the lowest cover depth. Therefore, the first 12% deck area to incur damage is associated with the shallowest 12% cover depths over the reinforcement. Because cover depths in concrete bridge decks have a distribution, the span lane with the lowest cover depth cannot be decided by the mean value alone. The standard deviation of the cover depth distribution must also be taken into account. The 12th percentile value of the cover depth is influenced by both the mean and the standard deviation. With this in mind, the 12th percentile values of the cover depth were compared for each span lane. The cover depth measurements used in the service life prediction model were the ones corresponding to the span lane with the lowest 12th percentile cover depth measurement. Therefore, approximately 40 cover depth measurements are available for each of the 10 bridge decks. For this simulation, the cover depths are normally distributed (Weed 1974; Pyc 1998).

Powdered Samples for Chloride Content Analysis

Powdered samples to be used for chloride content analysis were extracted from three locations on each of the bridge decks. The samples were extracted adjacent to a reinforcing bar and in 12.7 mm (0.5 in) increments to a depth of approximately 76.2 mm (3 in). Powder removed from the upper 6.4 mm (0.25 in) of the deck was discarded. The specified maximum nominal diameter of the coarse aggregate used in the bridge decks was 25.4 mm (1 in) (VDOT 1991). However, the actual maximum size of aggregate is typically 19 mm (0.75 in). A drill bit with a diameter 1.5 times the maximum aggregate size is recommended for extraction of powdered samples. Therefore, a 28.6 mm (1.125 in) hollow core drill bit with a vacuum collection device was used to collect the powder at each incremental depth.

The samples were then analyzed for the acid soluble chloride concentration using the silver nitrate titration method (ASTM C1152-90). The diffused chloride concentrations were adjusted by subtracting the background chloride concentrations.

Cores

In addition to powdered samples, cores were drilled from the bridge decks to be used in the time to cracking study currently underway. Approximately 12 cores were removed from locations on each deck where no cracks were observed. Approximately 3 cores were removed from locations on each deck where cracks were observed. The cores were marked and stored to preserve the in-field moisture condition.

Although the primary use of the field cores was to determine the time to cracking of bridge decks constructed with ECR, as compared to bare reinforcement, a chloride content analysis was performed on concrete removed from two locations on each core. Chloride sampling depths directly above the reinforcing bar were 12.7 mm (0.5 in) below the top surface of the core and 19 mm (0.75 in) above the top reinforcing steel. The samples were 12.7 mm (0.5 in) thick. Figure 3 shows the field core chloride content sample locations. The cores were dry cut to prevent leaching out of the chlorides due to wet cutting.

Each partial core disk sample was crushed and ground in a two-step process. First, the half-disks were placed in a roll crusher to reduce the concrete to pebble-size particles. Then, the samples were reduced to a powder suitable for use in chloride content testing using a hammer crusher. Between crushings, the roll and hammer crushers were cleaned with compressed air and ethyl alcohol to prevent cross contamination of the samples. In addition, care was taken to minimize the amount of material that was lost in the grinding process.

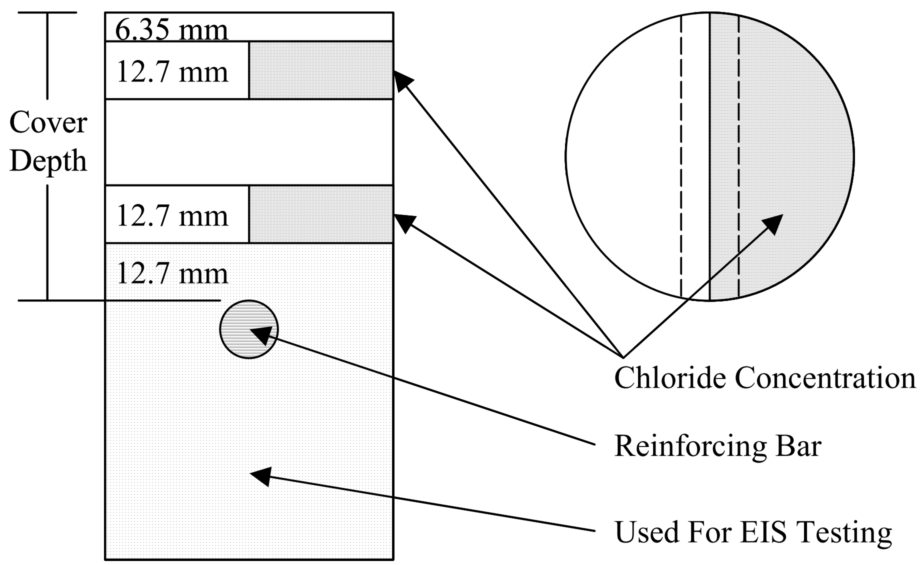


Figure 3 – Location of Chloride Samples from Field Cores

The powdered samples removed from the concrete cores were also analyzed for the acid soluble chloride concentration according to ASTM C 1152-90, which were adjusted for the background chloride content.

Surface Chloride Concentration

For bridge decks, the surface chloride concentration, C_o, is commonly taken as the concentration of chlorides located 12.7 mm (0.5 in) below the deck surface (Weyers et al. 1993). For the field-drilled powdered and core samples, the first sample removed from the deck was centered at 12.7 mm (0.5 in) below the surface, and therefore represents C_o. Therefore, 3 C_o values were available from the field-drilled powdered samples and between 4 and 12 C_o values were available from the cores for each bridge deck. The surface chloride concentration was best described by a gamma distribution (Zemajtis 1998).

It is important to note that, according to the model, if the value of C_o is smaller than the chloride initiation concentration, corrosion will not take place on the bridge deck.

Apparent Diffusion Coefficient

The apparent diffusion coefficient, D_c, is back-calculated from each set of chloride concentration measurements obtained from the bridge decks. A minimum sum of square error procedure, as described by Weyers (1993), was used to back-calculate D_c from the measured chloride profiles.

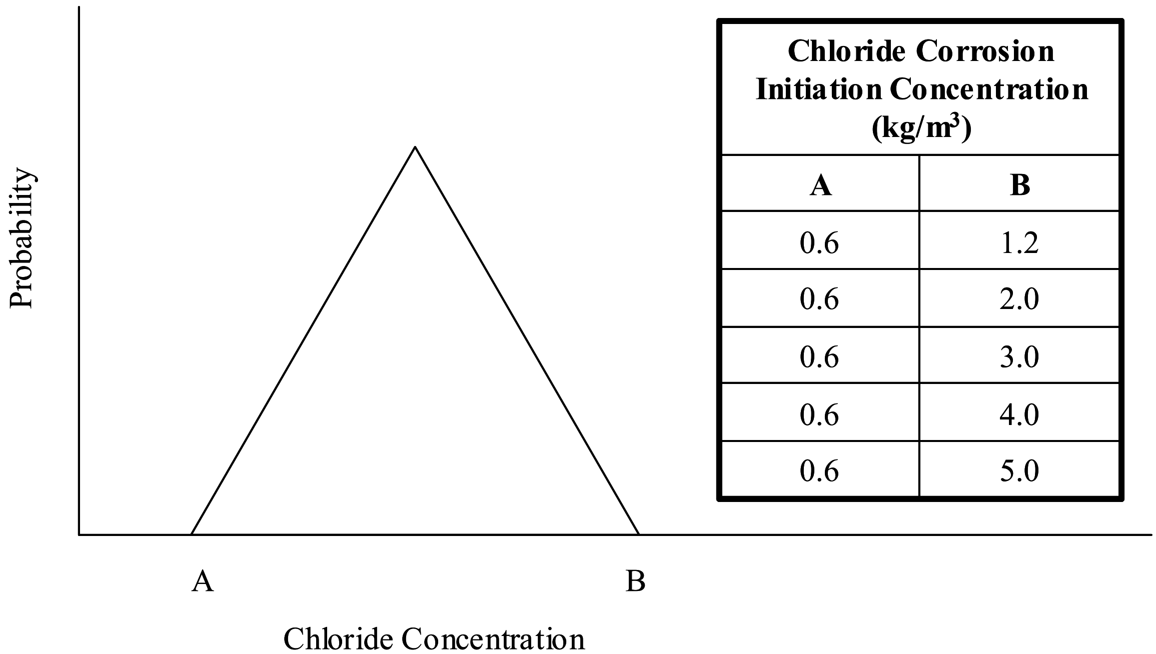
Using the measured C_o value and one deeper chloride value, a trial D_c was back-calculated from Equation 1. Then, using the trial D_c, Equation 1 was plotted against the actual chloride profile. The sum of square errors between the actual chloride profile and the chloride concentration predicted by the trial D_c was determined. Then, using an iterative procedure, the trial D_c was altered. A new sum of square error was calculated for each new trial D_c. Finally, the D_c value that predicts the chloride concentration with the minimum sum of square errors, when compared with the measured chloride profile, was taken as the best apparent diffusion coefficient. A computer program developed in a previous phase of this study was used to calculate D_c values for the field-drilled powdered samples (Zemajtis 1998). The computer program was validated with beta versions of two programs currently in development at other institutions. After correcting for discrepancies in the boundary condition assumptions, all three programs provided similar results.

For the core samples, only two chloride concentration measurements were available to back calculate an apparent diffusion coefficient. In this case, back calculating D_c from Equation 1 represents an exact fit of the two measured chloride values and the best fit given the limited chloride data available. However, the calculated D_c may not produce the minimum sum of square errors and may not represent the true apparent diffusion process of the bridge deck. Therefore, D_c values for a given bridge, calculated from two chloride concentration measurements, were carefully compared to D_c values calculated using the minimum sum of square errors procedure and a full chloride profile. D_c values calculated using two chloride measurements that were significantly out of the range of D_c values calculated using a full chloride profile were discarded.

The same number of D_c and C_o values was available for each bridge deck. The apparent diffusion coefficient was best described by a gamma distribution (Zemajtis 1998).

Chloride Corrosion Initiation Concentration

As discussed in the introduction section, an exact concentration of chlorides necessary to initiate corrosion of the reinforcing steel is possible but is a variable, which depends on several factors. The range typically reported in the literature is from 0.6 to 5.5 kg/m^3 (1.0 to 9.2 lb/cy) (Glass and Buenfeld 1997). No indication is given as to the shape of the distribution of the chloride initiation concentration. The authors' experience and available literature suggest that the shape may be weighted toward the center of a range of values. Since the initiation concentrations within the range have not been definitively found to follow a normal or other particular distribution, a distribution with a triangular shape was used as a best estimate for this simulation. Also, because there is a lack of agreement in the literature about the range of values of initiation, the time for diffusion was determined using several ranges of initiation. The lower limit of all ranges was 0.6 kg/m^3 (1.0 lb/cy). The upper limits were set at 1.2, 2, 3, 4, and 5 kg/m^3 (2.0, 3.3, 5.0, 6.7, and 8.3 lb/cy). Figure 4 presents the distributions of the chloride corrosion initiation concentration used in this study.



Chloride Corrosion Initiation Concentration (kg/m^3)	
A	**B**
0.6	1.2
0.6	2.0
0.6	3.0
0.6	4.0
0.6	5.0

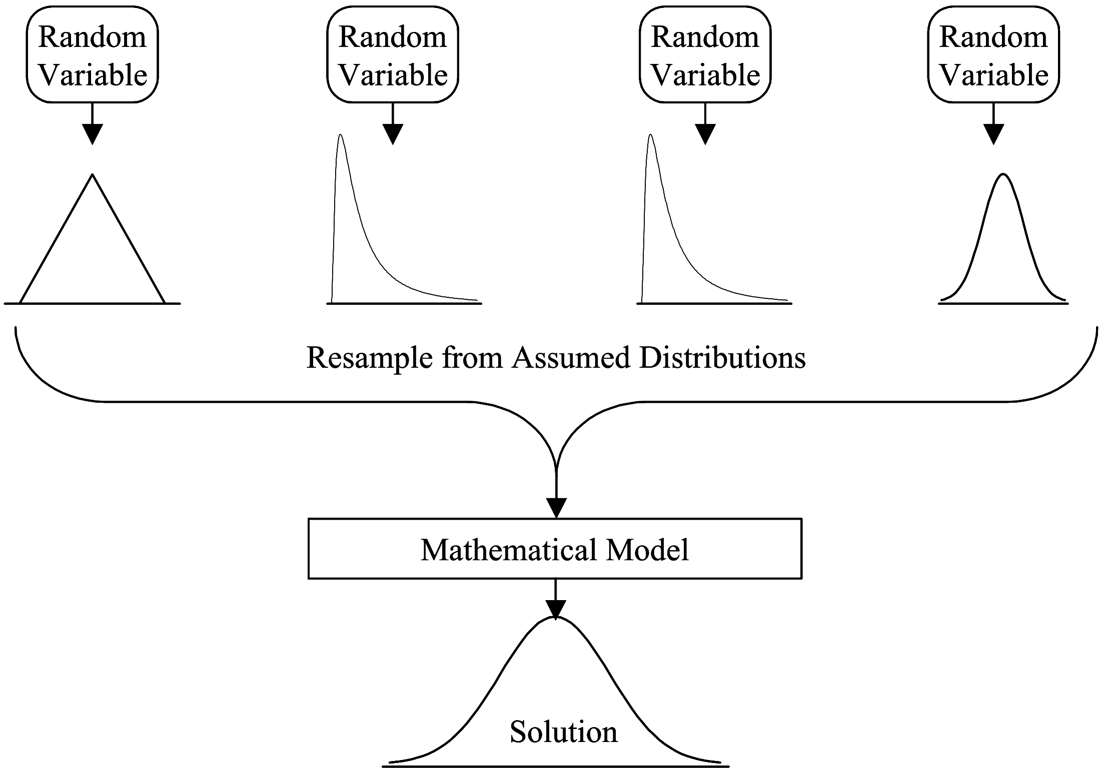
Figure 4 – Probability Distribution of Chloride Corrosion Initiation Concentration

Time to Corrosion Damage

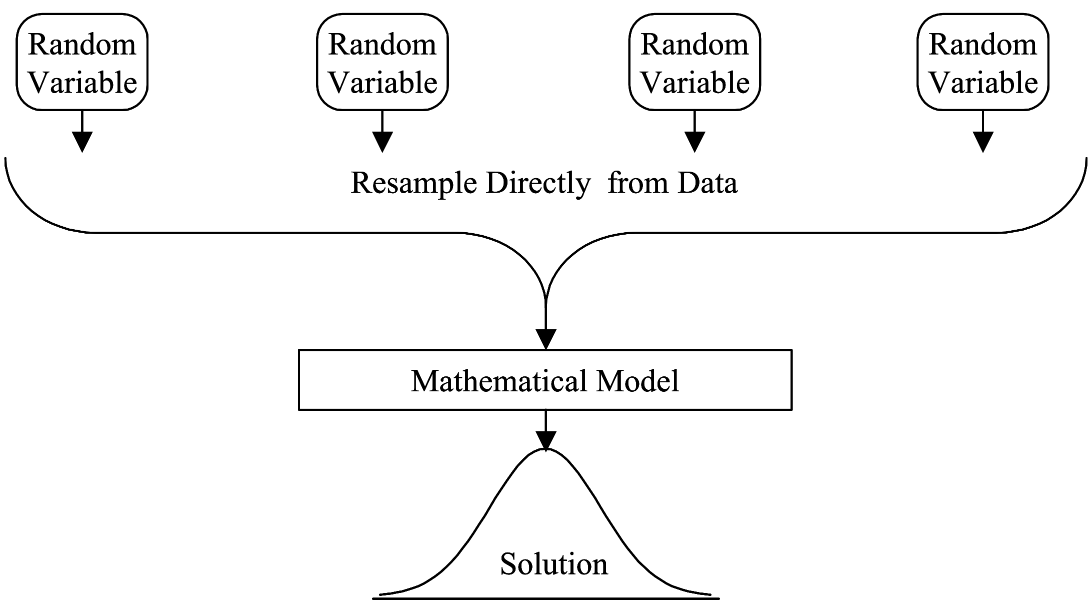
The time for corrosion deterioration after initiation is currently under debate among researchers. Although the time for corrosion damage for bare bar is generally accepted to be approximately 4 to 6 years, less is known about the time for corrosion damage for ECR (Weyers et al. 1994; Cady and Weyers 1984). Field studies have estimated 1 to 7 additional years for the time to corrosion damage for ECR (Covino et al. 2000; Clear 1998; Weyers et al. 1997). The focus of this study was not on the time to corrosion damage; rather it was on incorporating probabilistic considerations into the service life model, with particular emphasis on the time for diffusion. Therefore, a single point estimate for the time for corrosion of 4 years will be used for both bare bars and ECR. As the time for corrosion damage is better determined, the results can be easily incorporated into the service life model.

Simulation

The data described were used in a statistical simulation that will provide a stochastic solution for the predicted service life. To provide confidence in the simulation, the solution was obtained using two resampling techniques. The first was the parametric bootstrap, and the second was the simple bootstrap. Both techniques are part of a larger class of statistical resampling techniques generally known as Monte Carlo. The parametric bootstrap uses the sample data to determine parameters for a known distribution that is assumed to best fit the total population. During the resampling process, values are randomly sampled from the population distribution for each input



Parametric Bootstrap



Simple Bootstrap

Figure 5 – Parametric and Simple Bootstrap

variable. The simple bootstrap assumes that the distribution of the population is the same as the distribution of the sample. In other words, during resampling, the values of the sample data are sampled directly for each input variable, and the population is not assumed to fit a known distribution. Figure 5 presents the process for the parametric and simple bootstrap. The use of Monte Carlo techniques has been significantly enhanced by modern computing power, since a large number of iterations must be performed to obtain sufficiently descriptive results. Several computer packages capable of performing these simulations are available today. The statistical package S-Plus 2000, developed by MathSoft, was selected for this simulation because of its flexibility, power, and speed.

S-Plus

S-Plus is a user programmable software package with many powerful built-in functions that are geared toward data exploration and statistical simulations. The program runs on either a UNIX or PC platform. On the PC platform, a graphical user interface is provided. Functions created by the user are stored and can be accessed just like built-in routines. S-Plus is unique because it combines the power of built-in statistical resampling techniques with basic mathematical manipulation. Therefore, the time for diffusion and service life determined from Equation 1 can be solved directly from within S-Plus. S-Plus 2000, Professional Release 3 for the PC platform, was used in this study.

Simulation Routine

The simulation routine created for this project performs both the parametric and simple bootstrap and reports the results for each. The numerical results reported by the routine can be used to generate descriptive graphs and summary statistics.

For a given bridge or set of bridges, the input parameters for the simulation routine include field data for x, C_o, and D_c; the time to corrosion deterioration after initiation; the range of the expected chloride initiation concentration; and the number of iterations. The basic routine is presented in Figure 6 and is identical for both the parametric and simple bootstrap, except in the parametric bootstrap, data are generated from the assumed distributions with optimal parameter values estimated from the data. Both methods were considered to test how dependent the results were on distributional assumptions. The basic outline of the parametric bootstrap is described first, and then differences in the simple bootstrap are highlighted.

Parametric Bootstrap

For the parametric bootstrap, the field data for x, C_o, and D_c must be used to determine the appropriate parameters for the distribution assumed to represent the population. The distribution of the cover depth has been shown to be normal. The parameters required to define the shape of a normal distribution are the mean and variance, or standard deviation. Therefore, the mean and standard deviation of the cover depth are calculated and used to define the appropriate normal distribution that matches the field data for a particular bridge or set of bridges.

Based on quantile-quantile plots, which test the appropriateness of different distributions based on the observed data, the surface chloride concentration and apparent diffusion coefficient are best described by a gamma distribution. The gamma distribution is described by two parameters: the shape and the rate (Bury 1999). The definitions of the shape and rate differ slightly depending on the source and mathematical formulation of the gamma distribution. However, in S-Plus, the rate is equal to the mean over the variance and the shape is equal to the mean times the rate (S-Plus 2000).

