CONDITION SURVEY AND SERVICE LIFE PREDICTION OF FOUR MARINE BRIDGE SUBSTRUCTURES CONTAINING EPOXY-COATED REINFORCING BARS

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EXECUTIVE SUMMARY

Epoxy-coated steel reinforcing bars have been used to resist deterioration caused by corrosion in numerous marine bridges, many of which have been in service for more than 20 years. In the late 1980's, premature corrosion of epoxy-coated bars used in Florida marine environments was reported. As performance data of marine bridge substructures outside Florida are limited, it is uncertain whether such corrosion was the result of isolated poor coating and construction practices or because the epoxy coating is not fully effective in marine environments. To determine what level of corrosion protection is currently being provided by the epoxy coating in aged marine bridges substructures, an investigation was performed on four coastal bridges that were constructed in the mid-1980s: MacKay River Bridge near Brunswick, Georgia; Ocean Isle Bridge in Ocean Isle, North Carolina; Holden Beach Bridge in Holden Beach, North Carolina; and Atlantic Beach Bridge in Atlantic Beach, North Carolina. The investigation included a field survey, concrete and steel sampling, and laboratory analyses. An established statistical model considering distributions of concrete cover over the steel and chloride concentrations within the concrete was adapted to estimate the remaining service life of these structures. The main goals of this investigation were to obtain long-term corrosion performance data for epoxy-coated bars and to predict the service life of epoxy-coated bars in marine applications based on up-to-date data obtained from in-service structures.

The field survey did not reveal any epoxy-coated bar corrosion-induced distress on any of these four bridges after 21 years of service. Three of the four bridges were found to have limited early-age thermal or shrinkage cracking. The Atlantic Beach Bridge, however, had a significant amount of cracking (expected to affect approximately 8 percent of the total surface) that appears attributed to early-age thermal or shrinkage-related causes. Core sampling at the cracked areas revealed no corrosion of the epoxy-coated reinforcing steel.

Approximately twelve 3.75-in. cores were extracted from each bridge. The effective diffusion coefficient, D, and surface chloride content, C_s, were calculated based on the chloride concentrations measured in slices cut at various depths from these cores. Based on the chloride profiles, the chloride concentration at the depth of the steel was estimated. Both the effective diffusion coefficient and the surface chloride content typically decreased with increasing elevation and can be grouped into two zones: tidal zone (between low and high tide) and splash zone (0 to 5 ft. above high tide).

The reinforcing steel segments in the cores were extracted to allow visual estimates of damage and corrosion, and for adhesion testing and coating thickness measurement. Coating damage varied, but the average amount of damage present was under 2% for each bridge. Many bar segments, especially those from the MacKay River Bridge, have poor coating adhesion. Nevertheless, a number of epoxy-coated bar segments (14 unique bars) were found to have resisted high chloride concentrations (up to 0.156 wt%, five times the 0.030 wt% threshold normally assumed for uncoated bars). This indicates that coating adhesion is not a good indicator of epoxy-coated bar corrosion performance. Active corrosion of epoxy-coated bar segments was observed on three epoxy-coated bar segments (one from the MacKay River Bridge and two from the Ocean Isle Bridge) that had significant coating damage (2 to 5%). This coating

damage appears to have been present since construction. These bars were all from tidal zones and exposed to chloride concentrations (0.183 to 0.251 wt%) well above the threshold for uncoated bar (0.030 wt%). As no corrosion-induced distress to the concrete structure was observed during the survey and corrosion of the corroding bars is unlikely to have initiated prior to 1989 according to chloride diffusion models, it was estimated that these worst performing, damaged epoxy-coated bar segments have a corrosion propagation time, t_2, of 17 years or greater in the worst-case tidal zone exposure.

Most bars examined from these bridges had greater coating damage and lower coating thickness than admissible by current standards governing the use of epoxy-coated reinforcing steel. The coating used in the MacKay River Bridge appears to be an older generation of epoxy coating and demonstrated particularly poor performance in terms of adhesion and thickness. Modern epoxy-coated bars, which are required to have thicker coatings, should exhibit greater corrosion resistance than was observed in these four bridges.

To support service life modeling, normal distribution functions of the diffusion coefficient, surface chloride concentration and concrete cover in each zone were constructed for each bridge, based on the core samples and field measurements. Using these distributions as inputs, a finite element-based damage function (percentage of area deteriorated versus time) was developed following the Tuutti corrosion sequence (corrosion damage to the structure becomes apparent following a corrosion initiation period and a subsequent corrosion propagation time). The corrosion initiation time was calculated based on Fickian Diffusion and a chloride threshold that was assumed to be constant throughout the bridge. A corrosion propagation time (t_2) was estimated for the tidal and splash zone exposure conditions based on the site observations and experience. The estimated minimum chloride thresholds for corrosion initiation of the epoxy-coated bars varied in each bridge and ranged from 0.079 to 0.156 wt%.

Modeling calculations yielded projections consistent with the field observations. The analyses suggested that the most critical areas of these bridges would have developed significant concrete damage if uncoated bars had been used. Therefore, the use of epoxy-coated bars is likely responsible for the observed lack of deterioration of these structures. Based on the chloride content of the concrete at this time, the service life extensions are conservatively predicted to range from 7 to 15 years in tidal zones and from 20 to over 45 years in splash zone locations. However, the actual service life extensions are likely to be much higher since these coatings are expected to continue to protect the steel even as chloride contents become higher. The actual service life extensions will depend on the amount of coating damage on the installed epoxy-coated bars, future exposure conditions, and the concrete conditions (chloride diffusion coefficient, cover and extent of cracking).

The benefits of epoxy-coated bars, in terms of service life extension, vary with the specific application and environmental conditions. The concrete cover was demonstrated to have a large effect on the service life and the service life extension provided by epoxy-coated reinforcement. This finding emphasizes the importance of sufficient concrete cover, highlights the additional benefits of epoxy coating, and may explain why relatively poor performance of epoxy-coated bars in tidal zones of marine sub-structures with low concrete cover has been observed in the past.

INTRODUCTION

Epoxy-coated steel reinforcing bars have been used to resist deterioration caused by corrosion in numerous marine bridges, many of which have been in service for more than 20 years (Sagüés 1994; Reaves 1995). In the late 1980's, concrete spalling due to corrosion of several bridge substructures located in the Florida Keys and built with epoxy-coated bars was reported (Kessler 1987). These failures were attributed to epoxy coating defects and subsequent adhesion reduction. However, poor construction practices employed during the construction of these bridges may have also played a significant role in the poor performance. Sagüés and Kessler have reported that the concrete cover on the reinforcing steel in some elements of the substructure was as low as 1 in. or even non-existent due to inaccurate reinforcing steel cage positioning. Some of the concrete used in those structures had an initial chloride as high as 0.03-0.09 wt% (1.2-3.5 lbs/yd^3), which is higher than the typical chloride threshold for carbon steel reinforcement (approximately 0.03 wt% or 1.2 lbs/yd^3). Moreover, the tie wires used to build the reinforcing cages were uncoated steel wires instead of the coated wire currently recommended to protect the coating and electrically isolate adjacent bars (Kessler 1987; Sagüés 2001). These observations raise an important question: Is the Florida Keys failure an isolated case of poor performance by epoxy-coated steel reinforcing bars in a marine environment caused by poor coating and placement practices?

The available performance data of marine bridge substructures outside Florida are limited. In 1993, the Georgia Department of Transportation performed a field survey of the substructure of a marine bridge located near Brunswick, Georgia (Griggs 1993). A visual inspection was performed, and a total of six cores were extracted from tidal and splash zones on two bents. (The tidal zone is the region between low and high tide, and the splash zone is the region above high tide, sometimes capped at five feet above high tide.) While the investigators found no evidence of corrosion-induced distress in the concrete, poor adhesion was observed on two epoxy-coated bar samples taken from a column (no corrosion observed on the steel) and corrosion on a piece of epoxy-coated bar was observed at a bent where the concrete was poorly consolidated. Based on this survey, the author of the study suggested that the use of epoxy-coated bar be discontinued in Georgia provided that "improved deck concrete placement techniques, vigilant quality control and aggressive concrete cover enforcement" are realized.

In the same year, the North Carolina Department of Transportation conducted a similar survey of three marine bridges of similar ages (Reaves 1995). This survey included a visual inspection and sampling of eight cores per bridge from submerged, tidal, splash and dry zones. The chloride content was determined at selected depths in these cores. No corrosion-induced concrete distress was detected. However, it was found that the sampled epoxy-coated bar experienced light corrosion at defects in the epoxy coating (holidays, small holes, and other damaged areas). No loss of adhesion was observed. While not measured directly, it was estimated that at some bar locations, the chloride concentration could be higher than the corrosion threshold for uncoated steel bars. The investigators concluded that the epoxy-coating provided adequate protection for the steel and that its use should be continued.

To determine what level of corrosion protection is currently being provided by the epoxy coating now that the age of these four bridges exceeds 20 years, a follow-up investigation was performed. The bridge locations, construction dates, descriptions of the coating and the properties of the surrounding seawater for these bridges are shown in Table 1. The recent investigation included a field survey, concrete and steel sampling, and laboratory analyses. An established statistical model considering distributions of cover and chloride concentrations was adapted to predict the remaining service life of these structures.

FIELD SURVEY

For each bridge, the general condition of the substructure concrete was viewed at close proximity from a boat. Based on the general survey, at least two representative bents were selected for detailed inspection that included the following activities: a) physical condition survey including crack mapping, delamination survey and carbonation testing; b) rebar mapping and concrete cover measurement; c) electrical continuity check and half-cell potential measurements; and d) core sampling.

For each selected bent, a delamination survey was performed, and the location and width of any cracks present were noted. The depth of carbonation was determined using a phenolphthalein indicator applied to freshly exposed concrete surfaces. The indicator shows a bright pink color on concrete with high pH and is colorless on carbonated concrete where the pH is less than about 9.

A reinforcing steel locator was used to locate the reinforcing steel and to measure the concrete cover thickness. Within the same element surface, two 5/8-in. diameter holes were typically drilled to expose reinforcement to confirm the cover thickness. The electrical continuity between these locations was also measured using a high impedance multimeter. Resistances of less than 5 ohms were considered indicative of continuity.