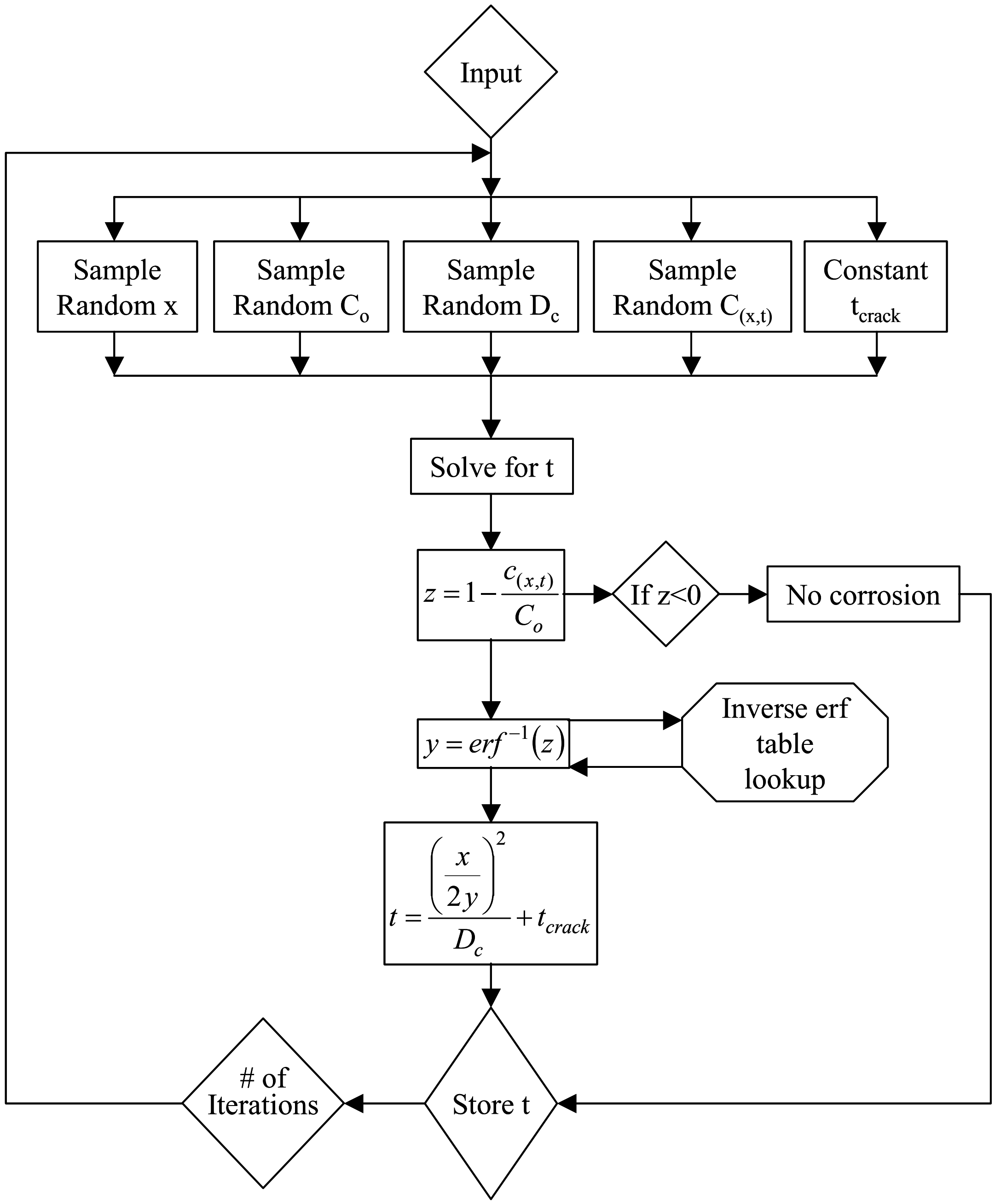


Figure 6 – Schematic of Simulation Routine

Once the distributions of x, C_o, and D_c have been determined based on the field data, the routine uses a random number generator to sample from each of the distributions of x, C_o, D_c and the chloride initiation concentration (already defined as triangular in shape). The number of sets of values sampled from each distribution is equal to the number of iterations specified by the user.

The next step in the routine is to solve for the time for diffusion in Equation 1. Equation 1 can be rearranged such that the time for diffusion is expressed as a combination of x, C_o, D_c, and $C_{(x,t)}$. Then, a time for diffusion is calculated for each set of input variables. For example, the fifth randomly sampled x, C_o, D_c, and $C_{(x,t)}$ are used to solve for the fifth estimate of time for diffusion. The total number of estimated diffusion times is the equal to the number of iterations.

The process up to this point is relatively straightforward. However, the solution of Equation 1 for the diffusion time requires the inverse of the error function (erf). Tables for the error function are readily available, but no simple mathematical expression is available (Crank 1975). Although table lookups are generally inefficient in computer programs, the vector-based programming nature of S-Plus makes table lookups relatively efficient. Therefore, a simple table lookup was used to solve for the inverse of the error function.

It should be noted that when the surface chloride concentration is lower than the chloride initiation concentration, the resulting time for diffusion is undefined. In other words, chlorides will never be present at the bar depth in sufficient concentration to initiate corrosion. No time for diffusion was calculated for iterations of the routine that produce a surface chloride concentration less than the initiation concentration (no corrosion predicted). Instead, the number of iterations that predicted no corrosion was counted and reported as a percentage of the number of iterations.

The routine had then randomly sampled each input variable and solved for the time for diffusion for a specified number of iterations. For each iteration, the service life was calculated by adding the time for diffusion to the time for corrosion (also specified in the routine's input). The result was a distribution of the service life for the bridge or set of bridges.

A bridge deck is estimated to be at the end of functional service life when 12 percent of the worst span lane has deteriorated. Likewise, the time to first repair is often defined as the time it takes for 2.5 percent of that span to deteriorate. These conditions are represented in the probabilistic service life model as the 12^{th} and 2.5^{th} percentile values of the distribution of the service life, respectively.

The routine looks at the distribution of simulated service life values and estimates the 2.5^{th} and 12^{th} percentiles based on the matching fraction of values. The full list of calculated service life estimates is available for creating other summary statistics or descriptive graphs. The results of several intermediate steps are also available for diagnostic testing.

Simple Bootstrap

The procedure for the simple bootstrap is identical with that of the parametric bootstrap except that no distributions are fitted to the field data. Instead, the routine samples directly from the field data for each of the input variables x, C_o, and D_c, a number of times equal to the specified number of iterations. The distribution of the chloride initiation concentration is still assumed to be triangular in shape.

The routine runs both the simple and parametric bootstraps from the same set of input parameters and returns the results of both the simple and parametric bootstrap simultaneously.

RESULTS

In the previous section, a method that incorporates the probabilistic nature of chloride diffusion and corrosion initiation in bridge decks was developed to predict the time to first repair and subsequent rehabilitation of concrete bridge decks subjected to chloride-induced corrosion of the reinforcing steel. In this section, the results of further development of the model in predicting the service life of 10 Virginia bridge decks are presented.

Number of Iterations

Before the model could be used to predict the service life of real structures, the appropriate number of iterations required for the model to provide precise results had to be determined. In this case, the precision of the results represents the range of predicted service life estimates expected for successive runs of the model with the same input variables. The appropriate number of iterations is a compromise between the time required to run the simulation and the precision of the results. As the number of iterations increases, the width of confidence intervals for predicted service life estimates decreases for a given set of input values.

One bridge was arbitrarily selected to determine the appropriate number of iterations for this simulation, optimizing precision against processing time. The service life was estimated 20 times each for numbers of iterations ranging from 10 to 100,000. The resulting service life estimates were used to generate the 95 percent confidence intervals for the predicted time to repair and rehabilitation for each number of iterations. The data from Bridge 1001 were used for the calculations. Figure 7 and Figure 8 show the converging behavior and tighter confidence intervals for increasing numbers of iterations.

Based on the results of these trials, 10,000 iterations were chosen for this simulation because of the good balance between the precision of the estimate and the time to run the simulation. The same number of iterations was chosen for a similar project involving bridge decks in Pennsylvania (Gannon 1998).

Sensitivity of the Model

To interpret the results of the simulation, it is useful to understand the sensitivity of the predicted time to first repair and rehabilitation to variations in each input parameter. The sensitivity indicates the expected change in the predicted time to first repair and rehabilitation associated with a change in the input variable. The sensitivity of each input variable on the time for diffusion to corrosion initiation was investigated. Because probabilistic considerations are not currently included in the post-initiation deterioration portion of this model (see page 10), the sensitivity of the input parameters was not investigated for the time of corrosion deterioration.

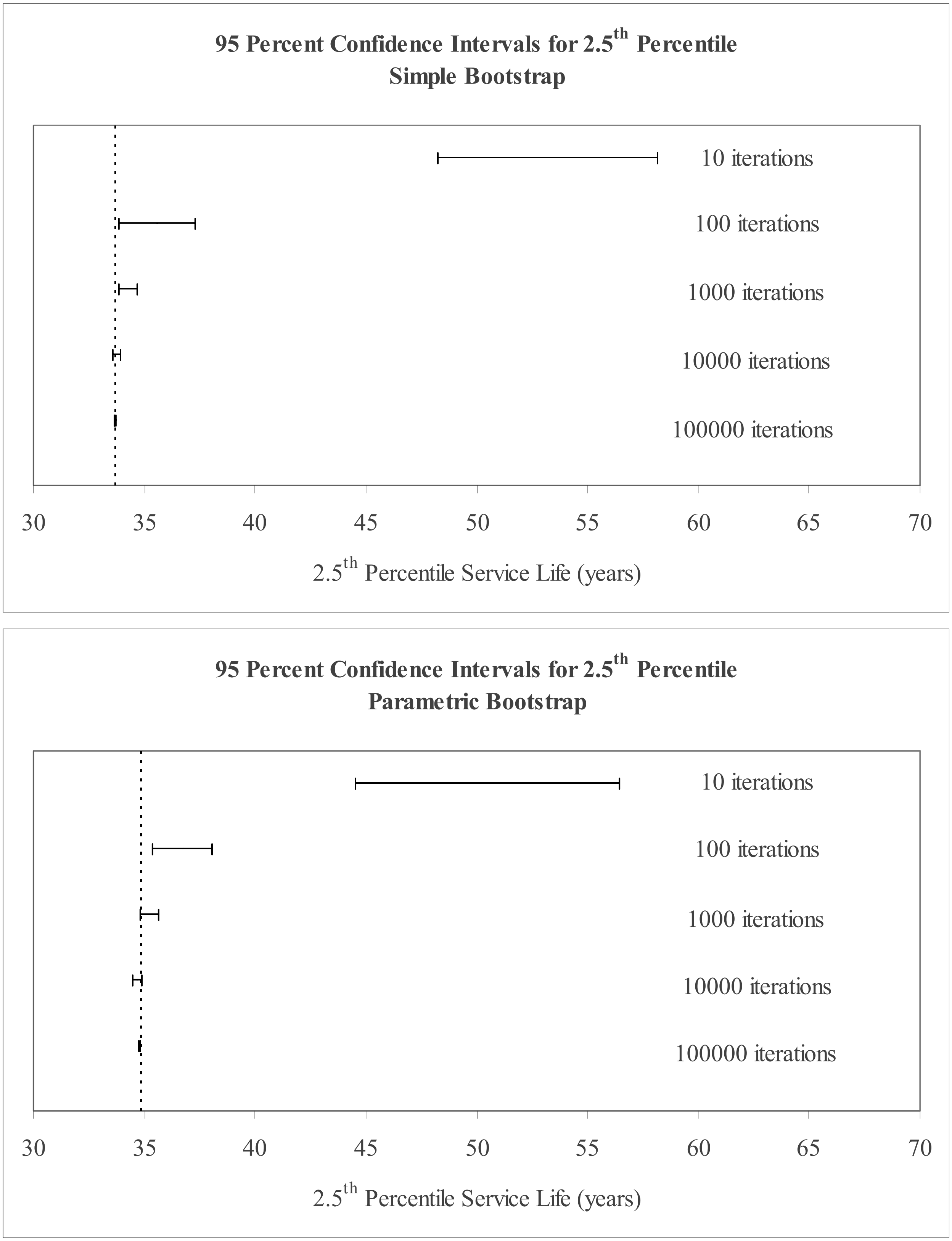
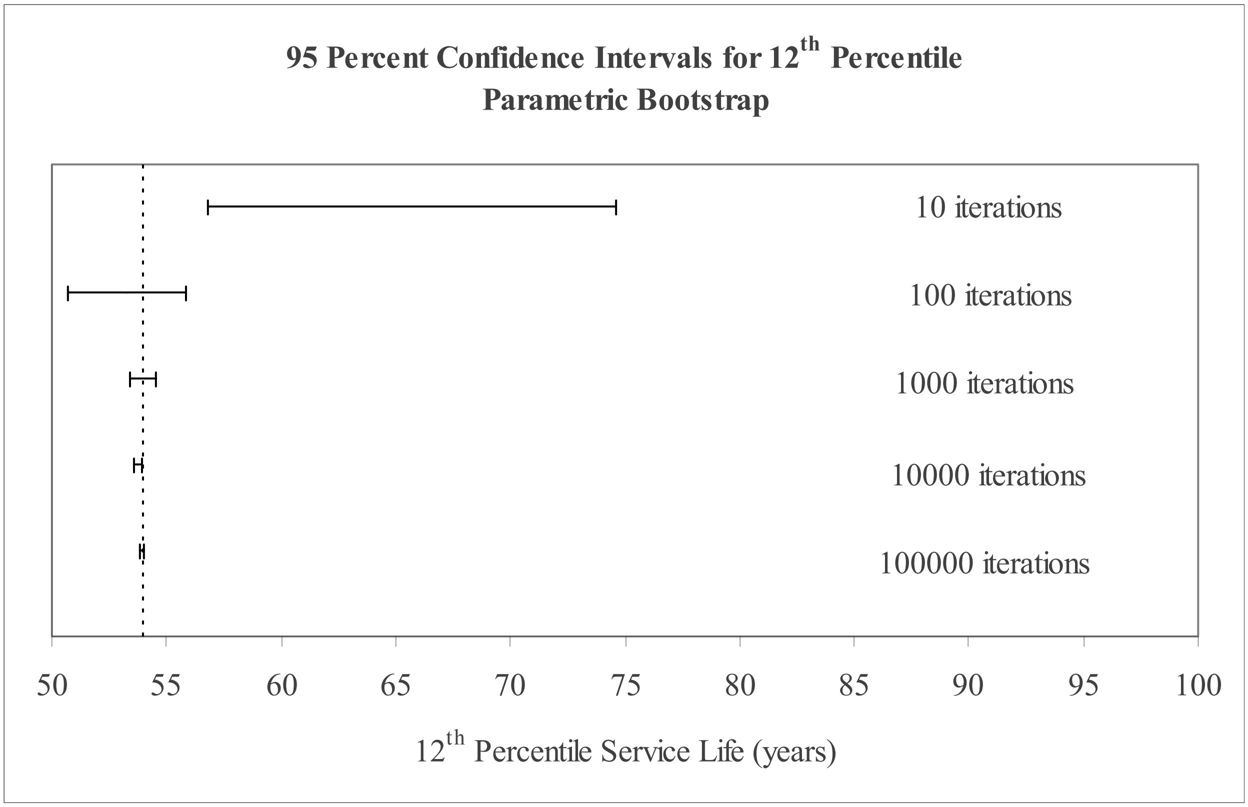


Figure 7 – Number of Iteration Plots for 2.5th Percentile



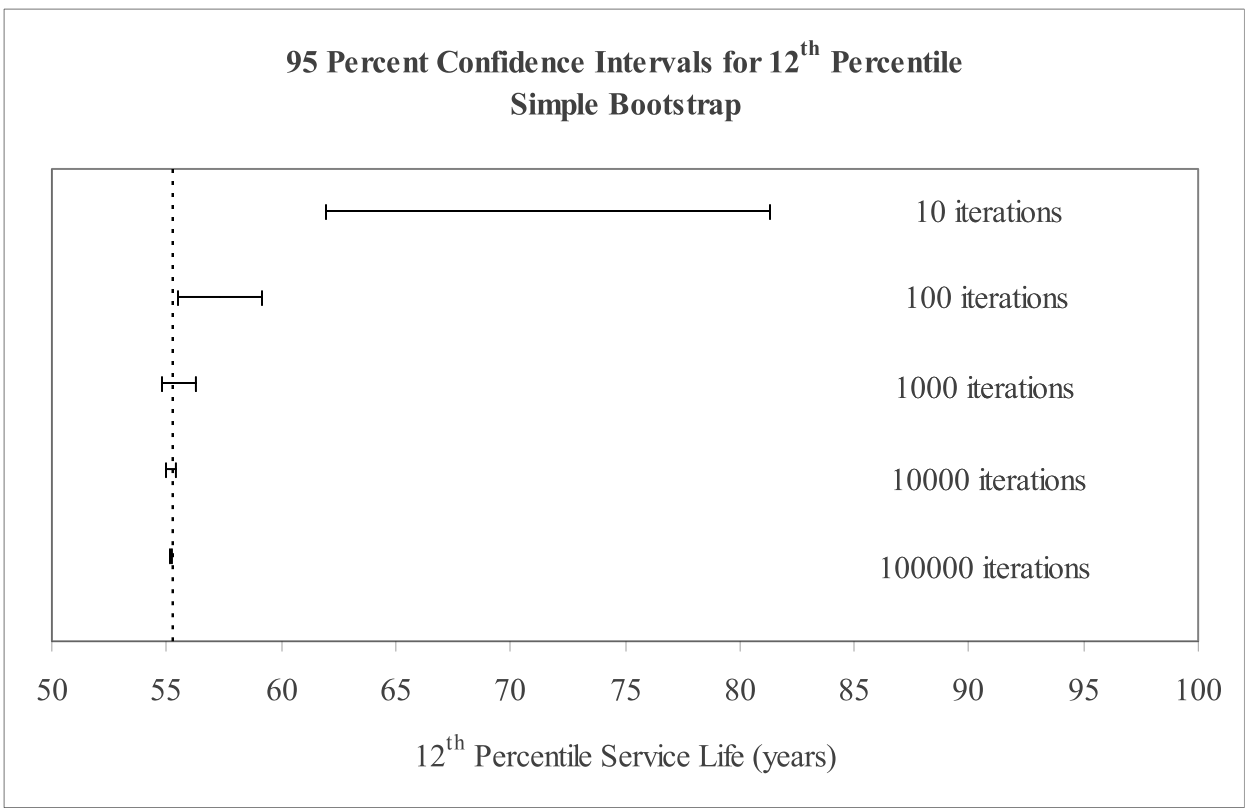


Figure 8 – Number of Iteration Plots for 12th Percentile

Figure 9 presents the relationship between the apparent diffusion coefficient and the time for diffusion to corrosion initiation. The time for diffusion to corrosion was calculated for a range of apparent diffusion coefficients from close to 0.0 to 100 mm^2/year (0.0 to 0.155 in^2/year). The value of the chloride initiation concentration was held constant at 0.9 kg/m^3 (1.5 lb/cy), and the cover depth was 50 mm (2.0 in). Curves were generated for values of the surface chloride concentration equal to 2.0, 4.0, and 6.0 kg/m^3 (3.3, 6.7, and 10.0 lb/cy), which represent mild, moderate, and severe exposure conditions, respectively.

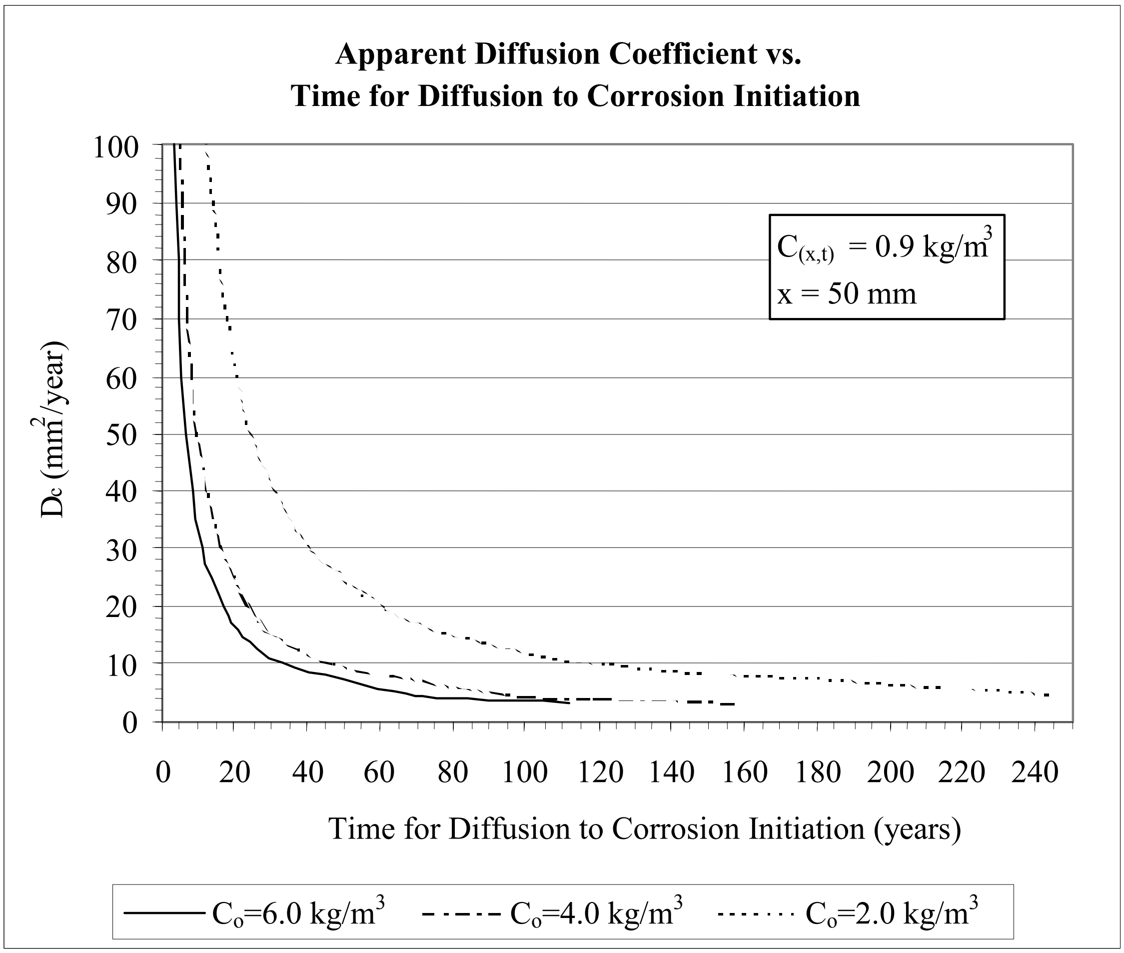
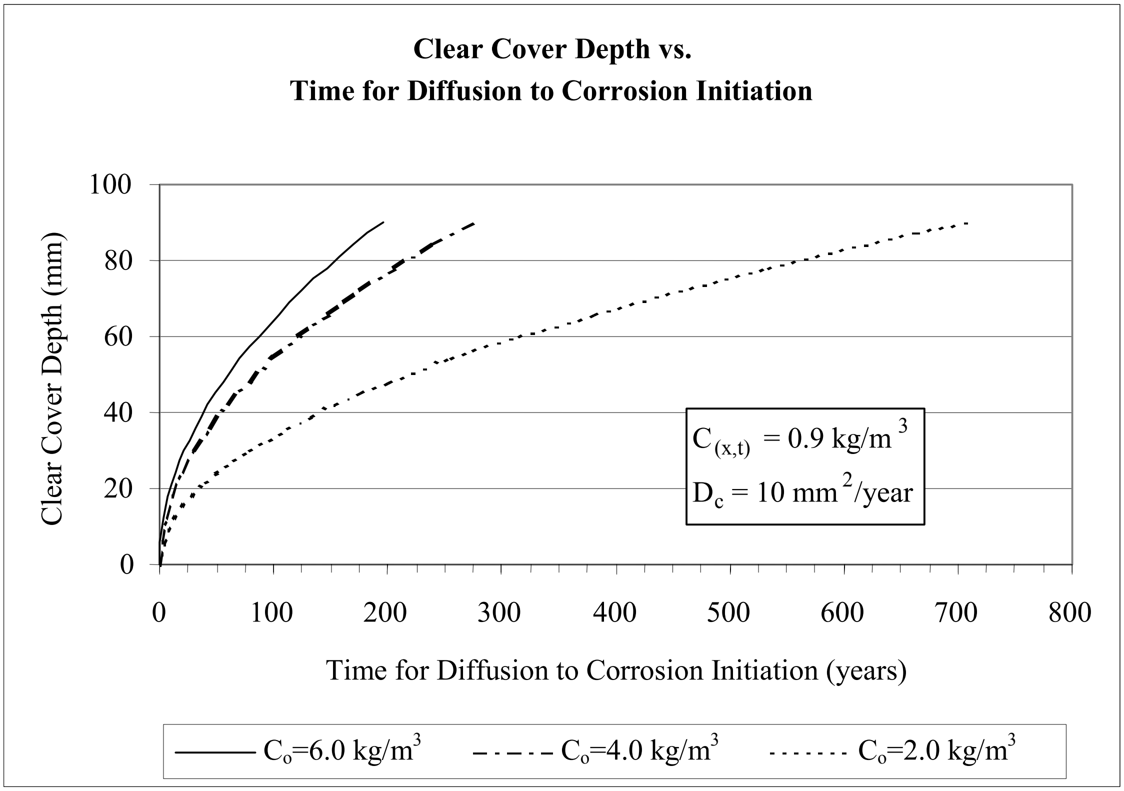


Figure 9 – Apparent Diffusion Coefficient vs. Time for Diffusion to Corrosion Initiation

Figure 10 presents the relationship between the clear cover depth and the time for diffusion to corrosion initiation. The curves were generated for a range of clear cover depths between 0.0 and 90 mm (0.0 and 3.5 in). The value of $C_{(x,t)}$ was held at 0.9 kg/m^3 (1.5 lb/cy). Separate curves were generated for D_c values of 10 and 50 mm^2/year (0.016 and 0.078 in^2/year) and C_o values in the three exposure conditions.



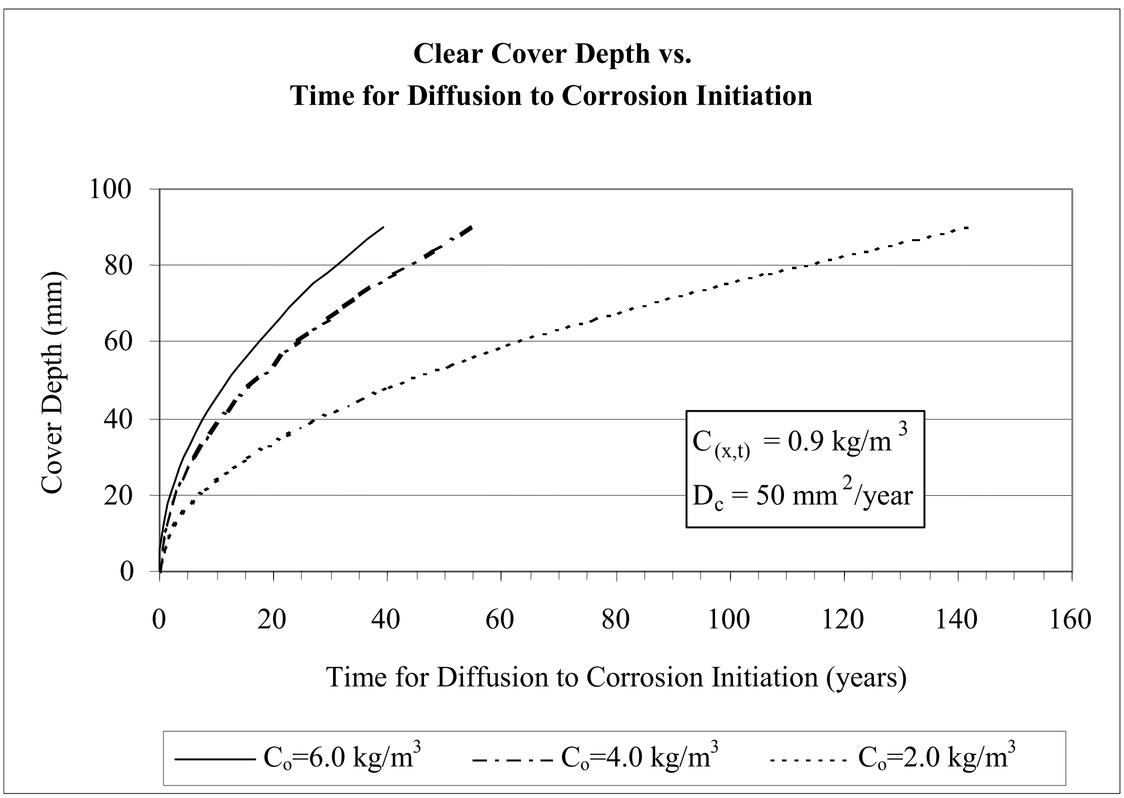


Figure 10 – Clear Cover Depth vs. Time for Diffusion to Corrosion Initiation

Figure 11 presents the relationship between the ratio of $C_{(x,t)}/C_o$ and the time for diffusion to corrosion initiation. The value of the cover depth was held at 50 mm (2.0 in). Curves were generated for values of D_c equal to 10, 30, and 50 mm^2/year (0.016, 0.047, and 0.078 in^2/year).

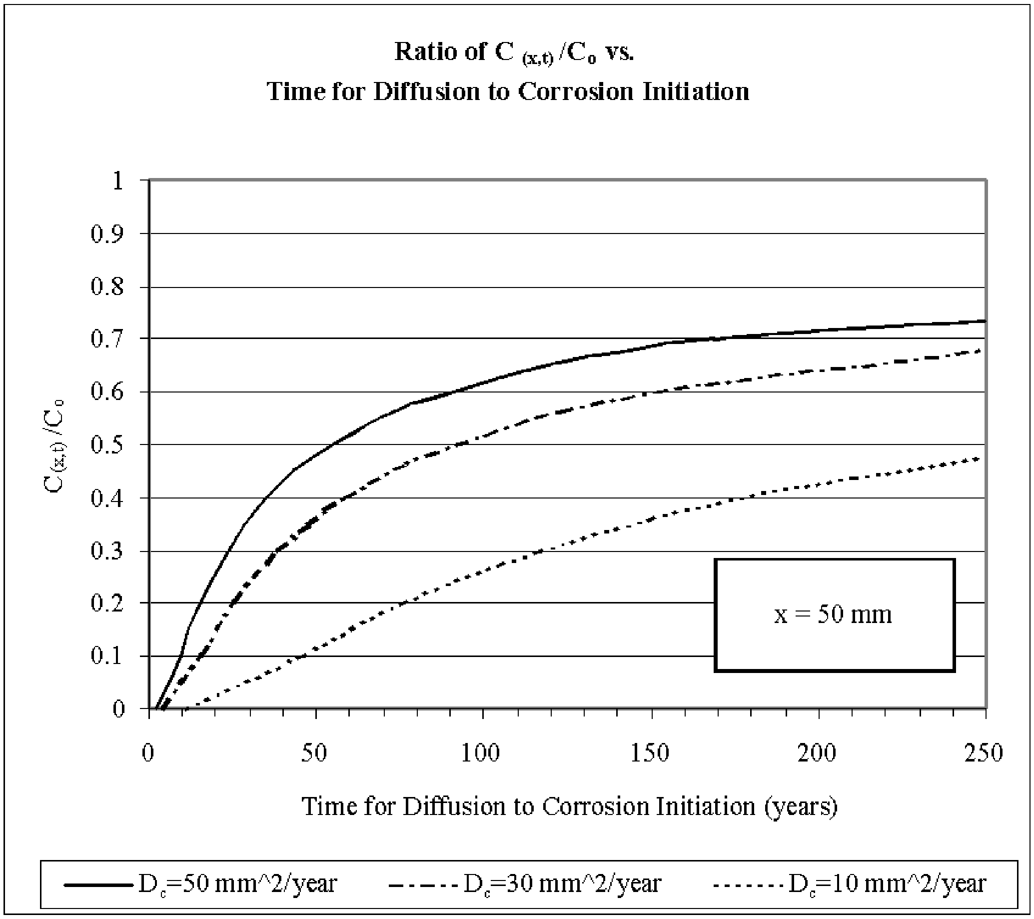


Figure 11 – Ratio of $C_{(x,t)}/C_o$ vs. Time for Diffusion to Corrosion Initiation

Results of the Simulation

Once the number of iterations and the sensitivity of the model were determined, the time to first repair and rehabilitation was determined for the 10 bridge decks included in this study.

Parametric vs. Simple Bootstrap

Both the parametric and simple bootstraps were used for each bridge. Table 2 shows the estimated percent corroded and 2.5th and 12th percentile service life estimates for each bridge for both the parametric and simple bootstrap for the range of chloride initiation from 0.6 to 1.2 kg/m^3 (1.0 to 2.0 lb/cy). These values include an estimated time for corrosion damage (cracking and spalling) after initiation of 4 years. The percent corroded corresponds to the number of model iterations in the simulation that predict corrosion damage in a bridge deck (C_o is larger than $C_{(x,t)}$). The 2.5th and 12th percentile values correspond to the time to first repair and time to rehabilitation, respectively. Predicted times to first repair and rehabilitation longer than 100 years were deemed to be unrealistic and were reported as 100 years.

Figure 12 through Figure 20 show histograms of the results of the simulation for the parametric and simple bootstrap for nine of the bridges and for the range of chloride initiation from 0.6 to 1.2 kg/m^3 (1.0 to 2.0 lb/cy). The percentage of iterations that predict no corrosion is noted on the graphs where appropriate, and for clarity, extreme values in the right tail were not shown. The histogram for Bridge 6128 was not included because of the 10,000 iterations of the model, none predicted corrosion of the reinforcing steel. The values for the 2.5th and 12th percentile of the distributions are most important because they represent the time to first repair and rehabilitation, respectively. Comparing the shape of the distributions for the parametric and simple bootstrap is also useful to evaluate the dependence of the results on distributional assumptions.

Effect of the Chloride Initiation Concentration

Table 3 summarizes the results of the parametric and simple bootstrap for each bridge and each range of chloride initiation concentrations. An estimated time for corrosion damage of 4 years is included in the values. Figure 21 through Figure 26 are cumulative distribution plots for the distributions of service life determined from the model. The 12th percentile line is shown on the graph and represents the same time to rehabilitation as presented in Table 3. The effect of the chloride initiation concentration on the time to rehabilitation can be seen from these graphs. No graphs were generated for Bridges 2021, 1004_6, 6037, or 6128 because corrosion was not predicted to occur on these structures within 100 years at any of the chloride initiation concentrations.

Average Value Solution

The time to first repair and rehabilitation determined by the probabilistic model was compared with the time to first repair and rehabilitation determined by the average values of the input variables. The average value solution is commonly used to predict the time to first repair and rehabilitation of bridge decks at the present time. The procedure involves the solution of Equation 1 using the average values for C_o, D_c, and $C_{(x,t)}$ and either the 2.5th or 12th percentile of the cover depth (Weyers et al. 1993). The diffusion time calculated using the average values is then added to the corrosion time to determine the time to first repair or time to rehabilitation. The averages of the input values used in the analysis are shown in Table 4. The time for diffusion to 2.5th and 12th percent of the steel calculated by each method is presented in Table 5 (the results of the simple bootstrap are not shown).