Measurement of half-cell potentials, a standard method to evaluate the corrosion tendency of uncoated steel reinforcement, was performed at select locations. A survey of the reinforcing half-cell potentials requires that electrical continuity be established between the reinforcing network, which can be difficult for epoxy-coated reinforcing since the coating may electrically isolate the bars. In addition, potential readings are affected by many factors, such as the presence of sea water, temperature, carbonation, oxygen content and degree of saturation. Therefore, caution must be taken when interpreting half-cell potential data taken on marine substructures containing epoxy-coated reinforcing. To establish continuity, each bar exposed was grounded and each ground wire was connected during the half-cell potential survey.

Approximately twelve 3.75-in. diameter concrete cores were extracted at various heights from the tidal and splash zones of each bridge. Both cracked and sound concrete were sampled, and all cores were quickly sealed in plastic bags after extraction. When reinforcing steel segments were exposed during the coring process, an adhesion test was performed immediately (see description of adhesion testing in next section). Water samples were also collected from the bridge sites for chloride and pH analyses.

LABORATORY ANALYSES

The concrete cores and the steel samples they contained were characterized within seven days of the coring process. The concrete cover thicknesses and lengths of the cores were measured. The reinforcing steel segments were extracted for visual inspection, adhesion testing and coating thickness measurement.

The extent of coating damage on each extracted bar was visually determined as a percentage of bar surface area. In general, active corrosion was judged to be occurring if corrosion product was present under the coating or significant rust staining was present surrounding damaged areas of the coating.

The coating adhesion was evaluated according to the knife-peel test outlined in Report FHWA-RD-94-103, at four locations on each bar (McDonald 1995). In this test, an “X” is cut in the coating using a utility knife and the coating is peeled back. The adhesion is quantitatively evaluated by a 5-point rating system. A rating of "1" corresponds to excellent adhesion, and a rating of "5" correlates to poor adhesion.

Coating thickness was measured with a digital electromagnetic coating thickness gage. In most cases, three readings were taken from two sides of each bar segment for a total of six readings per bar segment. The average of three readings taken between consecutive deformations was considered as one measurement.

The concrete cores were sectioned to obtain slices at approximately the following depths (inches): 1/4 to 1/2; 1 to 1 1/4; 2 to 2 1/4; 3 to 3-1/4. The slices were pulverized for acid-soluble chloride content analysis according to a modified version of ASTM C1152 *Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete*. For each bridge, at least one slice at a depth of 5 inches or more was obtained and its chloride content was used as the baseline chloride concentration (C_0).

Chloride diffusion in concrete, driven by a concentration gradient, is usually described by Fick's Second Law of Diffusion:

$$\frac{dC}{dt} = D \times \frac{d^2C}{dx^2} \quad (1)$$

where C is the chloride concentration at a depth of x from the concrete surface at time t and D is the effective chloride diffusion coefficient.

If the surface chloride concentration C_s and D are assumed to be constants, the concentration C(x, t) at depth of x and time t is given by (Bentur 1997; Sagüés 2001 a):

$$C(x,t) = C_s - (C_s - C_0) \times erf\left(\frac{x}{2 \times \sqrt{D \times t}}\right) \quad (2)$$

where erf() is the Gaussian error function, and C_0 is the background or original chloride concentration.

Based on this relationship, the values of C_s and D that provided the best fit to the measured chloride concentration depth profiles were determined using a least squares fitting method. The term t was assigned as the age of the bridge. For the McKay River Bridge in Georgia, t is 22 years; for the three bridges in North Carolina, t is 21 years. With these values, the chloride concentration at any depth can be predicted for any given time at that location. Figure 1 shows an example result of this analysis.

SERVICE LIFE PREDICTION MODELING

Steel reinforcing bars (typically uncoated carbon steel) are usually passive in concrete because concrete provides a high pH (approximately 13) medium and also acts as a physical barrier isolating the steel from aggressive species in the environment. However, steel will begin to corrode once those aggressive species (e.g., chloride ions) penetrate through the concrete cover and reach a certain concentration (commonly called the chloride threshold, C_T, in the case of chloride ions). (Alternatively, corrosion could also take place if a concrete carbonation front reaches the reinforcing steel surface resulting in a decrease of pH at the steel level.) The time required for chloride to reach C_T is called **corrosion initiation time (t_1),** since it is assumed that the corrosion will initiate as soon as the threshold is exceeded. The corrosion products of steel in concrete occupy much more volume than the original steel and this introduces significant stresses in the surrounding concrete. Soon afterward, distress such as cracks or delamination in the concrete occurs and costly maintenance is required (Broomfield 1997). The time required for corrosion products to induce concrete distresses is called **corrosion propagation time (t_2)**, and the sum of t_1 and t_2 defines the total amount of time before corrosion-related damage is apparent at a given location. Figure 2 illustrates this corrosion sequence (Tuutti 1982). The service life of a structure is defined as the time until the amount of damaged area exceeds some level of acceptability.

For this investigation, a statistical model developed by Sagüés et al. (1998) was adapted to estimate the service life of the bridge substructures. In this model, concrete structures are virtually divided into finite elements of equal size. Each element is randomly assigned with a unique value of surface chloride concentration, concrete cover and diffusion coefficient based on assumed normal distribution functions for these three parameters determined from data collected during the field surveys. Using Fick's Law as described earlier, the model is used to calculate t_1, the time for the chloride concentration at the steel to exceed the chloride threshold, for each element. The propagation time, t_2, is estimated based on experience and observations. Combining t_1 and t_2, a damage function (percentage of area deteriorated versus time) is generated that can be used to predict the service life of the concrete structure.

Assumptions

For the purposes of this modeling effort, the following assumptions have been made:

1. The substructure is divided into two zones based on surface chloride concentrations (C_s) and effective chloride diffusion coefficients (D) data obtained from the chloride analyses of the sampled concrete:

    i=1, tidal zone (TZ) - the zone between low tide and high tide;
    i=2, splash zone (SZ) – the zone above high tide extending to a height of 5 ft. above high tide;

For the area of the structure located above the designated splash zone, corrosion damage is assumed to be negligible because of the much less severe chloride exposure conditions occurring there.

2. Each zone is considered to have Ni (1,2,…,j) surface elements with each element having a size of 1 sq. in. The concrete cover (cc) and the chloride surface concentration (C_s) are assumed to be constants within a given element.

3. Chloride ions move by near-flat geometry Fickian Diffusion. Each element j in zone i has one constant effective chloride diffusion coefficient (D_{ij}).The background chloride content (C_0) of the bulk concrete is assumed to be a constant value for all components of each bridge. This value is assigned based on chloride analyses of concrete samples taken 5 in. or more below the concrete surface.

4. The chloride concentration threshold (C_T) for corrosion initiation is the same everywhere in each bridge. For carbon steel, the chloride concentration threshold (C_T) is assumed to be 0.030 wt%. For epoxy-coated bar, C_T was chosen based on the chloride analysis data and condition of the extracted bars.

5. In zone i, the cumulative distribution function of elements with $cc_{ij} \leq x$ is defined as $Nc_i(x)$. The cumulative distribution functions for the modeled structural components are determined based on direct

measurements from concrete cores, field measurements through holes drilled to the steel, and cover measurements taken with a reinforcing steel locator.

6. The normal distribution of chloride surface concentration (C_s) in zone i is called PCs_i and the normal distribution of effective chloride diffusion coefficient (D) in zone i is called PD_i. These functions were estimated based on chloride analyses of concrete samples obtained from the bridges.

7. Visible damage of one element occurs when, for that element,

$$t = t_1 + t_2$$

where t_1 is the corrosion initiation time and t_2 is the corrosion propagation time. It is assumed that t_2 is the same everywhere in each elevation zone and that damage affects the entire surface of the finite element.

8. The results of the model are presented first without cracks and then the effect of cracking is examined. When cracks were included in the modeling, the following additional assumptions were made:

a) Cracks of certain density are present immediately after construction. The crack density was determined based on the field survey and varied from bridge to bridge.
b) The diffusion coefficient of concrete affected by cracks is assumed to be 10 times that of sound concrete. A crack will affect a concrete area with a width of 4 inches (2 in. on either side of the crack).
c) For each elevation zone i (i= 1 (tidal zone), i=2 (splash zone)), two different types of areas k (k=1 and 2) were considered: Area with cracks (case 1) and Area remaining (case 2).

9. The effect of the proximity of model elements to corners of the structure on the chloride diffusion process was not considered. This modeling error resulting from this choice is considered small and will not affect comparisons between uncoated bar and epoxy-coated steel bar performance.

10. Epoxy-coated bar has been reported to have longer corrosion propagation time, t_2, than uncoated bars (Sagüés 2001). For the splash zone, t_2 is assumed to be 7 years for epoxy-coated bar and 3.5 years for uncoated bars. The propagation time (t_2) assumed for uncoated bars in bridge structures has been used previously by other researchers and is widely reported in the literature (Weyers 1993; Sagüés 1994 a). For the tidal zone, as further explained in the section of this reported titled "Discussion of Findings of Field Investigations", t_2 is assumed to be 17 years for epoxy-coated bar and 12 years for uncoated bars. The propagation time assumed for the epoxy-coated bars is considered conservative based on the observations for these bridges. Future testing of these bridges may demonstrate that the actual propagation times are much larger for epoxy-coated reinforcing than assumed here due to the fact that the corrosion of epoxy bars is typically more localized than corrosion of uncoated black steel. Table 2 summarizes the assumed corrosion propagation time (t_2) for both uncoated bar and epoxy-coated bar.

11. In most cases, the entire substructure was not modeled. Instead, those elements deemed most likely to control the service life of the structure, i.e., first require repair, were selected for inclusion in the model. The specific elements chosen are discussed later in the sections describing each bridge.

Damage Function

Based on these assumptions, the following damage function can be derived for zone i

$$Nsi(t) = \int_{Dl}^{Dh}\int_{Csl}^{Csh} Ni \cdot PCs(C_s) \cdot PD(D) \cdot Nc_i(2 \cdot \sqrt{tt(t) \cdot D} \cdot inverf(\frac{C_s - C_T}{C_s - C_o}))dC_s dD$$

where Dl and Dh represent the lowest and highest values of D, respectively; Csl and Csh represent the lowest and highest values of C_s; and inverf() is the inverse Gaussian error function. When $t>t_2$, $tt(t)=t-t_2$ (t_2 represents corrosion propagation time, t is service age); otherwise, tt(t)=0. Nsi(t) is the total damaged area at zone i at age t. The total damage for both tidal and splash zones is

$$Ns(t) = \Sigma Nsi(t)$$

INVESTIGATION RESULTS AND DISCUSSION

MacKay River Bridge (Georgia)

Field Survey Results

The MacKay River Bridge, which is part of the St. Simons Island Causeway (formerly the F.J. Torras Causeway), was constructed in 1984 and has a total length of 2600 ft. with 27 bents. The pile caps of Bents 9 through 20 are located in the brackish water, while the other bents are on dry land. Most bents in the water have a spacing of 127.3 ft., and each of these bents consists of two pile caps connected by an 8 ft. tall and 35 ft. long strut wall. The MacKay River has a tide of approximately 7 ft., and the high tide is about 14 in. above the pile caps except for the channel bents, Bents 14 and 15. These two bents are slightly higher than others and high tide is 3 in. above the pile caps. Figure 3 shows a view of the bridge.