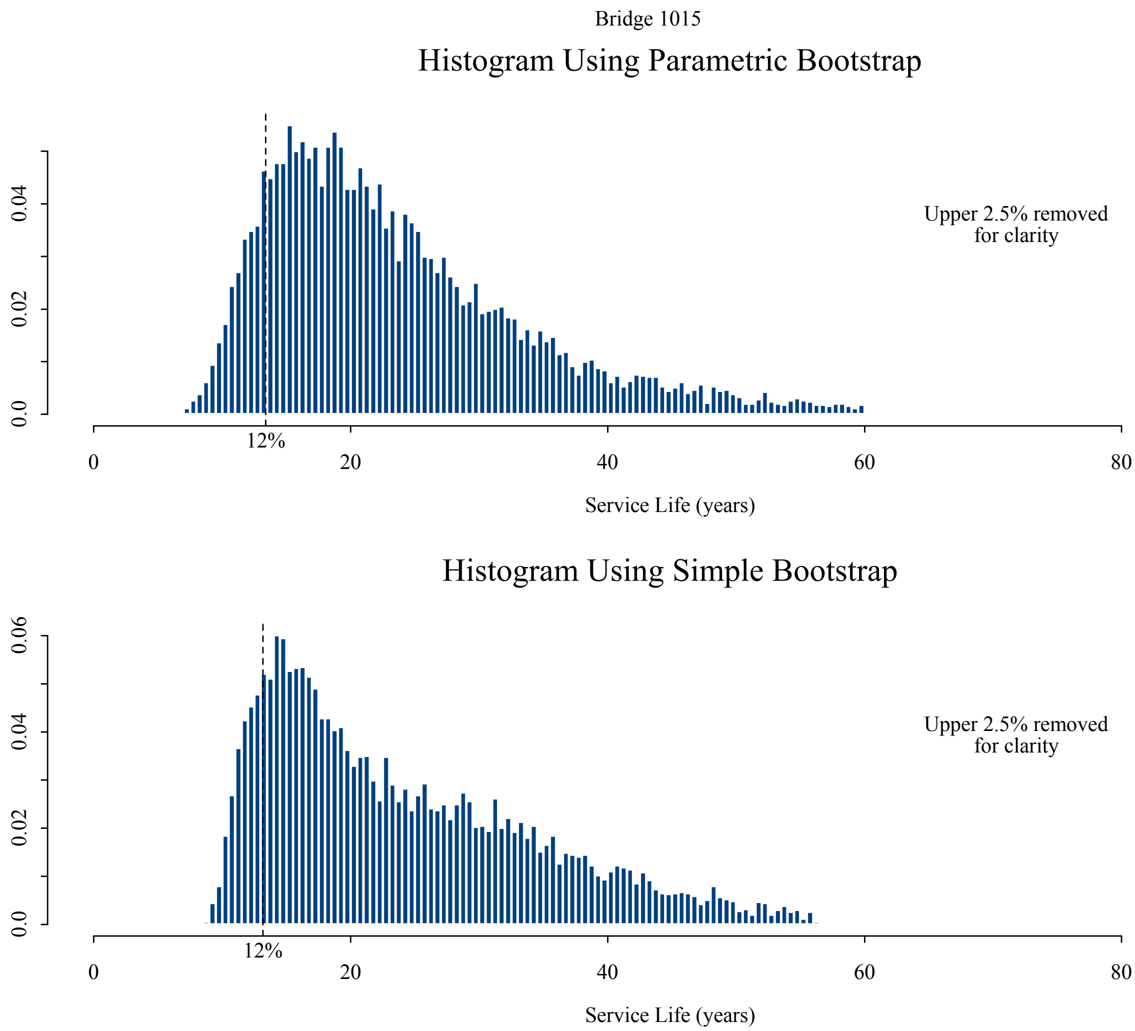


Figure 12 – Histogram for Parametric and Simple Bootstrap for Bridge 1015

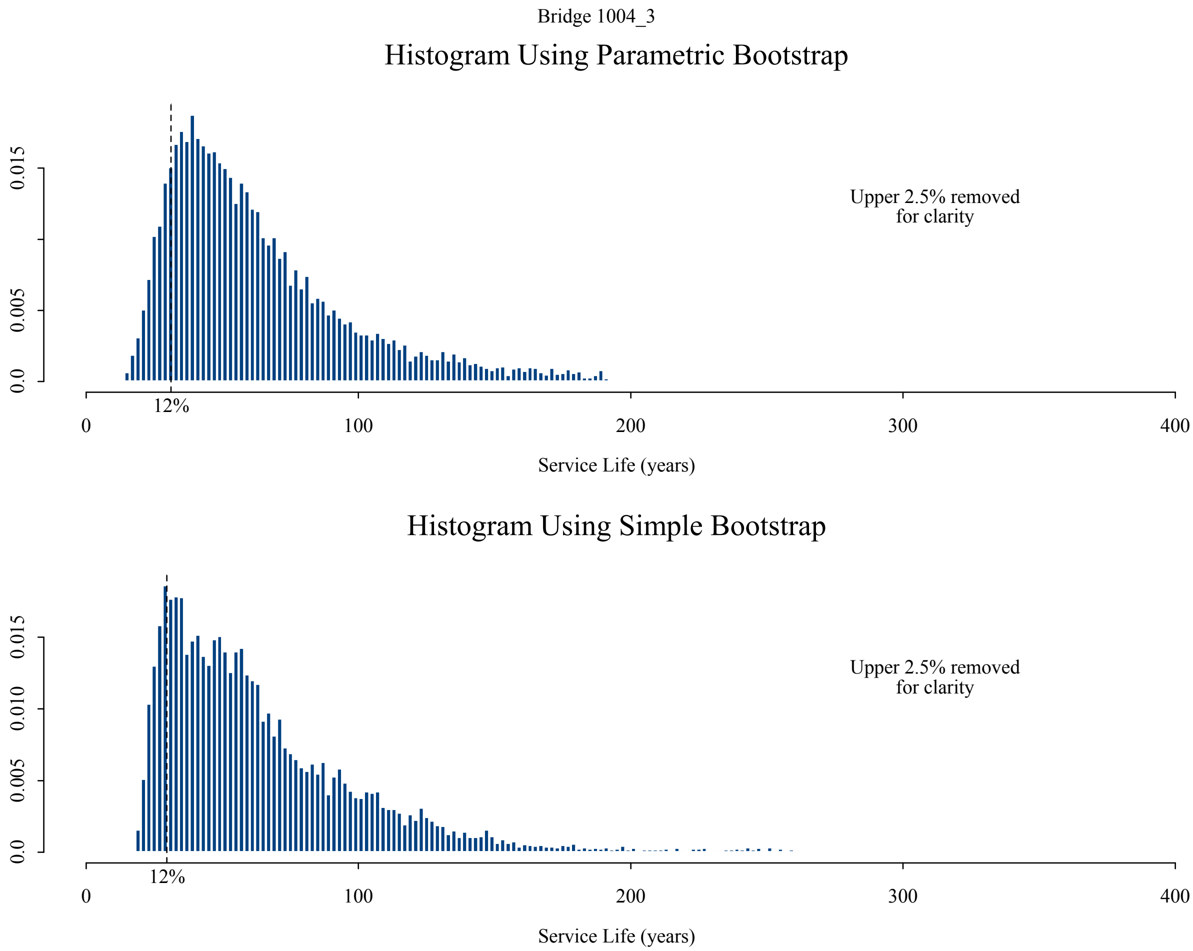


Figure 13 – Histogram for Parametric and Simple Bootstrap for Bridge 1004_3

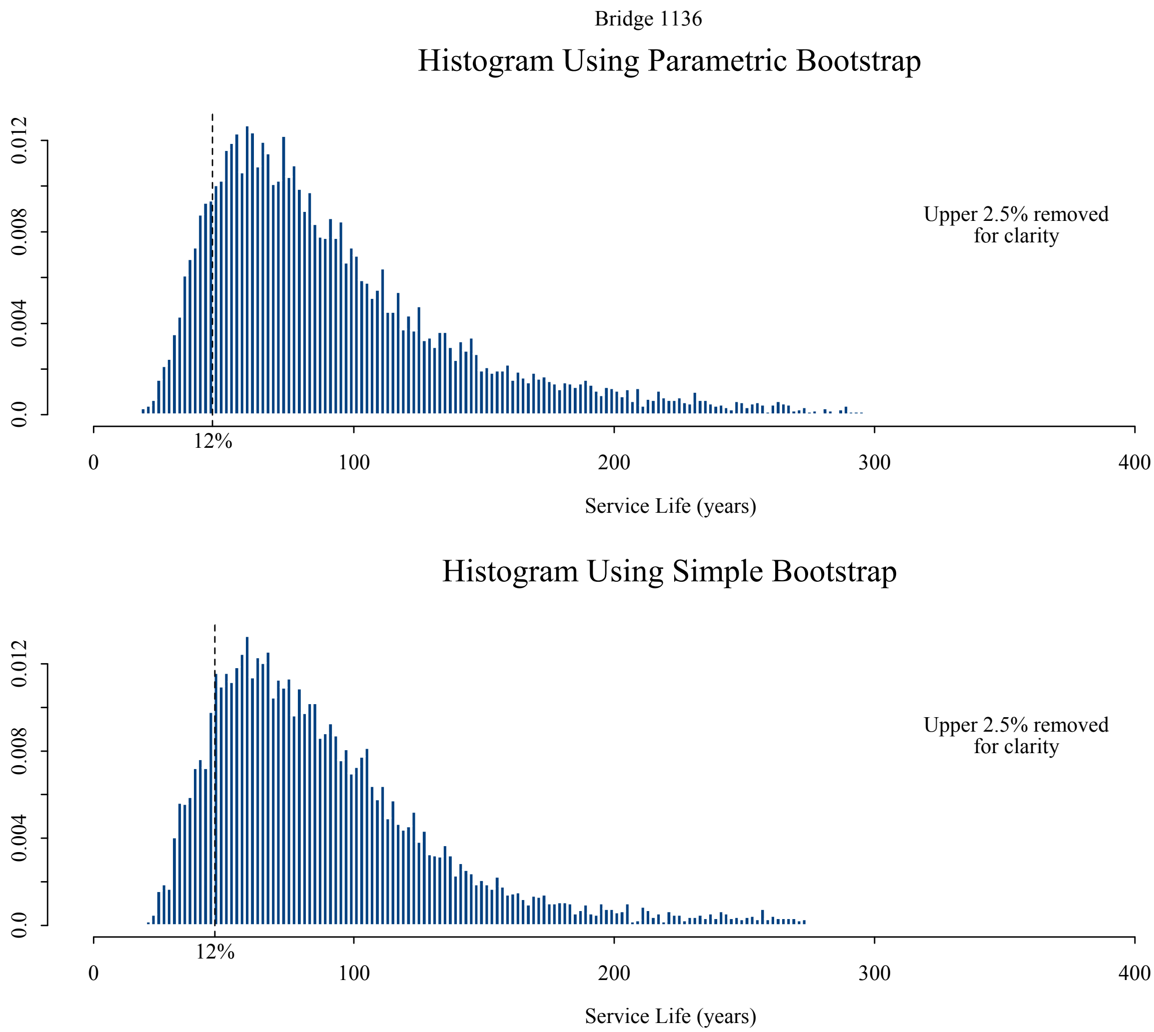


Figure 14 – Histogram for Parametric and Simple Bootstrap for Bridge 1136

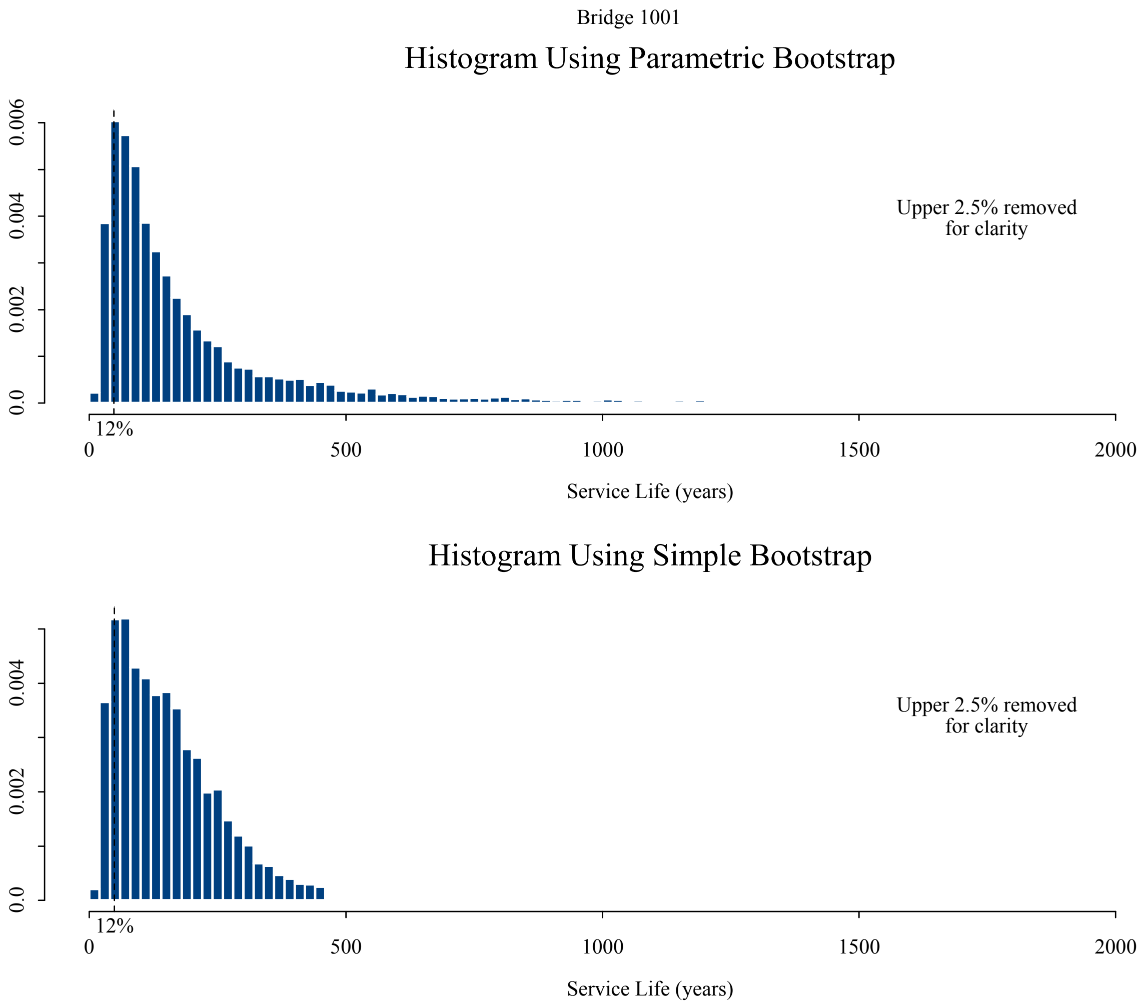


Figure 15 – Histogram for Parametric and Simple Bootstrap for Bridge 1001

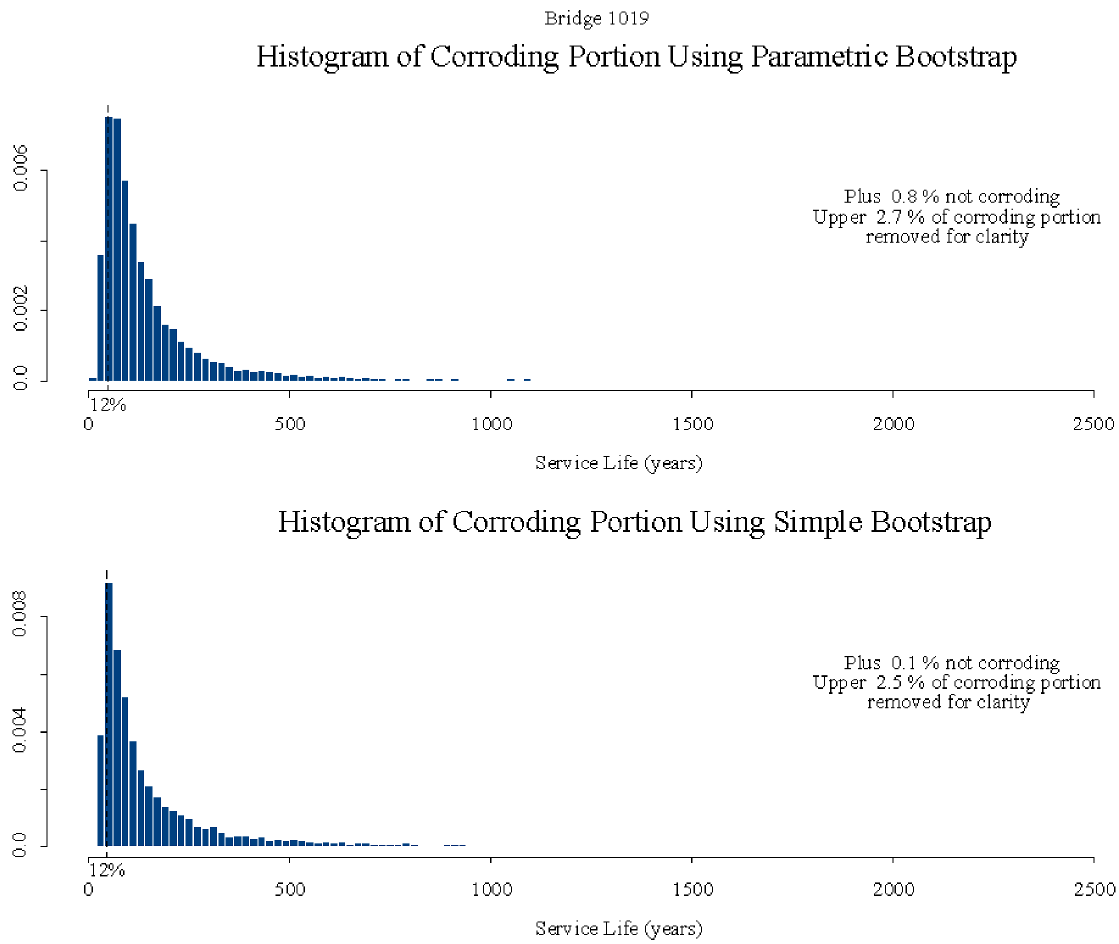


Figure 16 – Histogram for Parametric and Simple Bootstrap for Bridge 1019

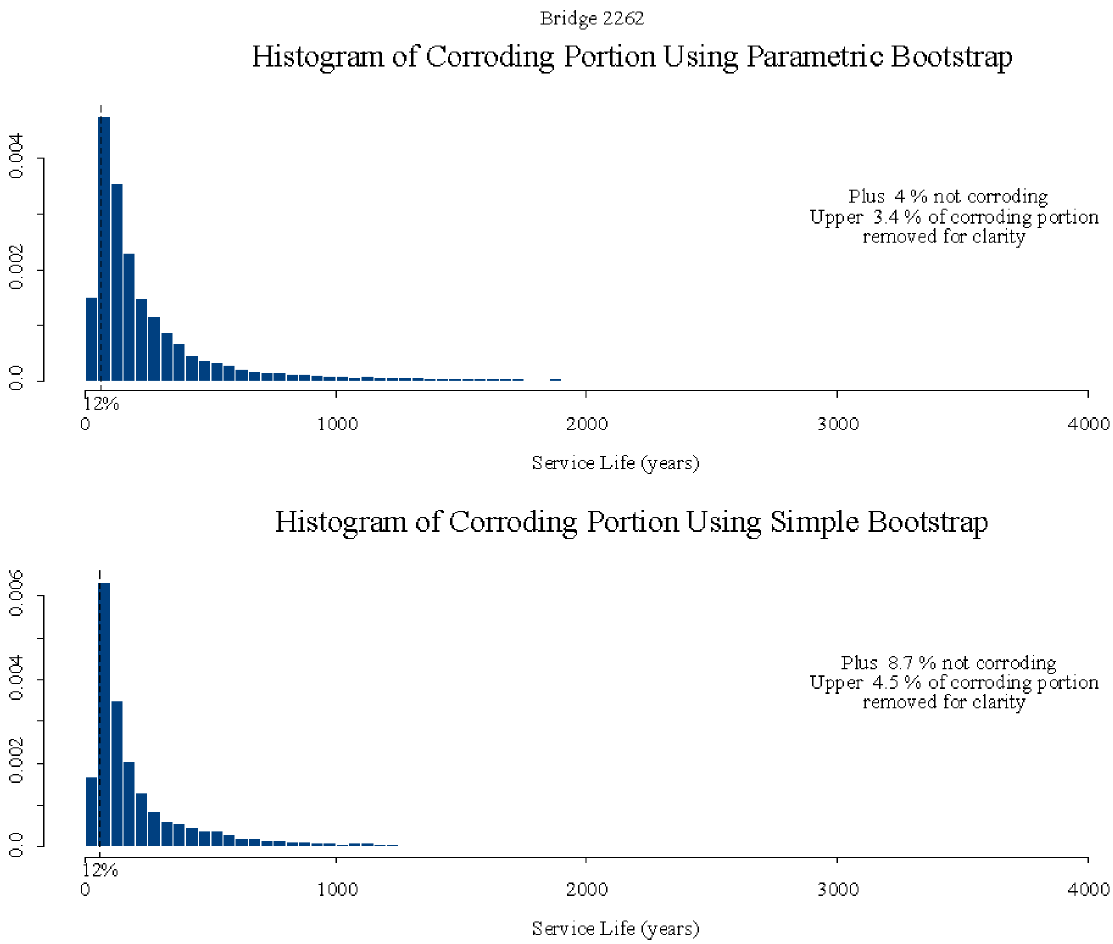


Figure 17 – Histogram for Parametric and Simple Bootstrap for Bridge 2262