The columns and strut walls of the MacKay River Bridge were in good condition and no spalling was observed. Occasional vertical cracks, attributed to thermal or shrinkage cracking, were observed on the strut walls. With the exception of the south column of Bent 15, which had several cracks, the columns were generally crack-free. Isolated spots of corrosion product on the concrete surface were observed and attributed to exposed steel chair tips. The pile caps were completely covered with barnacles on the sides and partially covered on the top. Figures 4 through 7 show the surface condition of several representative bents.

The south columns of Bents 15 and 17 were selected for detailed inspection. Bents 12, 15 and 17 were selected for core sampling.

No delaminations were detected by sounding and none of the concrete cores, including one taken on a crack, showed any signs of delamination or other damage related to reinforcement corrosion. Carbonation tests, as illustrated in Figure 8, showed no detectable carbonation.

Half-cell potentials were measured on the south column of Bent 17 and the adjacent strut. As shown in Table 3, steel bar potentials are moderately negative (from -0.145 to -0.323 V vs. CSE) and, as commonly seen in most marine structures, decrease with decreasing elevation (distance above the high tide mark). This behavior has also been seen on structures containing uncoated steel bars and is likely due to the presence of sea water, degree of saturation of the concrete, and the availability of oxygen. The potential readings from the west face of the column were all noble (in the vicinity of 0 V vs. CSE), and bar continuity indicated that the bars were not electrically connected in this location. The lack of continuity between epoxy-coated bars is considered good for epoxy-coated steel mats, since it indicates that the coating is generally intact and individual bars are not in direct electrical contact, which can reduce the corrosion rate by minimizing cathodic corrosion reaction sites. However, poor electrical contact between bars makes the potential readings on the west face unreliable. Electrical continuity was confirmed between selected bars at the east and north faces of the column. The guidelines provided in ASTM C 876 *Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete* are not useful in interpreting half-cell results in saturated marine reinforced concrete members. Generally, potentials measured on marine structures are very negative due to the low ocean water potentials, and they do not vary greatly along each elevation. Evidence of reinforcing corrosion, in the form of local "hot spots" in the potential gradients, was not observed.

A total of twelve 3.75-in. diameter cores were obtained from this bridge. Two cores (greater than 5 in. long) extracted from a pile cap contained no reinforcing steel. All other cores, which were taken from columns, contained at least one epoxy-coated bar segment. One core was taken on a crack on the south column of bent 15. All other cores, however, were crack free. Table 4 describes the cores.

Concrete cover thicknesses measured directly from the cores were consistent with reinforcing steel locator measurements. Figure 9 compares the measured column concrete cover data with a cumulative normal distribution curve constructed using the average and standard deviation of the data set. The cover measurements taken with the reinforcing steel locator were corrected based on the direct field measurements. This distribution, truncated at 2 and 4.5 in. and normalized, was used as the model input.

Chloride Analyses Results

A chloride concentration of 0.002% was detected at two depths (6.25 and 6.5 in.) in two cores taken from this bridge. This value was assumed to be the background chloride concentration (C_0).

The calculated surface chloride concentration (C_s) and effective diffusion coefficient (D) are summarized in Table 5. These values are plotted versus elevation in Figure 10. The concrete in tidal zone has the highest C_s and D, which both decrease with increasing elevation. Core 17-1, however, has an exceptionally high surface chloride concentration for unknown reasons. Core 15-1 has a D one order of magnitude higher than the D of other cores at comparable elevations due to the presence of a crack. Chloride concentrations at bar levels were also calculated for each core based on the Fick's law curve fits, and the results are listed in Table 5.

Inspection of Extracted Epoxy-Coated Bar Segments

The epoxy coating on the bars in the MacKay River Bridge had a lusterless brown color and was consistently soft and easy to scratch. In the limited on-site coating adhesion tests, all the coatings received a rating of "5" suggesting low adhesion strength. Figure 11 shows an adhesion test performed onsite.

The visual inspection and adhesion tests that were performed in the laboratory approximately one week after the field survey showed that all the extracted bars but one (from Core 12-1) had coating damage less than 1%. Despite low adhesion, these bar segments were free of active corrosion, except for some local rust at damaged areas in the coating. Such rust may have existed before construction and was judged not to be indicative of active corrosion. The highest chloride concentration at the bar level for the bar segments that showed only this negligible corrosion was estimated to be 0.079 wt%. Despite the large variation in chloride concentration at the bar level (see Table 5), all steels had a similar dark appearance under the coating. As a result, it was concluded that the dark steel color is not associated with chloride-induced corrosion.

The bar from Core 12-1 has approximately 5% coating damage and significant corrosion staining was present on the coating surface. This damaged bar was judged to be actively corroding, although there was no significant section loss. The chloride concentration at this bar was 0.251 wt%.

The findings of these inspections are given in Table 4. Figures 12 through 16 show the condition of several epoxy-coated bar segments.

A total of 22 coating thickness measurements of three readings each were taken for the 11 extracted epoxy-coated bar segments. The average coating thickness of all the measurements is 6.2 mils, and the standard deviation is 1.4 mils. However, there were four measurements (18% of the total population) that were smaller than 5 mils, the minimum coating thickness specified at the time of construction.

Ocean Isle Bridge over Intracoastal Waterway (North Carolina)

Field Survey Results

The Ocean Isle Bridge over the Intracoastal Waterway was constructed in 1985 (see Figure 17) and has a total length of 1897 ft. with 20 bents. Bents 7 though 12 are in open water, and Bents 13 to 18 are in marsh adjacent to the channel. Each of the two channel bents (Bents 9 and 10) has a haunch with a width of 24 ft. and the top surface of these haunches is 6.5 ft. above high tide. Other bents in the water have a 10 x 29 ft. pile cap, supporting two 3.5-ft. diameter columns. The elevation of the top surface of the pile caps (6 in. above high tide) for Bents 8 and 11 is slightly higher than for Bents 7 and 12 (3 in. under high tide). The tidal zone has a height of 4.6 ft.

The bents accessible by boat, Bents 7 to 12, were visually inspected. In general, the bents (pile caps, haunches and columns) are in good condition. Figure 18 shows the overall condition of the east faces of Bents 9 and 10. Some isolated corrosion stains that likely formed due to exposed steel chair tips were observed. In addition, some minor cracks, attributed to thermal or shrinkage cracking, were observed on the top surface of pile caps of Bents 7, 8, and 12, as well as on the columns of Bent 12.

Mechanical sounding detected two hollow-sounding areas on Bents 9 and 10. The area on the south face of Bent 9 is about 6 ft. above high tide and was not accessible for further inspection. The area on the east end of the Bent 10 was found from 6 to 30 in. above high tide and the total area was about 4 $ft.^2$ Core 10-1 was taken within this area, and a smooth uncoated bar that exhibited some surface corrosion was found at a depth of 4.25 in. However, no sign of delamination was found in the core.

In addition, two significant cracks (maximum width of 15 mils) with a total combined length of 65 in. were observed on the south face of Bent 9. A corrosion stain was visible on one of these cracks, through which Core 9-3 was taken. As shown in Figure 19, a No. 8 epoxy-coated bar free of any active corrosion was found in this core at a depth of 4.9 in., but the crack that extended through the core did not originate from the reinforcing steel.

In contrast to the general good condition of the areas of the substructure reinforced with epoxy-coated bar, the columns near the bridge deck (far above high tide) have developed several spalls due to inadequate cover over apparently uncoated spiral reinforcement, as illustrated in Figure 20.

Carbonation testing revealed a maximum carbonation depth of 1/8 in.

Half-cell potentials were measured on the north half of the west column and the top surface of the pile cap of Bent 11, and on the south face of Bent 9. As shown in Table 6, the potential readings from Bent 9 (south face) generally decrease with elevation, varying from -0.190 (6 ft. above high tide) to -0.532 V (at the high tide line). For Bent 11, as shown in Table 7, the potential readings from the north face of the west column and the top surface of the pile cap were consistently low (all less than -0.44 V), regardless of elevation. Local potential differences (more than 0.1 V in magnitude) at some elevations were observed on the south face of Bent 9 and the column of Bent 11, suggesting that some limited local galvanic corrosion may be occurring. The consistent strongly negative readings from the top surface the pile cap of Bent 11 are likely due to lower availability of oxygen in this saturated concrete and are not necessarily indication of active corrosion. In light of the lack of confirmed electrical continuity, these half cell surveys give only inconclusive evidence of reinforcing corrosion.

Cores were extracted from Bents 9, 10 and 11. A total of eleven 3.75-in. cores were obtained from this bridge, seven from the channel bents (Bents 9 and 10), two from the west column of Bent 11, and three from the top surface of the pile cap of Bent 11. As described earlier, one core from Bent 9 was located on top of a crack. Table 8 describes these cores.

The design cover for channel Bents 9 and 10 is 4 in. For the other bents in water, Bents 7, 8 and 11-18, the design covers are 3, 4 and 6 in. for the columns, pile cap top surfaces and vertical surfaces, respectively. For the bottom of the pile caps, however, the design cover is 15 in. Since the cover readings obtained from a pachometer did not consistently correlate with direct measurements in drilled holes or on concrete cores, only direct cover measurements are reported in Table 9. A total of eight unique readings were obtained for components with a design cover of 4 in., and the average and standard deviation for these data are 4.1 and 0.8 in., respectively. For the columns, which have a design cover of 3 in., only three readings were collected. For the purpose of modeling the column service life, a normal distribution with a mean of 3 in. (the design value) and a standard deviation of 0.6 in. (calculated based on the same coefficient of variation measured in the channel bents) were assumed.

Chloride Analyses Results

A chloride concentration of 0.004 wt% was detected at a depth of 5.25 in. in Core 11-1. This value was assumed to be the background chloride. This core was 69 in. above high tide.