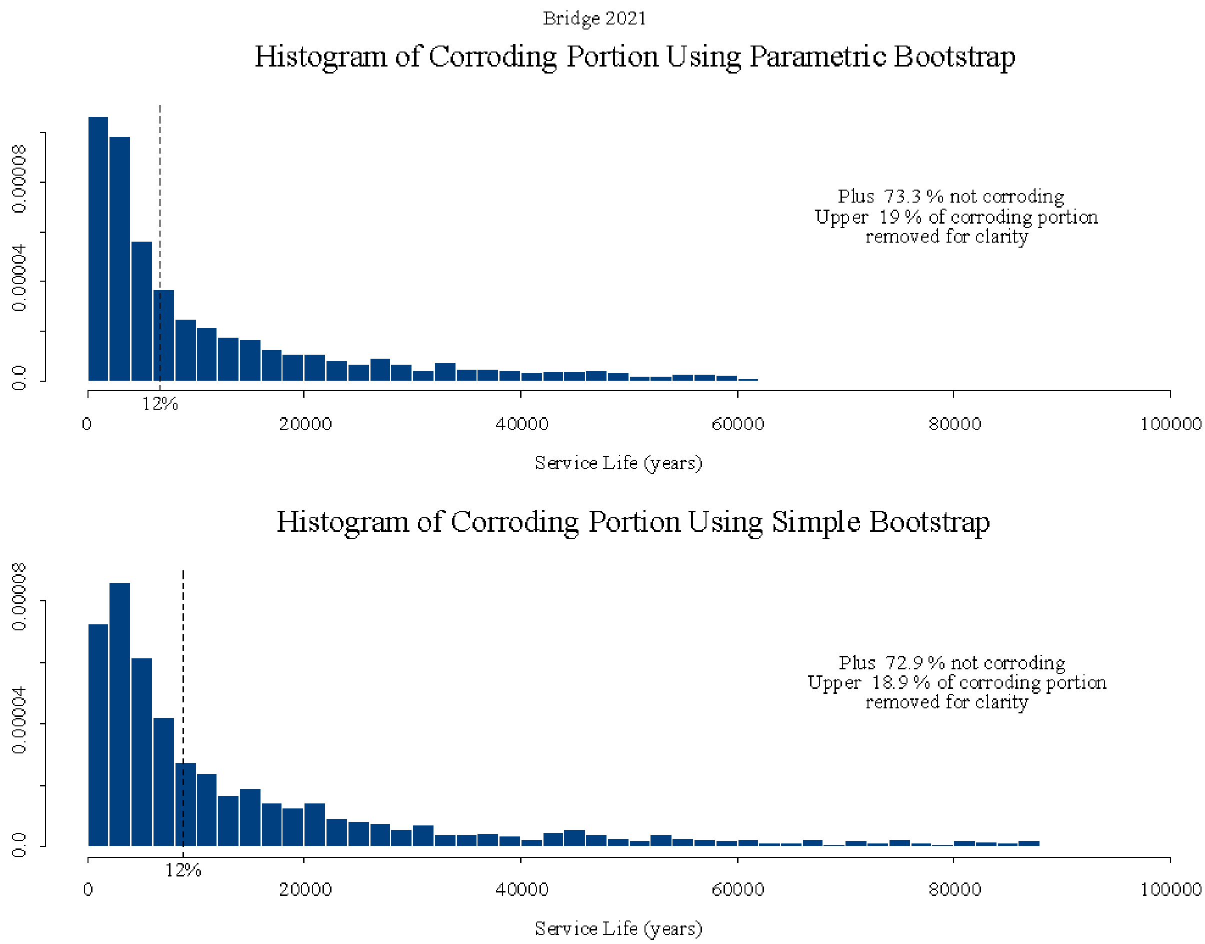


Figure 18 – Histogram for Parametric and Simple Bootstrap for Bridge 2021

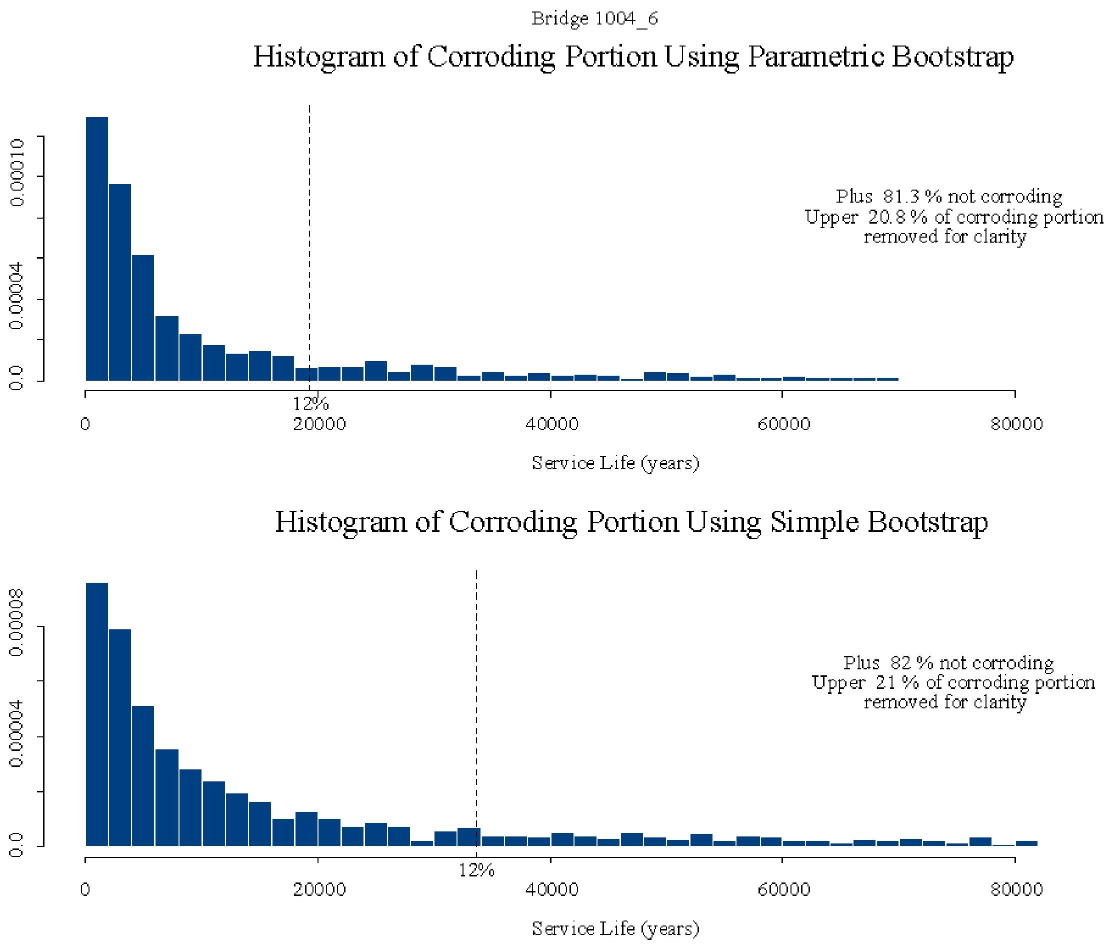


Figure 19 – Histogram for Parametric and Simple Bootstrap for Bridge 1004_6

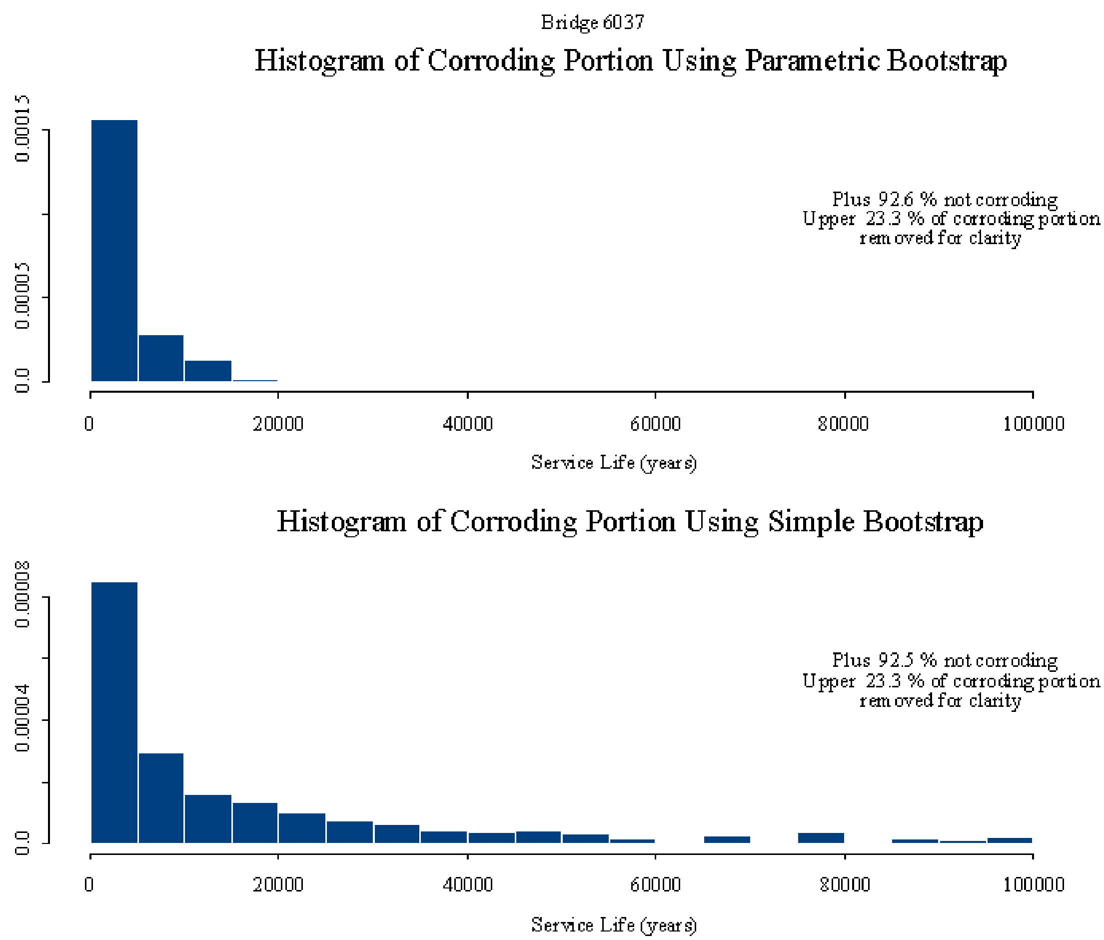
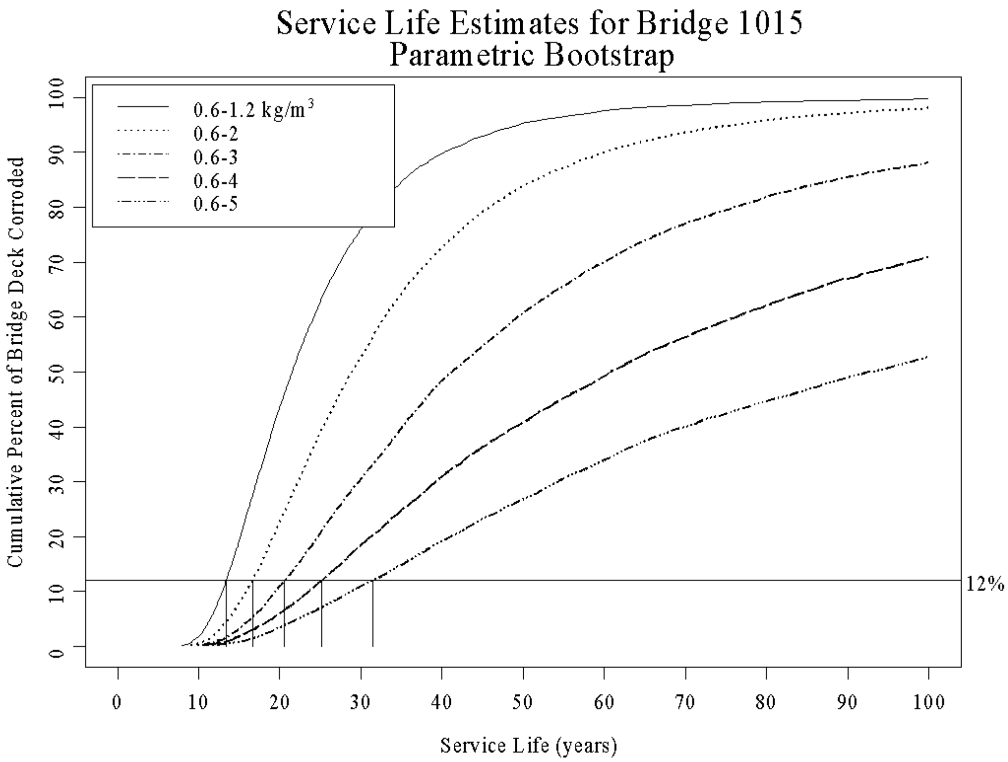


Figure 20 – Histogram for Parametric and Simple Bootstrap for Bridge 6037



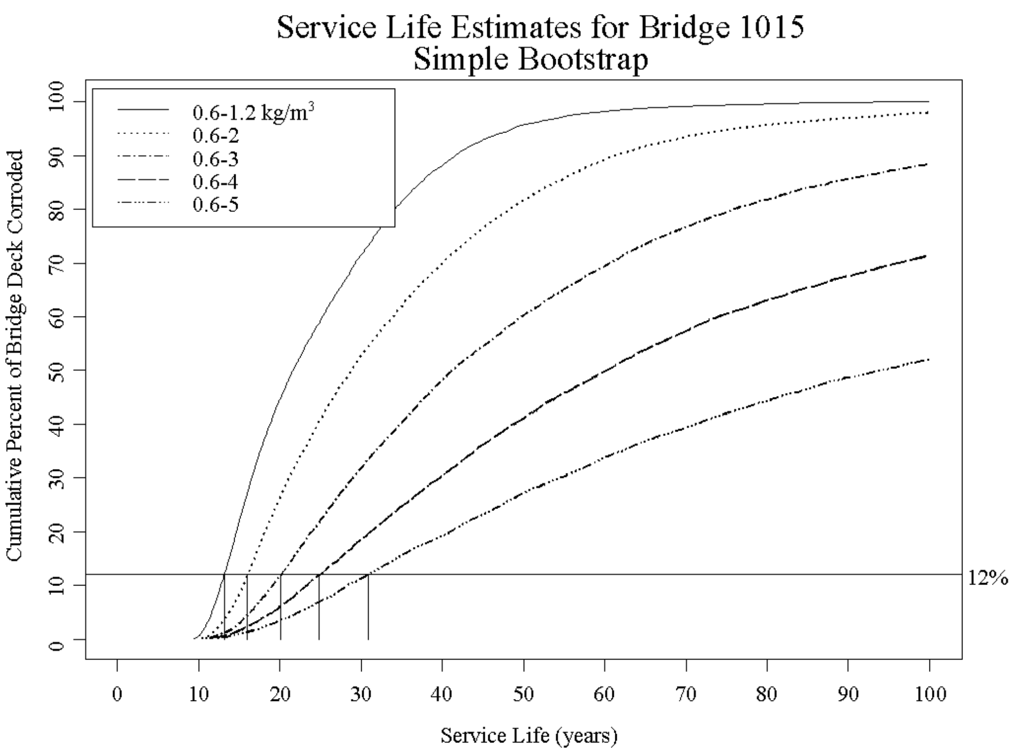


Figure 21 – Service Life Estimates for Bridge 1015

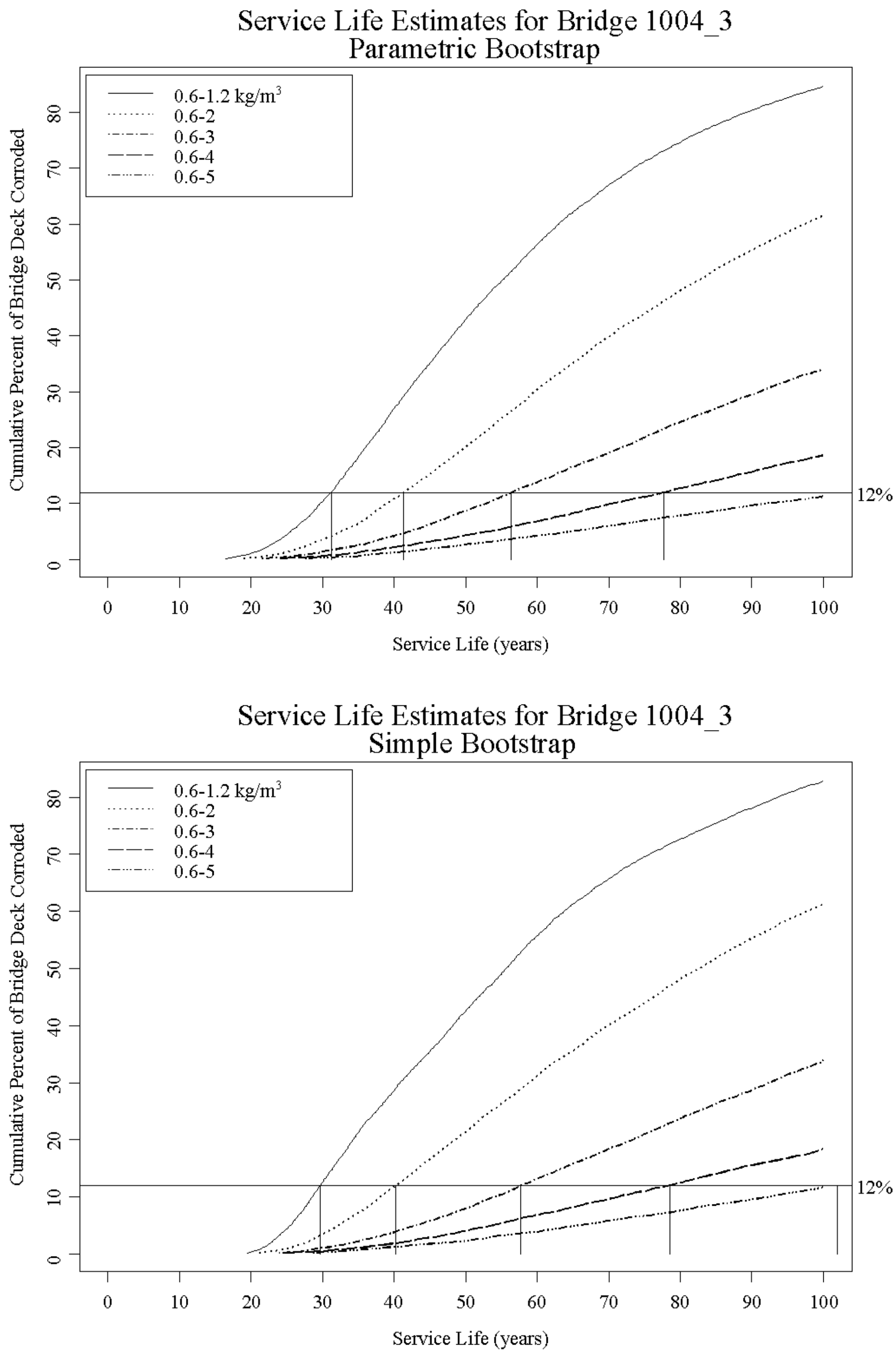


Figure 22 – Service Life Estimates for Bridge 1004_3

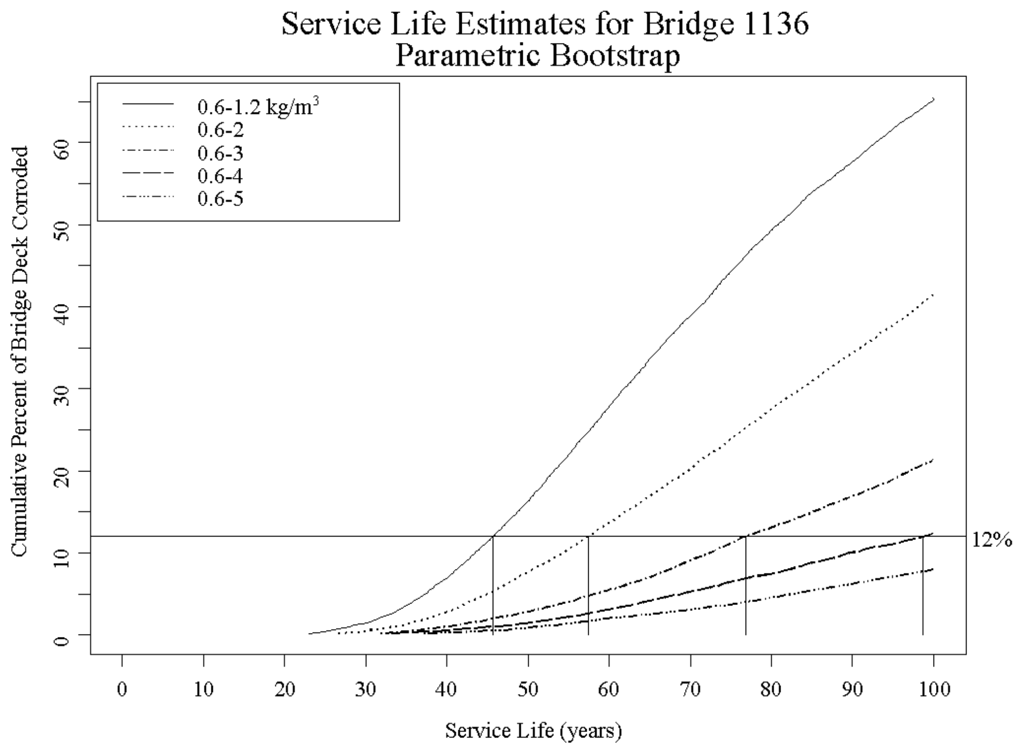
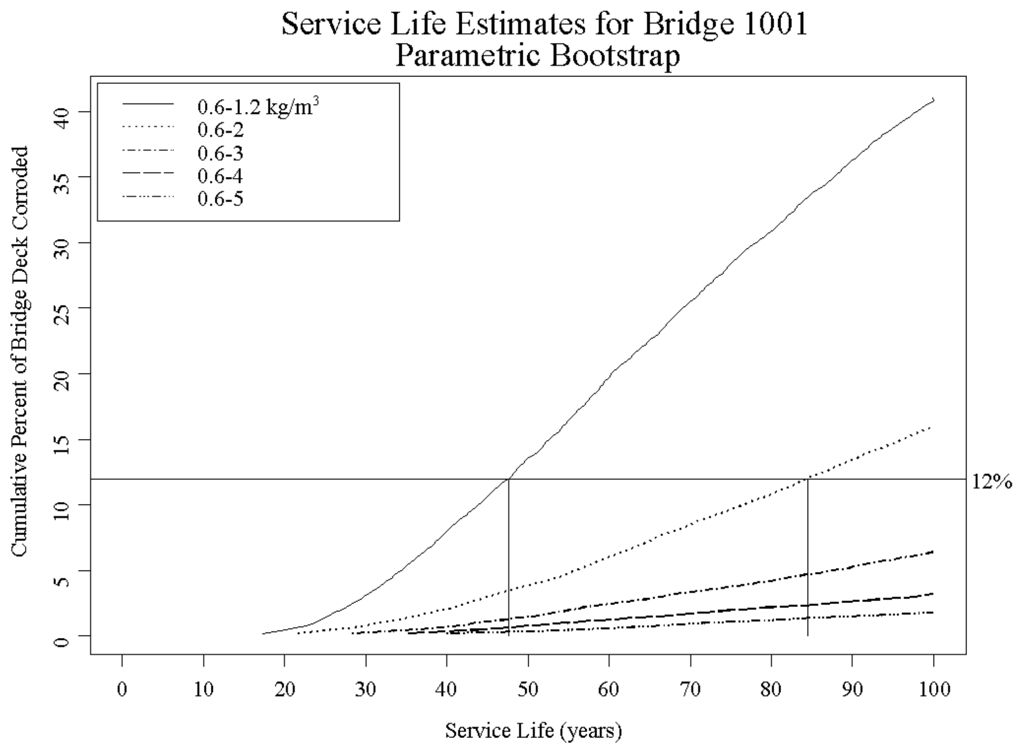