The calculated surface chloride concentration (C_s) and effective diffusion coefficient (D) are summarized in Table 10. These values are plotted versus elevation in Figure 21. The concrete in the tidal zone has the highest D, and, as expected, D decreases with increase of elevation. C_s follows a similar trend. Core 9-2 (32 in. below high tide) has a comparatively high D for unknown reasons. Though Core 9-3 (42 in. above high tide) contains a crack, the D measured there was still similar to those measured in other cores obtained from similar elevations. This is probably due to the limited exposure to chlorides at that elevation.

Chloride concentrations at bar level were also calculated for each core based on the Fick's Law curve fits. As shown in Table 10, six cores have chloride concentrations at bar level higher than the typical threshold for uncoated bars (0.030 wt%).

Inspection of Extracted Epoxy-Coated Bar Segments

From the 11 concrete cores, 15 segments of epoxy-coated bar and three segments of uncoated bar were obtained.

The epoxy coating on the bars had a lusterless green color and was harder to scratch with a utility knife than the coating on the MacKay River Bridge. Inspection performed in the laboratory showed that the epoxy coating had an average of 1.3% damage, varying from 0.3 to 3%. One bar has some light blue/green paint traces suggesting some coating repair was done during construction. Most epoxy-coated bar segments showed no sign of active corrosion. However, some rust was observed at damaged areas (Table 8). Such rust may have existed before construction and was judged not to be indicative of active corrosion. Figure 22 shows the appearance of an epoxy-coated bar segment that was located at 10 in. below high tide in Bent 9 and free of active corrosion. The steel under the coating, as shown, has a dark color typical of similar bar samples. The chloride concentration on top of this bar (4.6 in. cover) was estimated to be 0.056 wt%, approximately twice the typical threshold for uncoated bars. While most extracted bars showed no sign of active corrosion despite the high chloride concentration, significant rust staining was present on the two epoxy-coated bar segments extracted from Core 9-2. These bars were judged to be actively corroding, although there was no significant section loss. Figure 23 shows the appearance of these bars. This core was taken 32 in. below high tide, and calculation showed that the chloride concentration at the bar level of the outermost bar (4.75 in.) was as high as 0.199 wt%. These bars also have the highest level of coating damage (2 and 3%).

Knife-peel adhesion tests show that most epoxy coating had poor adhesion, and the average coating adhesion rating is 4.5.

A total of 29 coating thickness measurements were taken on 15 epoxy-coated bar segments. The average is 8.0 mils and standard deviation is 2.5 mils. Two measurements (or 7% of the measurements) were less than 5 mils, the minimum coating thickness specified at the time of construction.

Three uncoated bars were extracted from two cores. The smooth No. 3 bar from Core 10-1 had corrosion products on approximately 50% of its surfaces and the bar-depth chloride concentration was 0.021 wt%. Two uncoated deformed No. 4 bars were extracted from Core 10-3 and the bar-depth chloride concentration for each of these bars was under 0.010 wt%. There is some corrosion product on the surface of these bars, but in light of the low chloride concentration, this may have developed prior to construction and may not be active. Figure 24 shows the condition of these bars.

The findings of these inspections are summarized in Table 8.

Holden Beach Bridge over Intracoastal Waterway (North Carolina)

Field Survey Results

The Holden Beach Bridge over the Intracoastal Waterway (Figure 24) was constructed in 1985 and has a total length of 1800 ft. with 22 bents. Four bents (Nos. 10 to 13) are in water. Each of the bents consist of two pile caps (10 x 13 x 4 ft.) that are connected by a strut (3.5 x 17.2 x 3 ft.). The channel Bents 11 and 12 have an extended pile cap with an average width of approximately 40 ft. and a height of 10 ft. 2 in., and a haunch with a thickness of 6 ft. at the bottom and 9 ft. 8 in. at the top. The high tide is 14 in. below the top surfaces of pile caps of Bents 10 and 13 and about 32 in. above bottom of the haunches on Bents 11 and 12. The tidal zone has a height of 4.5 ft.

All four bents in the water were inspected, and no spalling was observed. Bents 10 and 13 were found to have a few tight cracks; Bent 11 was found to have vertical cracks approximately every 5 ft. on the south face (Figure 26), and the maximum surface width of these cracks was 0.007 in. Bent 12 had cracks on the north face; more than 10 cracks with a total length of about 70 ft. were observed. In addition, a patched area (40 in. above high tide) was observed near to the east edge of this face. The west face of Bent 12 also has several long cracks (Figure 27), with a maximum surface width of 0.007 in. Most cracks were vertical cracks and are likely not related to corrosion of the reinforcing steel.

As shown in Figure 28, the east column of Bent 11 was found to have a spall of approximately 1 $ft.^2$ due to corrosion of uncoated spiral steel. The spall site is about 10 ft. above high tide and the cover was about 1 in.

No delaminations were detected by sounding and none of the concrete cores, including the three cores taken on cracks, showed any sign of delamination or other damage related to corrosion. The depth of carbonation was consistently small and always less than 1/8 in.

Half-cell potential measurements were performed at selected locations in Bents 10 and 12. As shown in Table 11, reinforcing steel potentials from the west column of Bent 10 were mostly noble (more positive than -0.1 V) and decreased moderately at lower elevations (the lowest value was -0.241 V). Electrical continuity was established between two ground connections. The potential readings from the west face of Bent 12 largely decrease with elevation, varying from -0.016 to -0.357 V (see Table 12). Only one grounding connection was used at this location due to the difficulty in locating bars below the thick cover (design cover is 4 in.). Considering that the concrete saturation increases and oxygen availability decreases with lower elevation, these potential readings suggest that the epoxy-coated bars in the areas tested are in a passive state. The potential readings from the top surface of the strut in Bent 10, as shown in Table 11, were consistently low (between -0.6 and -0.7 V). Because of the high concrete saturation and low oxygen availability in the strut element, these readings are not considered an indication of active corrosion. Evidence of reinforcing corrosion, in the form of local "hot spots" in the potential gradients, was not observed.

A total of twelve 3.75-in. diameter cores were obtained from this bridge, five from Bent 10 (strut and column) and seven from Bent 12. While all the cores taken from Bent 10 were in sound condition, three of the seven cores from Bent 12 contained cracks. Table 13 describes these cores.

The columns of Bents 10 and 13 have a design concrete cover of 3 in., and all strut wall and channel bent faces have a design cover of 4 in. The pile caps have a design cover of 4 in. for top and vertical surfaces and a cover of at least 12 in. for the bottom surface. Due to this large cover, the readings obtained from a pachometer correlated poorly with direct measurement of either drilled holes or concrete cores. Table 14 summarizes concrete cover data collected by direct measurements. The average values were very close to the design values. The calculated averages were used together with their corresponding standard deviations to construct the normal distribution functions used as model inputs.

Chloride Analyses Results

A chloride concentration of 0.006 wt% was measured at a depth of 9 1/8 in. in Core 12-7. This value was assumed to be the background chloride (C_o). The core was located 11 in. below high tide.

The calculated surface chloride concentrations (C_s) and effective diffusion coefficients (D) are summarized in Table 15. These values are plotted versus elevation in Figure 29. The concrete in the tidal zone has the highest C_s, and as expected, C_s decreases with increase of elevation. While D largely follows a similar trend to that observed on other bridges, for unknown reason, Core 10-3 (which has no crack) has exceptionally high diffusion coefficient (0.324 in.^{2}/y). Cores 12-5 and 12-6, which both have full depth cracks, were found to have D and C_s values comparable to the sound cores taken at a similar elevation. Core 12-3, with a surface crack has a D several times higher than the D of other cores at comparable elevation. Chloride concentrations at bar level were also calculated for each core based on the Fick's Law least squares curve fitting analysis. As shown in Table 15, five cores have chloride concentrations at bar depths greater than the typical threshold for uncoated bars (0.030 wt%), yet no active corrosion was observed (Table 13).

Inspection of Extracted Epoxy-Coated Bar Segments

The epoxy coating on the bars in the Holden Beach Bridge had a lusterless green color. The coating was often hard to cut, and it was difficult to insert the blade under the coating during knife-peel (adhesion) tests, suggesting higher strength than the coating on the bars in the MacKay River Bridge.

Inspections performed in the laboratory showed that the bar coatings had an average of 0.6% damage, varying from 0 to 2%. Some bars have dark green paint traces suggesting repair materials were applied during construction. All the epoxy-coated bar segments were free of active corrosion, except some rust at pre-existing damaged areas in the coating. Such rust may have existed before construction and was judged not to be indicative of active corrosion. Coating adhesion ratings ranged from 2 to 5, and the average value was 3.6. The steel under the coating, regardless of chloride concentration at the bar level, has a dark appearance. Figure 30 illustrates the condition of these bars.

A total of 21 coating thickness measurements were taken on 11 epoxy-coated bars. The average is 8.5 mils with a standard deviation of 1.5 mils. No measurement was less than 5 mils, the minimum coating thickness specified at the time of construction.

The findings of these inspections are summarized in Table 13.

Atlantic Beach Bridge over Intracoastal Waterway (North Carolina)

Field Survey Results

The Atlantic Beach Bridge over the Intracoastal Waterway was constructed in 1985 and has a total length of 3918 ft. with 48 bents. The waterway has a tide of approximately 29 in. at the bridge location. The substructure of this bridge has four different bent configurations. Figure 31 shows a view of the bridge, and Figure 32 illustrates the bent configurations. These configurations are described as follows:

1) Type 1: Bents 1 to 14 and 44 to 48 have prestressed octagonal piles that are exposed at low tide and pile caps that are at least 5 ft. above high tide;
2) Type 2: Bents 15 to 25 and 34 to 43, with an equal spacing of 84 ft., have pile caps submerged in water. Each bent has three pile caps (8.5 x 11.5 x 3.5 ft.), which are connected by two thin struts (2.5 x 2.5 x 14 ft.) The top surface of pile caps and struts is about 19 in. above high tide. Each pile cap supports a 3.5-ft. diameter column;
3) Type 3: Bents 26 to 28 and 31 to 33, with an equal spacing of 111 ft., have a more complicated configuration. Each bent has one pile cap of 14.5 x 4.5 x 54.5 ft. On top of the cap, there are three shafts of 4 x 4 x 4.5 ft. that are connected by two strut walls of 3 x 18.5 x 4.5 ft. The top surface of the pile cap is about 22 in. above high tide;
4) Type 4: Each of the two channel bents, Bents 29 and 30, has a haunch of varying width. The top surface is about 77 in. above high tide.

Since the main objective of this investigation was to evaluate the condition of cast-in-place concrete components, including piles caps, strut walls and columns at critical areas (from tidal to splash affected areas), the bents with Type 1 configuration (prestressed piles, bents 1 to 14 and 44 to 48) were not included in this investigation. A brief inspection of other bents was performed and four bents (Bent 34 [Type 2 Configuration]; Bents 26 and 31 [Type 3 Configuration]; Bent 29 [Type 4 Configuration]) were inspected in detail. Cores were extracted from Bents 31 and 34.

Most of the inspected pile caps were found to have one or two large horizontal cracks in the tidal zone, and such cracks were likely the result of mass concrete thermal stresses that developed during or shortly after construction. Figure 33 shows the two cracks formed on the east pile cap of Bent 38.

The south face of the pile cap of Bent 26 was found to have nine vertical cracks over a width of about 55 ft. In addition, two delaminated areas were detected on the top portion of the cap near to the west face, and the total area was approximately 9 ft.^{2}. These delaminations resulted from large cracks with a width of 50 to 100 mils running parallel to and approximately 1 in. from the concrete surface (Figure 34). Since the design cover of the side faces of the caps is 6 in. and due to the lack of corrosion staining, these delaminations do not appear to be corrosion related. The south face of the strut walls (with a total width of 37 ft.) between the three shafts has three vertical cracks, and one small delaminated area (approximately 2 ft.^{2}) was detected on the west face near to the top edge (approximately 6 ft. above high tide). The cause of the delaminated area did not appear to be corrosion related, but it could not be investigated during the inspection.

On the pile cap of Bent 31, a spall was observed at one of the top corners of the east face and four cracks were located on the north face. The strut walls between the shafts have more cracks; at least nine vertical cracks and two horizontal cracks over a total length of 37 ft. On the top surface of the pile caps of Bent

34, five delaminated locations with a total area of approximately 15 $ft.^2$ were found. The surface layer is approximately 1/2-in. thick (see Figure 35) and no sign of reinforcing or corrosion-induced distress was observed. On the north face, two of the three pile caps have four vertical cracks, and the strut walls have three vertical cracks.

The south surface of Bent 29 (channel bent) has nine vertical cracks over a width of 51 ft. The maximum crack width measured was 0.025 in.

In summary, significant cracking of the massive concrete members (pile caps and strut walls) was observed, and the average spacing of the vertical cracks present on these elements was about 5 ft. The cracks and the observed delamination do not appear to be corrosion-induced distress. The columns and the shafts have much less cracking and no delamination.

Half-cell potentials were measured on the center column of Bent 34. As shown in Table 16, the readings obtained from the column were all noble (more positive than -0.150 vs. CSE), suggesting passive condition of the epoxy-coated bars (at least the bars to which electrical connection was made). The potentials measured on the top surface of the center pile cap are also given in Table 16 and were all more negative than -0.580 V vs. CSE. Such low readings, however, are not indicative of active corrosion because of the high concrete seawater saturation and low oxygen availability in this element. Evidence of reinforcing corrosion, in the form of local “hot spots” in the potential gradients, was not observed. Carbonation was found to be minimal (less than 1/8 in.).

A total of 10 cores were obtained from this bridge, three from Bent 34 and seven from Bent 31. Four cores extracted from pile caps did not contain reinforcing steel, while the other six cores contain a total of 10 reinforcing steel segments. All the cores were uncracked, except Core 34-4, which had a tight full-length crack. Cores 31-7 and 34-4 contained large voids. Table 17 describes these cores.

For the strut walls, shafts and top surface of pile caps, the design concrete cover is 4 in. For the columns, the design cover is 3 in. The side and bottom surfaces of the pile caps have a design cover of at least 6 in. and 12 to 15 in., respectively. As the cover readings obtained from a pachometer did not consistently correlate with direct measurements of drilled holes or concrete cores, only direct measurement results are reported (see Table 18). For the columns, a total of seven cover readings were obtained, yielding an average of 3.3 in. and a standard deviation of 0.4 in. For the shaft and strut walls, four unique readings were obtained and the average cover value is 3.7 in., slightly smaller than the designed value of 4 in. For the pile caps, a large concrete cover, averaging 5.3 in. for top surfaces and 6.9 in. for side surfaces of the pile caps was measured. For modeling purposes, the cover distribution on the top of Type 3 pile caps was based on an average equal to the design value (4 in.) and a 20% coefficient of variation (0.18 in. standard deviation).

Chloride Analyses Results

A chloride concentration of 0.003 wt% was measured at a depth of 8.6 in. in Core 31-2 (5 in. above high tide) and at a depth of 3.1 in. from Core 31-4 (46 in. above high tide). This value was assumed to be the background chloride concentration (C_o).

The calculated surface chloride concentrations (C_s) and effective diffusion coefficients (D) for the Atlantic Beach Bridge cores are summarized in Table 19. These values are plotted versus elevation in Figure 36. The concrete in the tidal zone has higher effective diffusion coefficients than the concrete in the splash zone. The effective diffusion coefficient appears to decrease with elevation and indicates that the most severe exposure is in the tidal zone. The calculated C_s values largely decreased with increase of elevation. Core 34-1, however, has an high C_s (0.342 wt%) for unknown reasons. Although Core 34-4 had a tight full-length crack, the effective diffusion coefficient measured in this core was comparable to those calculated from uncracked cores at similar elevations.

Chloride concentrations at the bar level were also calculated for each core based on the Fick's Law curve fits. As shown in Table 19, two cores extracted from the tidal zone have chloride concentrations at or exceeding the threshold value (0.030 wt%) for uncoated bars at the design cover depth (6 in.) In the splash zone, however, all bar-level chloride concentrations were very small (≤ 0.006 wt%).

Inspection of Extracted Epoxy-Coated Bar Segments

Among the 10 epoxy-coated steel segments extracted from the cores, four were round bars taken from spirals, while the rest were deformed bars.

The epoxy coating on the bars had a lusterless green color and was harder to scratch with a utility knife than the coating on the MacKay River Bridge. Laboratory inspection showed that the epoxy coatings had an average of 0.7% damage, ranging from 0 to 1%. One bar has some light blue/green paint traces suggesting some possible limited coating repair effort was done during construction. No sign of active corrosion was observed on any of the extracted bar segments. Chloride analyses suggest that chloride concentration at the depth of all the extracted bars were very low (<0.006 wt%). Figures 37 and 38 illustrate the condition of the bars. The findings of the core inspections are summarized in Table 17.

Knife-peel adhesion tests show that some epoxy coating retained some adhesion strength while others have low adhesion (see Figure 38). The average coating adhesion rating for the bars from the Atlantic Beach Bridge is 3.2.

A total of 19 coating thickness measurements were taken on 10 epoxy-coated bar segments. The average is 7.7 mils and standard deviation is 2.5 mils. Only one measurement (5% of the measurements) was less than 5 mils, the minimum coating thickness specified at the time of construction.

Summary and Discussion of Findings of Field Investigations

In general, the bridge substructures that were investigated are in good condition and no corrosion-induced distress from epoxy-coated reinforcing was observed. Chloride analysis indicated that a total of seventeen extracted epoxy-coated bar segments were exposed to chloride levels higher than the typical threshold for uncoated bars. However, only three bars showed signs of active corrosion and no significant steel section loss was observed in any case. These corroded bars (one from the MacKay River Bridge and two from Ocean Isle Bridge) were all from tidal zones and had the largest coating damage (2 to 5%) observed in those bridges. Moreover, chloride concentrations at these bars were extremely high (from 0.183 to 0.251 wt%). For these bars, calculations indicated that the chloride threshold for uncoated bars could have been reached by 1989. Seventeen years after the possible corrosion initiation, however, no concrete distress was observed, suggesting that the corrosion propagation time (t_2) for the badly damaged epoxy-coated bar in the tidal zone can be at least that long. For the purposes of the modeling effort, it was thus conservatively estimated that epoxy-coated bar in tidal zone has a t_2 of 17 years for all the four bridges. Bars having undamaged epoxy coating should have a much longer propagation time.

Most of the epoxy-coated bars were not corroding despite the presence of chloride concentrations well above the threshold for uncoated bars. For the MacKay River Bridge, calculation suggested that all bars were free of active corrosion at concentrations of 0.079 wt% or less; for the Ocean Isle Bridge, no bar corroded where the chloride concentration was 0.156 wt% or less; for the Holden Beach Bridge, in which no corrosion was observed, 0.100 wt% was the maximum chloride concentration estimated at the depth of a bar. For the purposes of the modeling effort, these values, below which no active corrosion was observed, were conservatively selected as chloride thresholds for epoxy-coated bar in the corresponding bridges. In the Atlantic Beach Bridge, where no active corrosion was observed, the chloride concentration at the depth of the sampled epoxy-coated bars was less than the typically-assumed uncoated bar threshold. Therefore, for this bridge, the epoxy-coated bar chloride threshold was assumed to be similar to the Ocean Isle Bridge for modeling purposes, since none of the bars with the green epoxy coating was found to be corroded below this threshold.

When interpreting the half-cell survey results in these marine substructures reinforced with epoxy-coated bars, two issues must be considered: oxygen availability and electrical continuity. In areas near and below high tide, concrete is often saturated and consequently, the oxygen availability is low. The potential readings measured near these areas was therefore consistently low (typically more negative than -0.6 V vs. CSE) and the guidelines provided in ASTM C 876 that were developed for bridge decks are not applicable. For similar reasons, the decrease of potential with elevation that was observed in these bridges is more likely due to a decrease in oxygen availability rather than a passive-active transition of the steel. In addition, as demonstrated in this investigation, electrical continuity is often hard to achieve in epoxy-coated bar reinforced structures due to the presence of the insulating coating. Overall, the half-cell potential surveys conducted during this investigation were not indicative of active corrosion. However, due to the additional issues involved in interpreting these results, this technique showed limited usefulness for evaluating the condition of epoxy-coated bar. Similar observations have also been reported by other investigators (Hededahl 1989).

Moderate to low coating adhesion was commonly observed. However, the epoxy-coated bar with low coating adhesion has resisted high chloride concentrations without corroding. According to a previous inspection of the MacKay River Bridge, the epoxy-coating on bars in the tidal zone had low adhesion as of 1993. Core 15-2 was recently taken only 2 in. above high tide. Based on the C_s, D and C_o measured at this location and assuming Fick's Law diffusion, the chloride concentration at bar level in Core 15-2 likely reached 0.030 wt% (the typical assumed threshold of uncoated bar) in 1997. The fact that this bar is still free of active corrosion and the surrounding concrete is distress free as of 2006 suggests that the epoxy coating has protected the steel for 9 years even when having a low coating adhesion. In fact, 14 epoxy-coated bars that were exposed to chloride concentration higher than the threshold of uncoated bars showed no sign of active corrosion and the majority of these coated bars had low coating adhesion. This observation is in agreement with findings reported by others (Lee 2004; Martin 1995; Vaca-Cortés 1998) and suggests that coating adhesion is not a good indicator of epoxy-coated bar performance.