Figure 23 – Service Life Estimates for Bridge 1136



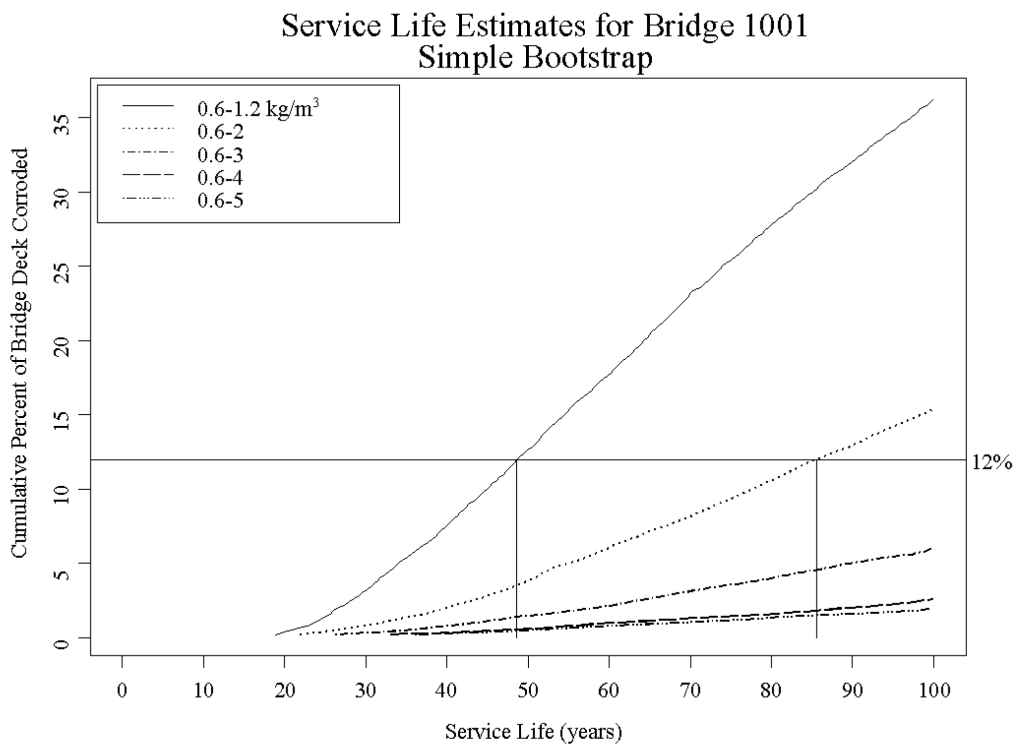


Figure 24 – Service Life Estimates for Bridge 1001

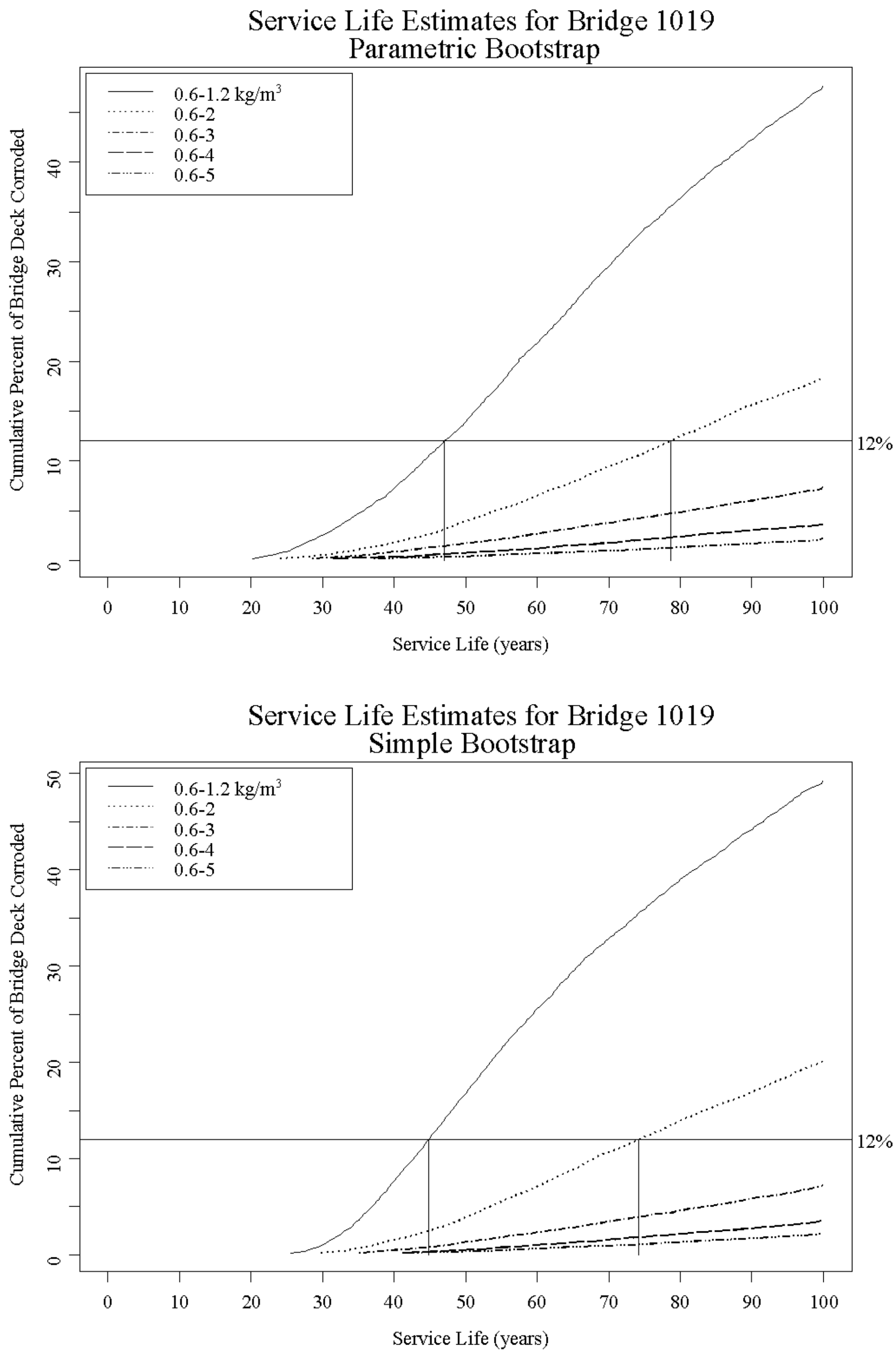
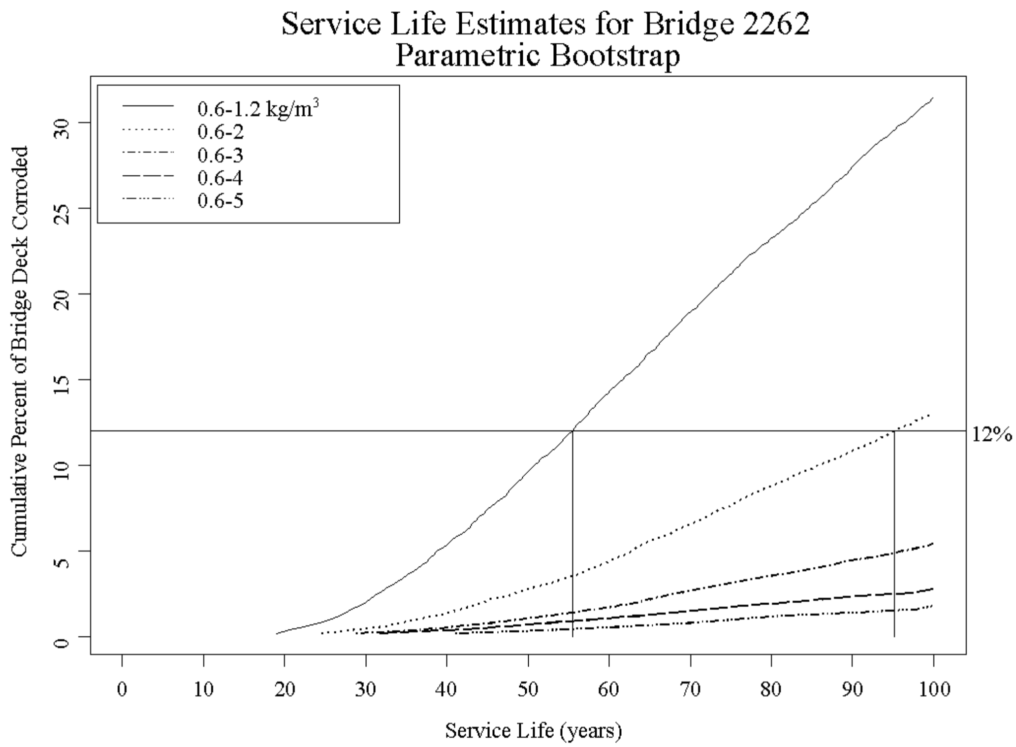


Figure 25 – Service Life Estimates for Bridge 1019



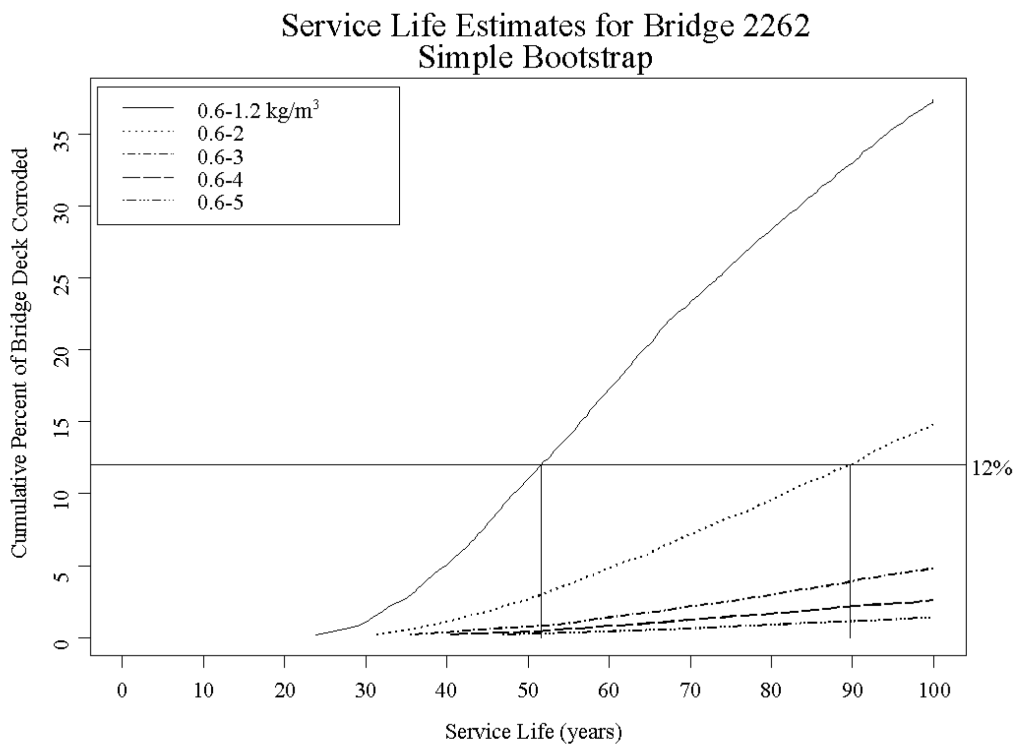


Figure 26 – Service Life Estimates for Bridge 2262

Table 2 – Time to First Repair and Rehabilitation

	Time to First Repair and Rehabilitation $C_{(x,t)}$ from 0.6 to 1.2 kg/m^3					
	Parametric Bootstrap			Simple Bootstrap		
Structure No.	% Corroded*	2.5% (years)	12% (years)	% Corroded*	2.5% (years)	12% (years)
1015	100	10	13	100	11	13
1004_3	100	23	31	100	23	30
1136	100	33	46	100	33	47
1001	100	28	48	100	28	49
1019	99	30	47	100	33	45
2262	96	31	56	91	34	52
2021	27	100	100	27	100	100
1004_6	19	100	100	18	100	100
6037	7	100	100	8	100	100
6128	0	100	100	0	100	100

* indicates the percentage of total bootstrap iterations that predict corrosion will occur at some point

Table 3 – Summary of Time to First Repair and Rehabilitation

$C_{(x,t)}$ (kg/m^3)	Parametric Bootstrap % Corroded*	Parametric Bootstrap 2.5% (years)	Parametric Bootstrap 12% (years)	Simple Bootstrap % Corroded*	Simple Bootstrap 2.5% (years)	Simple Bootstrap 12% (years)
		Service Life Summary for Bridge 1015				
0.6 to 1.2	100	10	13	100	11	13
0.6 to 2	100	12	17	100	13	16
0.6 to 3	100	14	21	100	15	20
0.6 to 4	100	16	25	99	16	25
0.6 to 5	97	19	32	95	18	31
		Service Life Summary for Bridge 1004_3				
0.6 to 1.2	100	23	31	100	23	30
0.6 to 2	100	28	41	99	29	40
0.6 to 3	99	35	56	95	36	58
0.6 to 4	94	42	78	93	43	79
0.6 to 5	82	49	100	84	52	100
		Service Life Summary for Bridge 1136				
0.6 to 1.2	100	33	46	100	33	47
0.6 to 2	100	39	57	100	40	59
0.6 to 3	98	48	77	100	50	78
0.6 to 4	93	57	100	97	56	100
0.6 to 5	84	65	100	87	67	100
		Service Life Summary for Bridge 1001				
0.6 to 1.2	100	28	48	100	28	49
0.6 to 2	99	43	84	99	43	86
0.6 to 3	80	62	100	81	64	100
0.6 to 4	51	87	100	50	100	100
0.6 to 5	32	100	100	32	100	100
		Service Life Summary for Bridge 1019				
0.6 to 1.2	99	30	47	100	33	45
0.6 to 2	88	44	79	86	45	74
0.6 to 3	61	58	100	63	62	100
0.6 to 4	37	81	100	39	86	100
0.6 to 5	23	100	100	24	100	100

* indicates the percentage of total bootstrap iterations that predict corrosion will occur at some point

	Parametric Bootstrap			Simple Bootstrap		
$C_{(x,t)}$ (kg/m^3)	% Corroded*	2.5% (years)	12% (years)	% Corroded*	2.5% (years)	12% (years)
Service Life Summary for Bridge 2262						
0.6 to 1.2	96	31	56	91	34	52
0.6 to 2	81	48	95	83	49	90
0.6 to 3	56	68	100	61	75	100
0.6 to 4	35	100	100	38	100	100
0.6 to 5	23	100	100	23	100	100
Service Life Summary for Bridge 2021						
0.6 to 1.2	27	100	100	27	100	100
0.6 to 2	8	100	100	8	100	100
0.6 to 3	3	100	100	2	100	100
0.6 to 4	1	100	100	1	100	100
0.6 to 5	1	100	100	1	100	100
Service Life Summary for Bridge 1004_6						
0.6 to 1.2	19	100	100	18	100	100
0.6 to 2	5	100	100	4	100	100
0.6 to 3	2	100	100	2	100	100
0.6 to 4	1	100	100	1	100	100
0.6 to 5	1	100	100	0	100	100
Service Life Summary for Bridge 6037						
0.6 to 1.2	7	100	100	8	100	100
0.6 to 2	2	100	100	1	100	100
0.6 to 3	1	100	100	0	100	100
0.6 to 4	0	100	100	0	100	100
0.6 to 5	0	100	100	0	100	100
Service Life Summary for Bridge 6128						
0.6 to 1.2	0	100	100	0	100	100
0.6 to 2	0	100	100	0	100	100
0.6 to 3	0	100	100	0	100	100
0.6 to 4	0	100	100	0	100	100
0.6 to 5	0	100	100	0	100	100

* indicates the percentage of total bootstrap iterations that predict corrosion will occur at some point

Table 4 –Input Parameters Used in Average Value Solution

	Average Value Input Parameters				
Structure No.	$C_{(x,t)}$ (kg/m^3)	C_o (kg/m^3)	D_c (mm^2/year)	$x_{2.5\%}$ (mm)	$x_{12\%}$ (mm)
1015	0.9	5.24	51.6	43.2	48.1
1004_3	0.9	3.89	39.3	63.0	66.3
1136	0.9	4.60	16.8	52.3	56.1
1001	0.9	2.32	28.1	50.1	54.9
1019	0.9	2.02	28.8	41.9	46.5
2262	0.9	1.95	27.8	43.5	49.7
2021	0.9	0.73	5.2	45.4	51.0
1004_6	0.9	0.67	11.1	53.9	58.9
6037	0.9	0.41	11.2	33.3	37.5
6128	0.9	0.16	39.1	51.2	54.5

Table 5 – Comparison of Average Value and Probabilistic Solutions

	Time for Diffusion to 2.5% of Steel		Time for Diffusion to 12% of Steel	
Structure No.	Average Values (years)	Parametric Bootstrap (years)	Average Values (years)	Parametric Bootstrap (years)
1015	10	6	12	9
1004_3	35	19	39	27
1136	49	29	56	42
1001	60	24	72	44
1019	52	26	65	43
2262	63	27	82	52
2021	-	-	-	-
1004_6	-	-	-	-
6037	-	-	-	-
6128	-	-	-	-

Validation of the Model

Historical service life data were compiled and updated for 129 bridge decks built in Virginia between 1968 and 1972. The bridge decks were originally part of a project to determine the impact of a specification change in the late 1960s (Newlon 1974). The service life of these bridges was investigated several years ago as part of the Strategic Highway Research Program (Weyers R E et al. 1994). Updated service life data through April 2001 were obtained from the Virginia Department of Transportation for this project and are presented elsewhere (Kirkpatrick TJ 2001).

The 129 bridges were separated into interstate highways, U.S. routes, and Virginia routes. In each route type, the number of bridges that received polymer overlays and the number of bridges that were replaced or received concrete overlays were recorded. Table 6 presents the distribution of bridges in each category that received either a polymer or concrete overlay. Using the data, the mean service life was projected using normal probability distributions for each route type or combination of route types. Figure 27 and Figure 28 are graphical depictions of the projected service life estimates, exclusive and inclusive of polymer concrete overlays, respectively. Table 7 summarizes the mean and standard deviation of the projected service life for each route type.

Table 6 – Summary of Bridges from Newlon Study as of April 2001

Route Type	No. of Bridges	Percent	No. Receiving Polymer Overlays	No. Receiving Concrete Overlays
Interstate	35	27	16	3
US Route	41	32	13	12
VA Route	53	41	4	2
Total	129	100	33	17

For interstate highways, the projected mean service life was 28 years for bridge decks receiving concrete or polymer overlays and 28 years for those receiving a concrete overlay (excluding those bridges that received polymer overlays). For U.S. routes, the projected mean service life was 28 years for bridge decks receiving concrete or polymer overlays and 32 years for those receiving concrete overlays. For Virginia routes, the projected mean service life was 45 years for those bridge decks receiving concrete or polymer overlays and 105 years for those receiving concrete overlays. When all of the bridges in the study were combined, the projected mean service life was 33 years for bridge decks receiving concrete or polymer overlays and 38 years for those receiving concrete overlays. An earlier SHRP project reported a projected mean service life of 36 years with a standard deviation of 13 years (Weyers et al. 1994).