The epoxy-coated bar of MacKay River Bridge has a distinctive brown color, while all three bridges in North Carolina had a consistent green color. The brown coating was much softer and probed more easily than the green coating.

Coating damage of various degrees was observed in all four bridges. The bar from Core 12-1 (brown coating) of the MacKay River Bridge has approximately 5% coating damage and significant corrosion staining was present. This highly damaged bar was judged to be actively corroding, although there was no significant section loss. The chloride concentration at this bar was very high (0.251 wt%). The other cores from this bridge had coating damage less than 1% and these bar segments were free of active corrosion, although some minor local rust was present at damaged areas in the coating. Such rust may have existed before construction and was judged not to be indicative of active corrosion. The highest chloride concentration at the bar level for these bar segments was estimated to be 0.079 wt%.

The samples from Ocean Isle Bridge (green coating) had an average of 1.3% coating damage, varying from 0.3 to 3%. Most epoxy-coated bar segments showed no sign of active corrosion, even with chloride concentrations of 0.156 wt%, approximately five times the typical threshold for uncoated bars although some light rust was present at damaged areas Significant rust staining was present on the two epoxy-coated bar segments extracted from Core 9-2. These bars were judged to be actively corroding, although there was no significant section loss. The chloride concentration at the bar level of the outermost bar (4.75 in. cover) was about 0.199 wt% and these two bars also have the highest level of coating damage (2 and 3%).

The samples from the Holden Beach Bridge (green coating) had an average of 0.6% damage, varying from 0 to 2%. Five cores have chloride concentrations at bar depths greater than the typical threshold for uncoated bars (0.030 wt%), yet no active corrosion was observed.

Samples from the Atlantic Beach Bridge (green coating) had an average of 0.7% damage, ranging from 0 to 1%. No sign of active corrosion was observed on any of the extracted bar segments and the chloride concentration at the depth of all the extracted bars was low, <0.006 wt%.

Active corrosion was limited to coated bar samples that had high amounts of damage (2 to 5 percent of the area) and high chloride contents of 0.199 to 0.251 wt%. These bars likely exceed the damage limit of 2%, specified in the ASTM A775-81 *Standard Specification for Epoxy-Coated Steel Reinforcing Bars*, which was the applicable specification at the time of construction of these bridges.

Figure 39 shows the cumulative distribution of coating thicknesses measured on samples of the four bridges and Table 20 compares the averages of the coating thicknesses. The epoxy-coated bars in the MacKay River Bridge had the lowest average coating thickness. No thickness specification was given in the NCDOT Project Special Provisions for the Ocean Isle and Holden Beach Bridges, dated March 1984. However, a specified thickness of 7 ± 2 mils is listed NCDOT Project Special Provisions for the Atlantic Beach Bridge from July 1984. ASTM A775-81, which was in effect at the time of construction of these bridges, specifies a thickness of 5 to 12 mils and that "at least 90% of measurements shall be within the specified limits." Based on the sampled bars, the coating on the bars used in the MacKay River Bridge was thin and would not meet the thickness standard in 1984. Based on this limited sampling, the coatings of the other three bridges potentially met this older standard. However, only the bars from the Holden Beach Bridge appear to have the potential to meet the requirements of the current version of the epoxy-coated bar specification, ASTM A775-06. This specifies a coating thickness of 7-12 mils and that no single thickness measurement be less than 80% of the specified minimum thickness (5.6 mils).

Table 20 also shows the average coating adhesion values for all four bridges, suggesting that the brown epoxy-coated bar of the MacKay River Bridge has poorest remaining adhesion strength. Adhesion was also typically lower in the tidal zone region, where the concrete is more saturated than in the splash zone.

The findings of this investigation suggest that the brown coating is of inferior quality to the green coating: lower coating thickness, easier to cut, lower adhesion ratings, and possibly more coating damage. This difference in coating quality perhaps explains the conflicting conclusions drawn by the Georgia and North Carolina engineers in 1993.

SERVICE LIFE MODELING RESULTS

Predictions of the service life anticipated for each bridge were generated based on the model presented earlier. In general, these predictions focused on the elements of the bridge substructure deemed most likely to first develop distress that would prompt initiation of a repair. The output of the modeling effort is a damage function, which is expressed as a damaged percentage of area versus time. The threshold of damage at which rehabilitation is necessary may vary, but 10 to 12% is often considered the maximum allowable damage at which the end of the functional service life has been reached (Koch 2002; Lee 2003). Crack-free substructures were first modeled and then the effect of cracking on service life was investigated.

MacKay River Bridge (Georgia)

In modeling the condition of the MacKay River Bridge, it is not anticipated that the pile caps will experience any relevant corrosion-induced deterioration since the concrete cover over the steel in these elements is much greater than elsewhere in the substructure. The strut walls were constructed with the same concrete and target cover as the columns and are expected to exhibit similar behavior to the columns. Therefore, for simplicity, only the columns are modeled in detail. Given the similarity between column and strut walls, it is expected that the damage function can be readily applied to the strut walls with little adjustment.
Table 21 shows the structural members considered in the modeling calculations.

As shown in Figure 10a, the C_s data largely fell into two groups corresponding to elevation: 1) tidal zone and 2) splash zone (0 to 60 in. above high tide). For the tidal zone, the distribution of C_s was assumed to follow a normal distribution with a mean equal to the average of the data collected in that region (3 points). For the splash zone, however, a linear fitting was performed, and the value at the mid-height of this region, i.e., 30 in. above high tide, based on the fitted line was used as the mean for developing the normal distribution. For both zones, the measured standard deviations of the data sets were used to define the assumed normal distributions. This approach was also used to define the normal distribution functions for the effective diffusion coefficient, D. Table 22 summarizes the distribution functions of the input parameters including D, C_s and cc.

The north column of Bent 11 has some honeycombing in the tidal zone. Corrosion of epoxy-coated bar was observed in the previous study (Griggs 1993) and is likely ongoing. The effect of this honeycombing was not considered in the modeling, since it was only observed over a small fraction of the surface area.

For uncoated bar, a chloride threshold of 0.030 wt% (1.2 lb/yd^3) was used. It is recognized that epoxy-coated bar can corrode at areas where the epoxy coating is damaged once chloride concentrations reach the threshold of uncoated bar. On the other hand, many investigations, including this one, have observed epoxy-coated bar that was free of corrosion at much higher chloride concentrations (Smith 1996; McDonald 1998). For the McKay River Bridge, an effective chloride threshold, the chloride concentration above which widespread and active corrosion of epoxy-coated bar is expected for a given level of coating damage, of 0.079 wt% (3.2 lb/yd^3) was assumed for the epoxy-coated bar. This value corresponds to the highest calculated chloride concentration at the depth of the epoxy-coated bar sampled in this bridge that was not actively corroding. No active corrosion of any epoxy-coated bar sample was observed when the bar level chloride concentration was less than this value. This effective threshold is likely a conservative value because active corrosion was only seen on the one damaged bar taken from an area having 0.251 wt% chloride. Therefore, the practical threshold value for this bridge can be somewhere between 0.079 and 0.251 wt% chloride or higher. Other threshold values approximately five and ten times the threshold for uncoated bar (0.150 wt% and 0.300 wt%) were also considered to explore the possible benefits of epoxy-coated bar in these conditions when bars that meet current specifications are used and initial bar damage is avoided.

Figures 40 and 41 show the projected damage functions for columns reinforced with both uncoated bar and epoxy-coated bar. For the tidal zone, if 10% damage is considered to be the end of the service life, columns with uncoated bar would reach this limit in 17 years and the use of epoxy-coated bar will provide a service life extension of 7 years. For the splash zone, the columns reinforced with uncoated bar will have a projected service life of 24 years, while epoxy-coated bar will add an additional 20 years.

The calculations predict that the columns in the tidal zone would have developed approximately 0.3% distress after 22 years service (through 2006), but that the amount of distress will quickly reach 10% in two more years. No corrosion-related distress has been seen to date indicating that any error in the model is conservative. The estimates are highly dependent on the assumed effective chloride threshold and propagation times. Therefore, future field surveys will be useful to calibrate this model.

Also shown in the figures are epoxy-coated bar with assumed thresholds of 0.150 wt% and 0.300 wt% (6 lb/yd^3 and 12 lb/yd^3). Using these two threshold values, the additional service life provided by epoxy-coated reinforcing was 11 and 23 years for the tidal zone and 49 and over 60 years for the splash zone, respectively. Based on these higher but practical thresholds, epoxy-coated reinforcement shows a substantial service life extension compared to uncoated bars.

Ocean Isle Bridge over Intracoastal Waterway (North Carolina)

The vertical faces of the pile caps of the Ocean Isle Bridge have a design cover of 6 in. and thus are not expected to be the location where corrosion will force a repair. The channel Bents 9 and 10 also have a large design cover (4 in.) Therefore, service life predictions were made for only the columns (3-in. designed cover) and the top surface of the pile caps of Bents 7, 8, 11 and 12. Table 24 shows the structural members considered in the modeling calculations.

The distribution functions of modeling input parameters including D, C_s and cc, as summarized in Table 25, were determined based on field inspection results and chloride data analyses.

For uncoated bar, a typical chloride threshold of 0.030 wt% (1.2 lb/yd^3) was used, and an effective chloride threshold of 0.156 wt% (approximately 6 lb/yd^3) was assumed for epoxy-coated bar. This value (see Table 10) corresponds to the calculated chloride concentration at the depth of a corrosion-free epoxy-coated bar segment sampled from this bridge. No active corrosion was observed on any epoxy-coated bar when the bar level chloride was less than or equal to this value. Another threshold value (0.300 wt%) was considered to explore the potential benefits of better quality coatings.

Figures 42 and 43 show the projected damage functions for columns reinforced with both uncoated bar and epoxy-coated bar. For the tidal zone, if 10% damage is considered to be the end of service life, columns with uncoated bar would reach this limit in 13 years and the use of epoxy-coated bar will provide an extension of 10 years. For the splash zone, the columns reinforced with uncoated bar will have a projected service life of 9 years, while epoxy-coated bar will add an additional 24 years.

The calculations predicted that the columns in the tidal zone would have developed approximately 2.6% distress after 21 years service (through 2006), but the amount of distress will quickly reach 10% in less than 3 years. Currently, no distress has been noted.

Also shown in these two figures are damage functions for epoxy-coated bar having a less conservative assumed threshold of 0.300 wt%. Using this threshold value, the additional service life compared to uncoated bars provided by epoxy-coated reinforcing was 36 years and over 70 years for the tidal and splash zones, respectively.

Holden Beach Bridge over Intracoastal Waterway (North Carolina)

The columns of Bents 10 and 13 of the Holden Beach Bridge were modeled because they have the smallest design concrete cover (3 in.), and are thus expected to deteriorate at the highest rate. For the structural components with a design cover of 4 in., the pile caps of Bent 10 and 13 were modeled. The strut walls and the channel Bents 11 and 12, which also have a design cover of 4 in., were not modeled. However, it is expected that the projected damage function for the pile caps can be readily applied to these elements. Table 27 describes the structural members considered in the calculations.

The distribution functions of modeling input parameters including D, C_s and cc, as summarized in Table 28 were determined based on field inspection results and chloride data analyses.

For uncoated bar, a typical chloride threshold of 0.030 wt% (1.2 lb/yd^3) was used and an effective chloride threshold of 0.100 wt% (4 lb/yd^3) was assumed for epoxy-coated bar in this study. This value (see Table 15) corresponds to the calculated chloride concentration at the depth of one corrosion-free epoxy-coated bar sampled from this bridge, and no active corrosion was observed on any samples when bar level chloride was less than or equal to this value. Other potential threshold values (0.150 wt% and 0.300 wt%) were also considered.

As shown in Figure 44, the model predicts that the pile caps (tidal zone) reinforced with uncoated bars would reach their service life limit (assuming a 10% damage threshold) in 21 years and the use of epoxy-coated bar would add 15 years. In the splash zone, as shown in Figure 45, epoxy-coated bar with a threshold of 0.100 wt% would add 20 years of service life for the columns and even more for the pile caps (see Table 29).

The calculations predict that the Holden Beach Bridge substructure would have developed significant amount of damage if uncoated bars were used as reinforcement by 2006: approximately 10% damage for pile caps (tidal zone) and 50% damage for the columns (splash zone). Apparently, the use of epoxy-coated bar has been the cause of the currently good performance of this bridge. Assuming a coated bar threshold of 0.100 wt%, the model predicts that the columns would have developed 0.9% damage by 2006 and 10% by 2017.

Also shown in these two figures are damage functions for epoxy-coated bar having a less conservative assumed threshold of 0.150 and 0.300 wt%. Using these threshold values the additional service life provided by epoxy-coated reinforcing was 24 years and over 60 years for the tidal zone and 57 and over 70 years for the splash zone, respectively.

Atlantic Beach Bridge over Intracoastal Waterway (North Carolina)

The Atlantic Beach Bridge has four bent configurations. Only "Type 2" bents were modeled in detail because these bents have the elements with least design cover (columns, 3 in.) In addition, because the vertical faces of the pile caps in the tidal zone have a large design cover (6 in.), only the splash zone was considered in the calculations. Table 30 shows the structural members considered in the calculations.

The distribution functions of modeling input parameters including D, C_s and cc, as summarized in Table 31, were determined based on field inspection results and chloride data analyses.

For uncoated bar, a typical chloride threshold of 0.030 wt% (1.2 lb/yd^3) was used, and effective chloride threshold values of 0.150 wt% and 0.300 wt% were examined for epoxy-coated bar. The 0.150 wt% (6 lb/yd^3) is similar to the 0.156 wt% threshold that was assumed for epoxy-coated bars from the Ocean Isle Bridge. This is justified since the steel is protected by a similar green epoxy coating and is conservative since the actual effective corrosion threshold on the Ocean Isle Bridge is likely between 0.156 wt% and 0.183 wt%.

As shown in Figure 46 and Table 32, the model predicts that the columns in the splash zone would have a service life (assuming 10% damage threshold) of 30 years if uncoated bars were used and a service life extension of over 45 more years if the epoxy-coated bar is used assuming a threshold of 0.150 wt%.

Discussion of Modeling

Modeling Approach

The concept of effective chloride thresholds was used as the basis for modeling service life in this investigation. In this approach, the epoxy-coated bar is assumed to have a single chloride threshold that is constant throughout the life of the structure. The threshold is obtained based on examination of sampled bars or by finding a value that produces the best match between the model and field performance data. Using this approach, Lee et al. generated bridge deck deterioration curves that reasonably matched field survey data from bridge decks (Lee 2003). One weakness of this approach lies in the fact that epoxy coatings deteriorate at unknown speed and past performance may not be adequate to project future performance. In addition, the projections are highly dependent on the assumed effective chloride threshold and propagation times.

In this investigation, a precise estimation of effective chloride threshold was not possible as field performance data, in terms of deteriorated areas, were not available since no corrosion-related distress has occurred in the bridges. The highest chloride concentrations at which epoxy-coated bars were free of corrosion were assumed to be the thresholds. Chosen in this way, these thresholds are certainly conservative. The models projected levels of distress comparable to what was observed during the recent survey, but future surveys will be valuable to determine the validity of the model assumptions.

Effect of Cracking

The modeled structural components of the MacKay River Bridge, Ocean Isle Bridge and Holden Beach Bridge all had limited cracks and the crack-affected area was in the vicinity of 1% of the total surface (hereafter assuming each crack affects a width of 4 in.) The Atlantic Beach Bridge, however, had more cracks, approximately one crack every 5 ft. (or approximately 8% of the total surface.) To investigate how cracks influence service life predictions, the model calculations were repeated on the MacKay River Bridge (worst case [brown coating] with limited cracking) and the Atlantic Beach Bridge assuming certain percentages of the elements were cracked. As mentioned earlier, it was assumed that diffusion coefficients of crack-affected concrete are ten times that of sound concrete.

Crack-affected area of 1%

For the 5 ft. wide MacKay River Bridge columns, it was assumed that the columns had 1% crack-affected area. This degree of cracking will correspond to three vertical full-length cracks (one per face, the fourth column face is mostly covered by a strut wall) for every seven columns. This degree of damage is a conservative estimation of the actual cracking in the MacKay River Bridge columns.

Figures 47 and 48 show the projected damage functions for columns with 1% crack-affected area in the tidal and splash zones. This limited amount of cracking apparently has negligible impact on the service life predictions for either uncoated bar or epoxy-coated bar (only the results for epoxy-coated bar with a threshold of 0.079 wt% are presented). Given the similarity in the degree of cracking, it is expected that performance of the Holden Beach and Ocean Isle Bridges will not be significantly affected by cracking either.

Crack-affected area of 8%

For the Atlantic Beach Bridge, it was estimated that the modeled substructures (pile caps and columns) had 8% crack-affected area based on field observation. As shown in Figure 49, this degree of cracking has a significant impact on the service life prediction for both uncoated bars and epoxy-coated bar. For example, the estimated service life for uncoated bars in crack-free columns was 30 years, and the presence of 8% crack-affected area reduced this prediction to 24 years. Assuming the epoxy-coated bar has a threshold of 0.150 wt%, as illustrated in Figure 49, the calculations still predict a service life longer than 75 years. Evidently, the impact of cracking on service life varies greatly with degree of cracking and corrosion resistance of reinforcing materials as well as concrete quality and cover thickness.

Performance of Epoxy-Coated Bar

The model results reported here suggested that epoxy-coated bar has provided significant service life extension to all the four bridges. As shown in Table 33, it is estimated that the use of epoxy-coated bar can provide a minimum of 7 to 15 years extension in tidal zones and at least 20 years extension in splash zones to these marine bridges. It is worth noting that the actual thresholds for epoxy-coated bar in these bridges may be higher than assumed and epoxy-coated bar may actually offer longer protection that predicted. Future inspections are needed to calibrate this model and gain additional knowledge in performance of epoxy-coated bar in aged structures.

While the magnitude of the chloride threshold of reinforcing materials is important in modeling structure performance, concrete cover and quality also plays a crucial role. Additional calculations based a set of hypothetical conditions were performed to examine the effect of concrete cover on corrosion initiation time using D and C_s data collected from the splash zone of the MacKay River Bridge containing non-specification, poor quality bars. As shown in Figure 50, the time needed for 10% of reinforcement to initiate corrosion increases dramatically with increase of concrete cover. For example, assuming a threshold of 0.079 wt%, it takes just 2 years for 10% of coated steel bar to initiate corrosion if the average cover thickness is 1 in., only 1 year longer than uncoated bar (threshold equation to 0.030 wt%). In contrast, corrosion initiation will be delayed 16 years if an average of 3.5 in. of cover is provided. This observation clearly attests to the importance of concrete cover on bridge performance and may help explain the poor performance of some elements of the Florida Keys bridges, which reportedly had concrete cover as low as 1 in. due to inaccurate placement of reinforcing steel cages (Sagüés 2001). Likewise, the importance of concrete should not be underestimated either because concrete quality influences the magnitude of chloride diffusion coefficient D. Given the reported combination of poor cover and poor quality concrete, the epoxy-coated steel bars in the Florida Keys bridges may have reasonably been expected to corrode shortly after construction even if the coating had a corrosion threshold as high as 0.150 wt%.

CONCLUSIONS

To assess the corrosion protection provided to reinforcing concrete structures in marine environments through the use of epoxy coatings, an investigation of four bridge substructures containing epoxy-coated reinforcing steel was conducted. These bridges are located in coastal Georgia and North Carolina and are 21 and 22 years old. This investigation included a field condition survey, chloride concentration measurements, coating condition evaluation, and service life modeling.

Based on this investigation, the following conclusions may be drawn:

1. No epoxy-coated bar corrosion-induced distress was observed on any of these four bridges.

2. A considerable number of epoxy-coated bars (14 unique bar segments) was found to have resisted chloride concentrations expected to corrode uncoated reinforcement (up to 0.156 wt%, five times of the threshold for uncoated bars).

3. Corrosion of epoxy-coated bar segments was observed on three epoxy-coated bars that had significant coating damage (2 to 5%) that appeared to have been present at construction. These bars appeared to not meet specifications at the time of construction and were all taken from tidal zones and exposed to very high chloride concentrations (0.183 to 0.251 wt%). No noticeable section loss or build-up of corrosion product had occurred in these bars.

4. The degree of coating damage varied greatly; however, the average values for each of the bridges were under 2%.

5. Coating adhesion is not a good indicator of epoxy-coated bar performance. Many coatings, especially the brown coating on the bars taken from the MacKay River Bridge, had poor adhesion. However, bars with no coating damage were consistently in a corrosion-free condition despite the presence of high chloride concentrations.