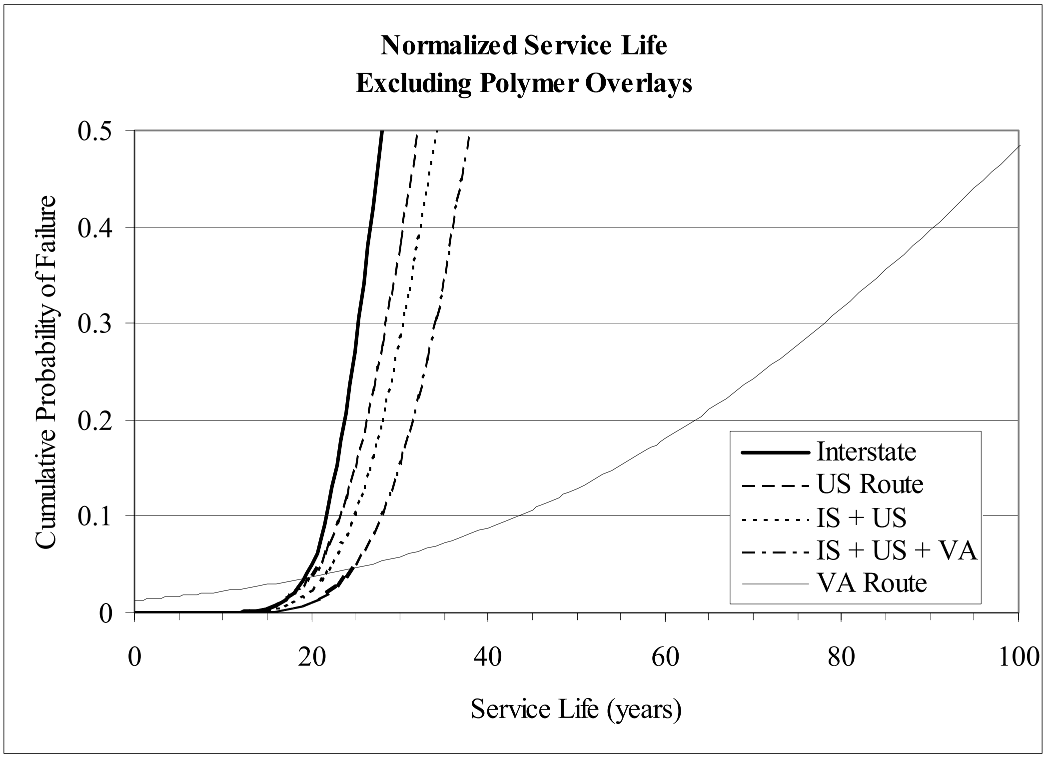


Figure 27 – Normalized Service Life Projections for Newlon Bridges – Excluding Polymers

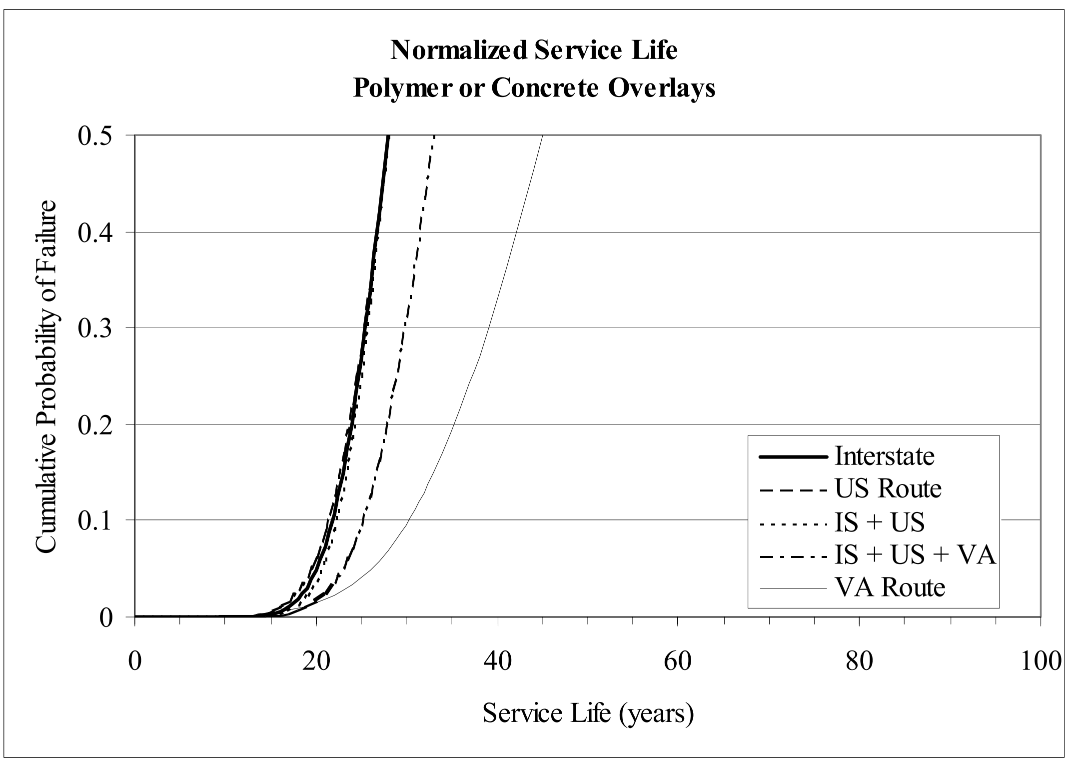


Figure 28 – Normalized Service Life Projections for Newlon Bridges

Table 7 – Projected Normalized Service Life (years) of Bridges from Newlon Study

Route Type	Bridges Receiving Concrete or Polymer Overlays		Bridges Receiving Concrete Overlays (Excluding Polymer Overlays)	
	Average	Standard Dev.	Average	Standard Dev.
Interstate (IS)	28	4.8	28	4.9
US Route	28	5.2	32	6.8
VA Route	45	11.5	102	45.9
IS + US + VA	33	6.1	38	7.9
IS + US	28	4.5	34	7.1

DISCUSSION

In this section, topics related to the prediction of the time to first repair and rehabilitation are discussed. The effects of the individual parameters of the model on the predicted service life are evaluated, and the model is validated using historical service life data.

Number of Iterations

The results used to determine the number of iterations required for the model to provide precise results was previously presented in Figure 7 and Figure 8. For both the parametric and simple bootstrap, the range of expected values of the time to first repair and rehabilitation tends to shorten and converge to a single value for increasing numbers of iterations. For very small numbers of iterations, the range of expected values predicted by several successive runs of the simulation is quite large, approximately 15 years for the time to first repair and 20 years for the time to rehabilitation. In addition, the predicted time at the center of the range for smaller numbers of iterations is higher than the predicted time at the center of the range for larger numbers of iterations. It is clear from the figures that at 10,000 iterations, the results have both converged and shifted to a near constant value.

The shifting behavior is of particular interest. It is expected that the range of predicted values will be larger for smaller numbers of iterations, but one might not expect the center of the range to be different for large and small numbers of iterations. If the behavior is not understood, the model may lead to inaccurate predictions of the time to first repair and rehabilitation. For smaller numbers of iterations, the shape of the input variables is not well defined, especially in the tails. For larger numbers of iterations, the distributions are better defined and more values are randomly sampled from the tails. The better defined input variables in turn lead to a better defined distribution of the predicted time to first repair and rehabilitation, especially in the tails. Therefore, for larger numbers of iterations, the model will tend to produce results that are more consistent and converge to a single value.

It is also important to distinguish between the precision of the model and the accuracy of the predicted time to first repair and rehabilitation. For successive runs of the model at 10,000 iterations, the model can be expected to predict times to first repair and rehabilitation that differ by less than one year for a given set of input variables. This does not imply that the prediction is accurate to one year, but that the error associated with the model has been limited to one year. The true accuracy of the prediction depends on the accuracy of the input variables, the sensitivity of the input parameters on the model and how closely the true deterioration mechanism matches the diffusion-cracking model employed here. The sensitivity of the input parameters is discussed in the following section.

Sensitivity of the Input Variables

Knowing the sensitivity of the input variables on the predicted time to first repair and rehabilitation is useful to understand possible sources of error in the model and to evaluate the likely affect of altering one of the input variables.

Apparent Diffusion Coefficient

The relationship between the apparent diffusion coefficient and the time for diffusion to corrosion initiation was shown previously in Figure 9. The relationship is inversely proportional and asymptotic for large values of D_c (small values of time for diffusion to corrosion initiation) and for large values of time for diffusion to corrosion initiation (small values of D_c). For values of D_c larger than approximately 30 mm^2/year (0.05 in^2/year), a large change in D_c creates only a small change in the time for diffusion to corrosion initiation. For values of D_c smaller than approximately 30 mm^2/year (0.05 in^2/year), a small change in D_c creates a large change in the predicted time to corrosion initiation. For values of D_c smaller than approximately 10 mm^2/year (0.016 in^2/year), which corresponds to highly impermeable concretes, the time for diffusion to corrosion initiation is very large. These relationships hold for concentrations of surface chlorides in all three of the exposure categories shown in the figure, although the trend is less pronounced for the mild exposure category.

The results of the sensitivity analysis indicate that the time to corrosion initiation is highly sensitive to D_c. This realization is troublesome because D_c is more difficult to obtain and is less well defined than the other input variables. It is clear that care must be exercised when calculating D_c from a chloride profile or when comparing D_cs from field data to those obtained experimentally from laboratory studies.

Clear Cover Depth

The relationship between the clear cover depth and the time for diffusion to corrosion initiation was presented previously in Figure 10. The relationship demonstrates that the time for diffusion to corrosion initiation increases as the clear cover depth increases. For clear cover depths greater than approximately 38 mm (1.5 in), the relationship is approximately linear with a slope that depends on the values of the other variables.

It is reasonable to conclude that increasing the clear cover depth will increase the time for chlorides to diffuse through the concrete to the reinforcing steel. It is also clear from the relationship that the predicted time to corrosion initiation is not highly sensitive to the clear cover depth. For clear cover depths typically observed on bridge decks today, a change in the clear cover of approximately 3.2 mm (0.125 in) would cause a change in the time for diffusion to corrosion initiation of approximately 10 percent.

Surface Chloride Concentration and Corrosion Initiation Concentration

The relationship between $C_{(x,t)}/C_o$ and the time to corrosion initiation was presented previously in Figure 11. The relationship demonstrates that the time for diffusion to corrosion initiation increases as the ratio of $C_{(x,t)}/C_o$ increases. The relationship is asymptotic for $C_{(x,t)}/C_o$ approaching 1. In other words, for values of C_o that are very close to the initiation concentration, a long time is required for the chlorides to diffuse through the cover concrete to the depth of the steel reinforcing in quantities sufficient to initiate corrosion of the reinforcing steel. For low values of D_c, the slope of the curve changes gradually and becomes asymptotic for very large times for diffusion to corrosion initiation. For higher values of D_c, the curve becomes asymptotic for much smaller values of times for diffusion to corrosion initiation.

For a corrosion initiation concentration of 0.9 kg/m^3 (1.5 lb/cy), C_o values in the negligible and mild exposure categories have $C_{(x,t)}/C_o$ values that are relatively close to 1.0 and cause the predicted time for diffusion to corrosion initiation to be very large. C_o values in the moderate and severe exposure categories have $C_{(x,t)}/C_o$ values that are much less than 1.0 and cause the predicted time to corrosion initiation to be shorter. The predicted time for diffusion to corrosion initiation when D_c is equal to 10 mm^2/year (0.016 in.^{2}/year) is much longer than the predicted time for diffusion to corrosion initiation when D_c is equal to 30 or 50 mm^2/year (0.047 or 0.078 in^2/year) for the entire range of $C_{(x,t)}/C_o$.

Generally speaking, bridges that are in negligible and mild exposure categories (typically rural routes with little traffic) are of much less concern than bridges that are located in moderate or high exposure zones (typically interstate or highway routes). Corrosion deterioration is much less likely to occur on bridge decks that receive only small salt applications than on bridge decks that receive higher salt applications. This observation is supported by the service life data originally collected by Newlon and updated for this project, as presented in Table 7 (Newlon 1974). The projected service life for bridge decks located on interstate and U.S. routes is 34, years while the projected service life for bridge decks located on Virginia routes is 102 years.

Results of the Simulation

Topics related to the results of the simulation for the10 bridge decks included in this study are discussed here.

Parametric vs. Simple Bootstrap

The parametric bootstrap makes the assumption that the populations of the input variables match known distributions and that the observed samples define the distributions by estimating the distributional parameters. This assumption must be evaluated along with the results of the simulation. The simple bootstrap makes the least amount of assumptions regarding the input data because the observed samples are assumed to adequately represent the entire population for each variable. Comparing the results of the two methods provides confidence in the simulation.

The results of the simulation for the parametric and simple bootstrap were presented previously in Table 2. The times to first repair and rehabilitation generated by the two methods agree well. Generally, the two methods are in closer agreement for shorter times to repair and rehabilitation, but the trend is not pronounced. The results of the two methods never differ by more than approximately 8 percent.

Along with the 2.5th and 12th percentiles, it is also helpful to compare the histograms of the predicted times calculated by the simulation. If the two methods provide results that are substantially the same, the shape of the histograms should be comparable. The histograms for 9 of the 10 bridges included in this study were shown in Figure 12 through Figure 20 (none of the iterations of the simulation predicted corrosion for Bridge 6128). The shape of the distributions calculated by the two methods generally agrees well for each bridge deck and has a positive skew and long right tail. The parametric bootstrap typically has a few values in the extreme right tail that are larger than those predicted by the simple bootstrap (for clarity, extreme values in the right tail were not shown on the histograms). The distributions used for the input variables by the parametric bootstrap are expected to have longer tails than the observed sample population, and it is expected that the occasional extreme input value will produce an extreme prediction of the service life. Since interest lies in the left tail, extreme values in the right tail do not affect the lower 2.5th and 12th percentiles, which represent the time to first repair and rehabilitation.

It has been shown that clear cover depths on bridge decks follow a normal distribution and that surface chloride concentrations and diffusion coefficients follow a gamma distribution (Zemajtis 1998; Weed 1974; Pyc 1998). Quantile-quantile plots reviewed for this project essentially confirm these observations, suggesting that the distributions used in the simulation accurately match the true population. However, it was also noted that the surface chloride concentrations and diffusion coefficients might be equally well described by normal distributions, suggesting that slight variations in the shape of the input distributions do not seriously affect the shape of the times to first repair and rehabilitation. These conclusions can be neither confirmed nor rejected based on the results of this study, and in any case are not the focus of the study. It is enough to note that the two methods provide results for each bridge that are comparable, providing confidence in the predicted times to first repair and rehabilitation.

Effect of Chloride Initiation Concentration

Because the true range of the chloride initiation concentration is not presently known, the times to first repair and rehabilitation were determined for several ranges of $C_{(x,t)}$. The results of the increasing range of $C_{(x,t)}$ are shown graphically in Figure 21 through Figure 26 and numerically in Table 3. The figures and table include an estimated 4 years for the time for corrosion damage of the reinforcing steel. As expected, increasing the range of $C_{(x,t)}$ increases the times to first repair and rehabilitation. For all of the bridges except Bridge 1015, the time to rehabilitation was predicted to occur after 100 years at the highest range of $C_{(x,t)}$, 0.6 to 5.0 kg/m^3 (1.0 to 8.3 lb/cy). Several of the bridges had predicted times to rehabilitation of approximately 50 years at the lowest range of $C_{(x,t)}$. For these bridges, the time to rehabilitation was predicted to occur after 100 years at some intermediate range of $C_{(x,t)}$, typically 0.6 to 3.0 kg/m^3 (1.0 to 5.0 lb/cy).

For bridge 1015, the time to rehabilitation was 13 years for the lowest range of $C_{(x,t)}$ and 32 years for the highest range of $C_{(x,t)}$. Even at the highest range, the predicted time to rehabilitation is less than the design life of 50 years. Four of the bridges included in the study (1136, 1001, 1019, and 2262) have predicted times to rehabilitation of approximately 50 years at the range of $C_{(x,t)}$ from 0.6 to 1.2 kg/m^3 (1.0 to 2.0 lb/cy) . Predictions of the time to rehabilitation using the higher ranges of $C_{(c,t)}$ are clearly longer, but for these bridges, the design life is nearly exceeded even at the most conservative estimate of $C_{(x,t)}$ used in this study, and the bridges would not be of concern at this time. Bridge 1004_3 had a time to rehabilitation that is less than 50 years at the lowest range of $C_{(c,t)}$ and longer than 50 years at an intermediate range of $C_{(x,t)}$. The time to rehabilitation was predicted to occur after 100 years at all levels of $C_{(x,t)}$ for the remaining four bridges in the study.

The observations of the data above suggest that increasing the range of the chloride initiation concentration may increase the predicted time to rehabilitation by a factor of two or more. However, if a bridge deck is of concern at the lowest range of chloride initiation, then it will likely be of concern at the higher ranges of chloride initiation. Likewise, if the bridge deck is not of concern at the lowest ranges, then it will not be of concern at the highest ranges. There will be bridge decks somewhere in between these extremes, and for these bridge decks, estimating the correct chloride initiation concentration is particularly important.

Probabilistic Solution vs. Average Value Solution

One approach to predicting the diffusion time to corrosion initiation is to solve Equation 1 using the mean values of the input variables. The times to first repair and rehabilitation are solved for by using the lower 2.5th or 12th percentile value of the cover depth in Equation 1. The average value method is simpler and easier to employ than the probabilistic method. However, with the exception of the cover depth, the average value approach does not reflect the variability of the input variables. In the average value approach, the surface chloride concentration, diffusion coefficient, and chloride initiation concentration are assumed to be constant over the entire deck surface, and the chlorides are assumed to diffuse through the concrete uniformly to the 2.5th or 12th percentile depth of the reinforcing steel.

In the probabilistic approach, the variability of each of the input variables is accounted for. The surface chloride concentration, diffusion coefficient, and chloride initiation concentration are not assumed to be constant over the entire deck surface. In the probabilistic approach, the deck is sectioned into many smaller points where the time for diffusion to corrosion initiation is calculated independent of the other locations, but in accordance with the probability distributions of the input variables. At individual simulated locations, the chlorides are assumed to diffuse through the concrete to the depth of the reinforcing steel at a rate defined by the probability distributions of the input variables. The times to first repair and rehabilitation are defined as the lowest 2.5th and 12th percentile values of all of the calculated times and, conceptually, correspond to 2.5th and 12th percent of the area of the deck that has corroded and spalled.

The predicted time to first repair and rehabilitation was determined for each bridge using both the average value method and the probabilistic method. The results of the two methods are presented in Table 5. For the 10 bridge decks included in this study, the probabilistic method predicted times to repair and rehabilitation that were consistently shorter than the times to repair and rehabilitation predicted by the average value method.

It was observed that the probabilistic solution is heavily influenced by the variability of the input variables. Table 8 presents the results of the probabilistic solution for the time for diffusion to corrosion initiation for Bridge 1001 using decreasing values of the coefficient of variation for the input variables. The first time for diffusion to corrosion initiation in the table uses the coefficients of variation for the data collected on the bridge deck. The coefficient of variation was then reduced for each input variable in steps to approximately 0. The percent difference between the resulting probabilistic estimate and the average value estimate is shown in the last column of the table. The percent difference between the estimates is reduced from 64 percent at the largest coefficients of variation to just 3 percent at the lowest coefficient of variation. For low coefficients of variation, the two methods produce similar results.

To investigate which of the input variables has the greatest influence on the predicted time for diffusion to corrosion initiation, the input values that contributed to the lowest 12 percent of the times for diffusion calculated in the simulation were separated out of the population of input values. The data from Bridge 1001 were used, and the results are summarized in Table 9. The table includes the minimum, maximum, and mean of the input variables that contributed to the lowest 12 percent of estimated times as well as the minimum, maximum, and mean of all of the input variables sampled during the simulation. The minimum, maximum, and mean of the actual data collected from the bridge deck are also included in the table.

Table 8 – Effect of Coefficient of Variation on Probabilistic Method

Time for Diffusion to Corrosion Initiation, Bridge 1001

COV x	COV C_o	COV D_c	Time for Diffusion Probabilistic Method (years)	Percent Difference from Average Value Solution
0.097	0.151	0.806	44	64%
0.097	0.151	0.500	50	44%
0.097	0.151	0.250	56	29%
0.097	0.151	0.150	59	22%
0.050	0.050	0.050	67	7%
0.010	0.010	0.010	70	3%
0.000	0.000	0.000	70	3%

Table 9 – Input Variables Contributing to Lowest 12th Percent of Calculated Times

Summary of Input Variables Contributing to the Time for Diffusion to Corrosion Initiation, Bridge 1001

Method	x (mm)			C_o (kg/m^3)			$C_{(x,t)}$ (kg/m^3)			D_c (mm^2/year)			Time
	min	mean	max	min	mean	max	min	mean	max	min	mean	max	
Pboot	45	59	76	1.6	2.5	3.7	0.6	0.85	1.17	22	67.9	202	Lowest 12%
Sboot	45	59	71	1.8	2.5	2.8	0.6	0.84	1.16	20.6	68.9	90	Lowest 12%
Pboot	41	62	87	1.3	2.3	3.8	0.6	0.9	1.2	0.1	25	202	Total
Sboot	46	62	71	1.8	2.3	2.8	0.6	0.9	1.2	10.2	28	90	Total
Actual	46	62	71	1.8	2.3	2.8	0.6	0.9	1.2	10.2	28.1	90.0	

The total population of data sampled during the simulation agrees well with the actual data collected from the bridge decks. As expected, the range of values sampled for the parametric bootstrap is longer than for the simple bootstrap. This implies that the simulation correctly samples the original data. The input variables that contribute to the lowest 12 percent of all of the estimated times for diffusion to corrosion initiation tend to be from the side of the input distribution that predicts the shortest time for diffusion. For example, the average of all of the cover depth measurements used in the simulation is 62 mm (2.4 in). The average of the cover depth measurements that contribute to the lowest 12 percent of the time for diffusion is only 59 mm (2.3 in).

The trend is especially pronounced for the diffusion coefficient. The average of all of the diffusion coefficients used in the simulation (for Bridge 1001) is 28.1 mm^2/year (0.044 in^2/year), while the average of the diffusion coefficients that contribute to the lowest 12 percent of the times for diffusion is approximately 68 mm^2/year (0.11 in^2/year). As noted previously, the prediction of the time for diffusion to corrosion initiation is highly sensitive to the diffusion coefficient, and large values of the diffusion coefficient predict very small times for diffusion to corrosion initiation. In addition, the coefficient of variation for the diffusion coefficient is larger than the coefficients of variation for the other input variables, so more extreme values are expected to be in the population of the diffusion coefficient than the other input variables. Therefore, it is reasonable to say that shorter times for diffusion predicted by the probabilistic method, as compared to the average value method, can be attributed in a large part to the influence and variability of the diffusion coefficient. The variability of the other input variables contribute to a lesser degree.

To determine whether the influence of the variability is a reflection of the actual behavior of a bridge deck, or simply a consequence of the probabilistic method, the model was validated using historical service life data for bridge decks in Virginia.

Validation of the Model

Bridges Built Between 1968 and 1972 (Newlon Study)

The average service life of bridge decks in Virginia has been reported to be 36 years with a standard deviation of 13 years, based on a normalized projection from a set of bridges built between 1968 and 1972 (Weyers et al. 1994). A review of updated service life data from the same set of bridges indicates that a bridge deck will be in service an average of 33 years before receiving either a polymer or concrete overlay (see Table 7). However, since polymer overlays are often installed for preventative maintenance on bridge decks that are in relatively good condition, it was not clear if bridges that received polymer overlays actually reached their end of functional service life as this current study has defined it. For this reason, bridge decks that received polymer overlays were not used to validate the model. The projected service life based on the updated data, excluding those that received polymer overlays, is 38 years with a standard deviation of 7.9 years.

In addition, it was observed that more bridges on interstate and U.S. routes received polymer or concrete overlays than those on Virginia routes. Therefore, the bridges were separated into interstate routes, U.S. routes, and Virginia routes. The distribution of the bridges is presented in Table 6, and the normalized projected service life for each category is presented in Table 7.

Approximately 40 percent of the bridges included in the original Newlon study are located on rural routes. The average time until these bridge decks receive concrete overlays is 102 years, and the average time until these bridge decks receive either polymer overlays (likely for preventative maintenance) or concrete overlays is 45 years. The service lives projected for these rural bridges are substantially longer than for the bridge decks on interstate and U.S. routes, probably because the rural routes typically receive fewer applications of deicer salts because of their remote location and lower traffic volume. Because the projected service lives of the rural structures was much longer than for the other structures, they were assumed to be of little concern and were not used to validate the model.

Of the 129 bridge decks included in the original Newlon study, 76 were found on interstate or U.S. routes. Of those 76 bridge decks, 29 received polymer overlays and were excluded. Of the remaining 47 bridge decks, 15, or 31.9 percent, received concrete overlays. The projected service life for these 47 bridge decks is 34 years, with a standard deviation of 7.1 years (see Table 7).

Bridges Built Between 1981 and 1994 (Current Study)

Of the 10 bridge decks included in this study, 4 had measured surface chloride concentrations that were below 0.73 kg/m^3 (1.2 lb/cy) on average. Because of the low surface chloride concentrations on these bridges, times to first repair and rehabilitation calculated for these bridges were well above 100 years. The low surface chloride concentration of these bridge decks suggests that they are located on routes with low traffic volumes or at remote locations. For this reason, they were treated as Virginia routes and, like the bridges on rural routes from the Newlon data set, were excluded from the validation analysis. For the remaining 6 bridge decks, the median time for diffusion to corrosion initiation, as determined by the probabilistic method, was 43 years. The median time to diffusion as determined by the average value method was 61 years.

The bridges built between 1968 and 1972 were constructed using bare reinforcement. Most of the bridges constructed between 1981 and 1994 were constructed with ECR. To compare the predicted service life of the bridges constructed between 1981 and 1994 to the historical service life data from the bridges constructed between 1968 and 1972, it was assumed that the time for corrosion deterioration to the end of functional service life for the bridges constructed between 1981 and 1994 was that of bare reinforcement, or approximately 4 years. The resulting median time to rehabilitation of the six bridge decks was 43 years for diffusion plus 4 years for corrosion deterioration (assuming bare reinforcement), or 47 years to the end of functional service life based on the probabilistic method. The resulting median time to rehabilitation was 65 years based on the average value solution.

Comparison

Although the bridges included in the Newlon study were built under a different specification than the bridges included in the current study, the as-built cover depths were very similar, as presented in Table 10. Because both sets of bridges were randomly sampled throughout the state, it is assumed that the average surface chloride concentration is the same for both sets of bridges. Likewise, the average chloride initiation concentration is assumed to be the same for both sets of bridges. The two sets of bridges were built under different specified maximum w/c ratios, and therefore the diffusion coefficients were not assumed to be equal for the two sets of bridges. Because there is very little difference between the two sets of bridge decks in terms of the average as-built clear cover depths, the assumed surface chloride concentrations, the assumed chloride initiation concentrations, and the assumed times for corrosion deterioration, the effect of the differing w/c ratio can be evaluated in terms of the difference in the probable diffusion coefficients.

Table 10 – Clear Cover Depth for Bridges

		As-Built Cover Depth	
Years Built	Specified Cover Depth (mm)	Average (mm)	Standard Deviation (mm)
1968 to 1972	50.8 to 63.5	61.0	12.4
1981 to 1995	63.5 to 76.2	65.0	8.9

The projected normalized mean service life of the bridges on interstate and U.S. routes, built between 1968 and 1972 is 34 years (excluding those that received polymer overlays). The median time to rehabilitation for the bridge decks built between 1981 and 1994, excluding those with very low C_o values, is 47 years based on the probabilistic method and 65 years based on the average value method. The difference in service life between the two sets of bridge decks is 13 years and 31 years for the probabilistic and average value methods, respectively.

Because the other input variables are shown, or assumed, to be nearly equal, the differences in the predicted service life of the two sets of bridges may be explained by the differences in their average diffusion coefficients. The average D_c for the six bridge decks included in this analysis that were built between 1981 and 1994 is approximately 32 mm^2/year (0.050 in^2/year). According to the results of the sensitivity analysis discussed earlier, the average D_c for the bridge decks built between 1968 and 1974 would have to be approximately 1.3 times the average D_c for the bridges built between 1981 and 1994 to account for the 13-year difference in the service life estimates using the probabilistic method. The average D_c for the bridge decks built between 1968 and 1974 would have to be approximately 2.0 times the average D_c for the bridges built between 1981 and 1994 to account for the 31-year difference in the service life estimates using the average value method.

A review of the available literature was performed to determine the effect of the w/c ratio on the diffusion coefficient. Three literature sources were identified that present the influence of the w/c ratio on the diffusion coefficient of laboratory-prepared specimens determined by various methods (Stanish 2000; Page et al. 1981; Goto and Roy 1981). In all cases, the concrete specimens were laboratory prepared and underwent diffusion in the saturated condition. The temperature of the tested specimens varied for each study. The diffusion coefficient is a function of the degree of saturation, where higher saturation levels typically produce higher diffusion coefficients. Therefore, the diffusion coefficient for saturated lab specimens is typically larger than the diffusion coefficient for specimens collected from bridge decks, since bridge decks rarely exist in the saturated condition.

Data from all three sources provided some indication of the expected difference in D_c between concrete mixes with w/c ratios of 0.45 and 0.47, although some interpolation or extrapolation was necessary for two of the sources. Also, because the diffusion coefficients were determined using slightly different methods and under slightly different conditions, direct comparison was not attempted. Instead, the ratio of D_c at a w/c equal to 0.47 over D_c at a w/c equal to 0.45 was determined in an attempt to negate the influence of differences in the temperature, level of saturation, and testing method. The results are presented in Table 11. It is clear that although the absolute magnitude of D_c estimated using data from the three sources differs, the ratio of $D_{c(0.47)}/D_{c(0.45)}$ is nearly equal for all three sources. The average ratio of D_c for a w/c equal to 0.47 over a w/c equal to 0.45 is 1.12.

Table 11 – Diffusion Coefficients Based on w/c

Stanish (2000)

w/c	Diffusion Coefficient (mm^2/year) 28 day @ 20° C	20 years @ 20° C
0.45	330	109
0.47	369	121
$D_{c(0.47)}/D_{c(0.45)}$	1.12	1.11

Goto and Roy (1981)

w/c	Diffusion Coefficient (mm^2/year) 27° C
0.45	341
0.47	391
$D_{c(0.47)}/D_{c(0.45)}$	1.15

Page et al. (1981)

w/c	Diffusion Coefficient (mm^2/year) 15° C	25° C
0.45	57	112
0.47	64	123
$D_{c(0.47)}/D_{c(0.45)}$	1.12	1.10

Earlier it was noted that the average D_c for the bridges built between 1968 and 1972 would have to be approximately 1.3 times larger than the D_c for the bridges built between 1981 and 1994 to account for the 13-year difference in the predicted time to rehabilitation using the probabilistic method at the most conservative level of chloride initiation, 0.6 to 1.2 kg/m^3 (1.0 to 2.0 lb/cy). The average D_c for the bridges built between 1968 and 1972 would have to be approximately 2.0 times larger than the D_c for the bridges built between 1981 and 1994 to account for the 31-year difference in the predicted time to rehabilitation using the average value method at the most conservative level of chloride initiation, 0.6 to 1.2 kg/m^3 (1.0 to 2.0 lb/cy). Based on the w/c data in the literature, the sensitivity of D_c, and assuming a time to cracking of 4 years, the change in w/c ratio between the two sets of bridges would account for approximately 40 percent of the additional service life determined by the probabilistic method and approximately 15 percent of the additional service life determined by the average value method.

Based on the sensitivity of the cover depth discussed earlier, the slight increase in as-built clear cover depth of the newer set of bridges would cause an increase in the time to rehabilitation of approximately 5 years, which is approximately 40 percent of the time to rehabilitation determined by the probabilistic method and approximately 15 percent of the time to rehabilitation determined by the average value method, assuming a time to cracking of 4 years.

Therefore, the combined effect of the decreased D_c caused by the lower w/c ratio and the slightly increased clear cover depth accounts for nearly all of the 13-year difference in time for rehabilitation between the historical data and the probabilistic method. The remainder can be attributed to inaccuracies associated with the projected normalized service life of the older bridges and inaccuracies associated with the probabilistic model. The combined effect of the decreased D_c and slightly larger cover depth accounts for only about 30 percent of the difference in the time to rehabilitation between the historical data and the average value method.

Based on these observations it is reasonable to conclude that the probabilistic method provides results that most accurately reflect the behavior of the bridge decks included in this study and that the additional service life expected for the set of bridges built between 1981 and 1994, when they are assumed to have times for corrosion deterioration similar to bare reinforcement, can be explained by the reduction in the w/c ratio and slight increase in the cover depth.

It should be noted that if the time to cracking was greater than 4 years, the difference between the service lives of the two sets of bridges would be larger. For instance, if the time to cracking were 10 years, instead of 4 years, the median time to rehabilitation for the set of bridges built between 1981 and 1994 would be 53 years based on the probabilistic method and 71 years based on the average value method. The difference between the service lives of the two sets of bridges would be 19 years and 37 years for the probabilistic and average value methods, respectively. When the time to cracking is assumed to be 10 years, the combined effect of the reduced D_c and increased cover depth would account for approximately 60 percent of the additional service life determined by the probabilistic method and approximately 30 percent of the additional service life determined by the average value method.

Simulated Supplementary Cementitious Materials

Work by Zemajtis (1998) on the service life of concrete bridge structures showed that D_c was generally lower for bridge decks containing supplementary cementitious materials (SCM) than for bridge decks with ordinary portland cement (OPC) concrete. Multiplication factors of 1/1.7 and 1/4.6 were suggested to reduce D_c determined for OPC concrete to the probable D_c if the concrete contained SCM. The multiplication factors used field and laboratory data and accounted for 90 percent and 50 percent of the measured D_c values, respectively (Zemajtis 1998). Table 12 shows the average of the reduced D_cs for each bridge and the time to first repair and rehabilitation determined from the model for each bridge using the range of 0.6 to 1.2 kg/m^3 (1.0 to 2.0 lb/cy) for $C_{(x,t)}$. The D_c for Bridge 1136 was not reduced because the deck concrete contained fly ash, based on petrographic analysis.

The median time to rehabilitation (assuming 4 years for the time for corrosion deterioration) for the six bridge decks included in the validation study was 62 years for the reduced D_c that accounted for 90 percent of the SCM data and 163 years for the reduced D_c that accounted for 50 percent of the SCM data. Assuming an equal coefficient of variation, the normalized times to rehabilitation predicted for the simulated SCM decks were plotted along with the normalized times to rehabilitation of the decks built between 1968 and 1972 and those built between 1981 and 1994. The results are presented in Figure 29.

It is clear from the table and figure that the reduced apparent diffusion coefficient that supplementary cementitious materials provide can significantly increase the service life of bridge structures, especially if the added SCM reduces the D_c below approximately 10 mm^2/year (0.016 in^2/year).

Table 12 – Time for Diffusion to Corrosion Initiation for Simulated SCM

SCM 90 Percent Probability

Structure No.	Average Reduced D_c (mm^2/year)	Parametric Bootstrap % Corroded	Parametric Bootstrap 2.5 % (years)	Parametric Bootstrap 12% (years)	Simple Bootstrap % Corroded	Simple Bootstrap 2.5% (years)	Simple Bootstrap 12% (years)
1015	30.4	100	11	16	100	12	16
1004_3	23.1	100	31	46	100	33	44
1136	16.8	100	29	42	100	29	43
1001	16.5	100	42	76	100	40	77
1019	16.9	99	45	72	100	49	69
2262	16.4	96	48	88	92	52	82
2021	3.1	27	-	-	27	-	-
1004_6	6.5	19	-	-	18	-	-
6037	6.6	7	-	-	8	-	-
6128	23.0	0	-	-	0	-	-

SCM 50 Percent Probability

Structure No.	Average Reduced D_c (mm^2/year)	Parametric Bootstrap % Corroded	Parametric Bootstrap 2.5 % (years)	Parametric Bootstrap 12% (years)	Simple Bootstrap % Corroded	Simple Bootstrap 2.5% (years)	Simple Bootstrap 12% (years)
1015	11.2	100	29	43	100	32	42
1004_3	8.6	100	82	122	100	89	117
1136	16.8	100	29	42	100	29	43
1001	6.1	100	113	203	100	107	206
1019	6.3	99	120	196	100	134	187
2262	6.0	96	128	234	91	142	219
2021	1.1	26	-	-	27	-	-
1004_6	2.4	19	-	-	17	-	-
6037	2.4	7	-	-	8	-	-
6128	8.5	0	-	-	0	-	-

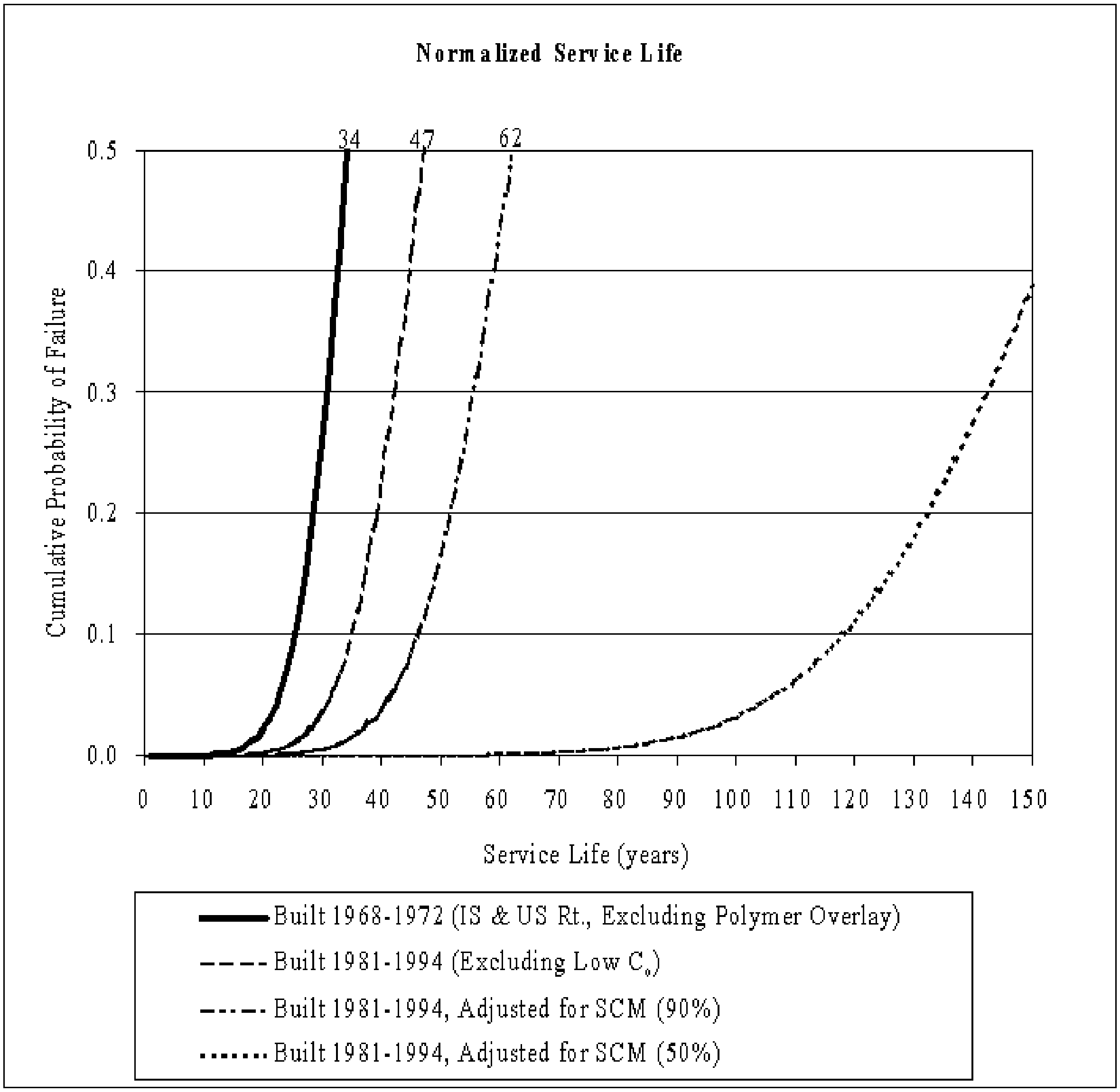


Figure 29 – Normalized Service Life Estimates

CONCLUSIONS

The following conclusions are made based on the results of the study:

- Existing models that predict the time to first repair and rehabilitation of bridge decks subject to chloride-induced reinforcement corrosion can be modified to incorporate the natural variability associated with bridge deck construction, environmental exposure conditions, and reinforcement corrosion using statistical computing techniques.

- For the 10 Virginia bridge decks included in this study, the time to first repair and rehabilitation predicted by the probabilistic method was shorter than the time to first repair and rehabilitation predicted by the average value method. The difference was primarily attributed to the variability of the apparent diffusion coefficient and the sensitivity of the time to first repair and rehabilitation to the apparent diffusion coefficient. For input variables with low coefficients of variation, the average value method and the probabilistic methods provide results that are similar.

- The fact that the parametric and simple bootstrap methods provide results that match well for each bridge deck suggests one of two conclusions: First, the shape of the distribution of the input variables does not seriously affect the shape of the predicted time to first repair and rehabilitation. Second, the distributions used to model the input variables in the parametric bootstrap closely match the true shape of the respective populations.

- The time to first repair and rehabilitation predicted by the probabilistic method more closely matches that of historical data than the time to first repair and rehabilitation predicted by the average value solution. The additional service life expected for the set of bridges built between 1981 and 1994 can be attributed to the decrease in w/c ratio from 0.47 to 0.45 and slight increase in as-built cover depth from approximately 50 mm (2 in). to 63.5 to 76 mm (2.5 to 3.0 in).

- The normalized mean time for diffusion to corrosion initiation of 12 percent of the steel for the bridges included in this study, excluding those with very low surface chloride concentrations, is 43 years. The mean time to rehabilitation depends on the time for corrosion deterioration for ECR.

- The addition of supplementary cementitious materials added to bridge decks similar to the ones included in this study should increase the time for diffusion to corrosion initiation by at least 15 years.

RECOMMENDATIONS

The following recommendations for future research are made based on the results of the study:

- The value of the chloride initiation concentration can have a significant effect on the time to first repair and rehabilitation of bridge structures. The accuracy of predictions of the time to first repair and rehabilitation will be limited until the value, or distribution, of the chloride initiation concentration is better defined. To improve future predictions of the time to first repair and rehabilitation, research should be done to investigate the chloride initiation concentration of field structures, including those with ECR.

- Lowering the apparent diffusion coefficient of bridge decks significantly lengthens the time for diffusion to corrosion initiation. Use of supplementary cementitious materials reduces the apparent diffusion coefficient. VDOT should continue to use supplementary cementitious materials in its bridge decks.

- At this time, the time for corrosion deterioration to the end of functional service life for ECR is not known. Future research should investigate the time for corrosion deterioration for ECR and should be included in the probabilistic model developed in this study.

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