6. Calculations demonstrated that regardless of whether the reinforcing steel is coated or uncoated, corrosion may result in significant premature damage if sufficient concrete cover is not provided.

7. Modeling calculations using estimated effective chloride thresholds for epoxy-coated bar yielded projections consistent with the field observations to date. The tidal zone concrete is subjected to more chloride exposure than the splash zones or upper concrete elements. This highlights the importance of protecting the tidal zone steel to achieve a significant increase in the service life of the structure. The analyses suggested that the most critical areas of these bridges would have developed significant damage if uncoated bars were used. Therefore, the use of epoxy-coated bar is likely responsible for the observed lack of overall deterioration of these structures. The predicted service life extensions, based on performance to date, conservatively range from at least 7 to 15 years in the tidal zone and at least 20 years in splash zone. However, the benefits of epoxy-coated bar, in terms of service life extension, can be much greater and realistically exceed 50 years or more as demonstrated by the modeling of the Atlantic Beach Bridge. Future inspections will be valuable to improve the accuracy of these predictions.

8. The brown coating used in the MacKay River Bridge appears inferior to the green coatings used in the North Carolina bridges; the brown coating being thin, soft, and easily damaged. While superior by comparison, most coatings in the North Carolina bridges still have lower coating thickness than required by current standards governing the use of epoxy-coated reinforcing steel. This suggests that modern epoxy coatings should provide greater protection to reinforcing steel than predicted in these four bridges.

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TABLES

Table 1. Bridge information

Bridge location	Construction year	Number of Bents	Coating color	1st Inspection	2nd (recent) Inspection	Water properties	
						Chloride concentration wt%	pH
MacKay River Bridge, St. Simons Island Causeway, Georgia	1984	27	Brown	1993	June 20, 2006	1.88	7.25
Ocean Isle Bridge, Route NC 904, North Carolina	1985	20	Green	1993	July 18, 2006	1.82	7.60
Holden Beach Bridge, Route NC 130, North Carolina	1985	22	Green	1993	July 19, 2006	1.87	7.09
Atlantic Beach Bridge, Route SR1182, North Carolina	1985	48	Green	1993	July 20-21, 2006	1.88	7.62

Table 2. Assumed corrosion propagation times (years)

Zone	Uncoated bar	Epoxy-coated bar
Tidal	12	17
Splash	3.5	7

Table 3. Half-cell potentials (V vs. CSE) of the south column and strut wall of Bent 17 of the MacKay River Bridge

Distance from high tide (in.)	Horizontal distance to corner edge										
	West Face				East Face				North Face		Strut-East
	12 in.	24 in.	36 in.	48 in.	12 in.	24 in.	36 in.	48 in.	8 in.	16 in.	8 in.
58	-0.029	-0.017	-0.016	-0.023	-0.222	-0.202	-0.187	-0.196	-0.175	-0.145	-0.200
34	-0.008	-0.004	-0.009	-0.001	-0.223	-0.205	-0.211	-0.238	-0.289	-0.250	-0.265
10	-0.019	-0.021	-0.028	-0.021	-0.313	-0.294	-0.282	-0.292	-0.320	-0.323	-0.307
-2	-0.024	-0.022	-0.025	-0.025							
-12	-0.028	-0.032	-0.031	-0.031							
Electrical continuity of bars	Not connected				Connected				Connected		Not connected

Table 4. Condition of the cores and reinforcing steel segments extracted from the MacKay River Bridge

Core ID	Bent	Element	Exposure	Distance above high tide (in.)	Length (in.)	Steel size and orientation (#)[1]	Cover (in.)	Coating thickness[2] (mils)	Coating Adhesion Rating	Coating Damage (%)	Active Corrosion Present?
12-1	12	Column	Tidal Zone	-10	3 1/2 - 5	5H	3	5.6	5	5	Yes[3]
12-2	12	Column	Splash Zone	15	7 1/4	5H	3 5/8	4.7	5	1	No
12-3	12	Column	Splash Zone	38	7 1/8	5H	3 1/2	5.8	4	1	No
12-4	12	Column	Splash Zone	62	6 1/2	5H	3 3/4	5.8	3.5	0	No
15-1[4]	15	Column	Splash Zone	22	5 - 5 3/4	11V	4 1/4	6.0	5	0.3	No
15-2	15	Column	Splash Zone	2	3 5/8 - 4	5H	3	7.4	5	1	No
17-1	17	Column	Splash Zone	29	4 1/4 - 5 1/4	11V	3 3/4	6.0	5	0.3	No
						5H	3 1/4	6.3	5	0.5	No
17-2	17	Pile cap	Tidal Zone	-14	5 1/2 - 6 3/8	None	--	--	--	--	--
17-3	17	Pile cap	Tidal Zone	-14	5 1/4 - 5 3/4	None	--	--	--	--	--
17-4	17	Column	Splash Zone	16	4 1/8 - 4 5/8	5H	3	9.1	5	0	No
17-5	17	Column	Splash Zone	41	3 1/2 - 4 1/8	5H	3 3/8	5.8	5	0	No
17-6	17	Column	Splash Zone	54	4 - 4 5/8	5H	3 5/8	5.8	5	0	No

[1] H-horizontal bar; V-vertical bar;
[2] Average of 6 or more readings;
[3] Corrosion staining spread over about 20% of the bar surface;
[4] Has a vertical crack in parallel to vertical bar.

Table 5. Chloride analyses results for the MacKay River Bridge

Core ID	Distance above high tide (in.)	Calculated C_S (wt%)	Calculated D ($in.^2/y$)	[Cl^-] @ bar	
				wt%[1]	Cover (in.)
17-2	-14	0.643	0.267	--	--
17-3	-14	0.933	0.190	--	--
12-1	-10	0.760	0.214	**0.251**[2]	3
15-2	2	0.621	0.087	**0.079**[3]	3
12-2	15	0.502	0.062	0.016	3 5/8
17-4	16	0.667	0.024	0.004	3
15-1[4]	22	0.323	0.261	0.073	4 1/4
17-1	29	1.200	0.032	0.009	3 1/4
12-3	38	0.366	0.060	0.013	3 1/2
17-5	41	0.414	0.023	0.002	3 3/8
12-4	62	0.289	0.029	0.002	3 3/4

[1] Typical chloride threshold for uncoated bar is approximately 0.030 wt%;
[2] Epoxy-coated bar segment displayed active corrosion;
[3] Highest chloride concentration at which no active corrosion was observed; selected as C_T for modeling.
[4] The core contained a crack parallel to a vertical rebar, however, the bar did not corrode.

Table 6. Half-cell potentials (V vs. CSE) of the Ocean Isle Bridge

Distance above high tide (ft.)	Bent 9: South face				
6	-0.291	-0.190	-0.280	-0.393	-0.300
5	-0.204	-0.145	-0.328	-0.360	-0.406
4	-0.290	-0.387	-0.376	-0.400	-0.367
3	-0.471	-0.339	-0.454	-0.373	-0.387
2	-0.502	-0.451	-0.526	-0.442	-0.461
1	-0.440	-0.526	-0.500	-0.520	-0.469
0			-0.532		
Electrical continuity of bars	Not tested (single ground)				

Note: Measurements were horizontally separated by 1ft.

Table 7. Half-cell potentials (V vs. CSE) of the Ocean Isle Bridge

Distance to long edge (in.)	Bent 11: The top surface of the pile cap (6 in. above high tide)					Distance above high tide (ft.)	Bent 11: North face of the west column					
8	-0.526	-0.483	-0.606	-0.612	-0.622	5.5	-0.580	-0.601	-0.586	-0.444	-0.612	-0.648
20	-0.524	-0.606	-0.624	-0.611	-0.643	4.5	-0.612	-0.601	-0.587	-0.456	-0.624	-0.640
32	-0.493	-0.627	-0.620	-0.627	-0.632	3.5	-0.644	-0.655	-0.621	-0.479	-0.666	-0.687
						2.5	-0.724	-0.690	-0.632	-0.492	-0.708	-0.690
						1.5	-0.751	-0.720	-0.729	-0.685	-0.736	-0.751
						0.5	-0.691	-0.731	-0.725	-0.732	-0.738	-0.739
Electrical continuity of bars	Not tested (single ground)					Electrical continuity of bars	Not connected					

Note: Measurements were horizontally separated by 1ft.

Table 8. Condition of cores and reinforcing steel segments extracted from the Ocean Isle Bridge

Core ID	Bent	Element	Exposure	Distance above high tide (in.)	Length (in.)	Steel size and orientation (#)[1]	Cover (in.)	Coating thickness[2] (mils)	Coating Adhesion Rating	Coating Damage (%)	Active Corrosion Present?
9-1	9	Strut Wall	Tidal Zone	-10	8 - 9 1/2	6H	4 5/8	8.8	5	0.3	No
						8V	5 1/4	8.6	5	0.5	No
9-2[3]	9	Strut Wall	Tidal Zone	-32	9 1/8	6H	4 3/4	7.5	5	3	Yes[6]
						8V	5 1/8	9.8	5	2	Yes[6]
9-3[4]	9	Strut Wall	Splash Zone	42	7	8V	4 7/8	8.1	5	1	No
9-4	9	Strut Wall	Splash Zone	18	5 1/4	5H	2 1/4	7.5	3	1	No
						8V	3 1/8	8.5	5	1	No
10-1[5]	10	Strut Wall	Splash Zone	22	4 3/4	3H[7]	4 1/4	uncoated	--	--	No
10-2	10	Strut Wall	Tidal Zone	-8	6	6H	3 7/8	6.6	5	0.5	No
						6H	5 3/8	6.6	2	1.5	No
10-3	10	Strut Wall	Splash Zone	50	4 1/2 - 6	4H	4 1/2	uncoated	--	--	No
						4H	4 1/8	uncoated	--	--	No
11-1	11	Column	Splash Zone	69	5 3/4	4S	5	14.7	4.5	2	No
						(11V)	5 3/8	--	--	--	No
11-2	11	Column	Splash Zone	48	5 1/8	4S	5 1/8	7.0	5	0.3	No
						(4S)	5 1/8	--	--	--	No
11-4	11	Pile cap	Splash Zone	6	6	6V	4 3/4	7.7	3.5	1	No
						6H	5 3/8	5.5	5	0.5	No
11-5	11	Pile cap	Splash Zone	6	8 3/4	6V	4 1/4	7.5	5	2	No
						6H	5	5.5	4.5	2	No

[1] H-horizontal bar; V-vertical bar; S - spiral bar; () bar left in structure;
[2] Typically average of 6 readings;
[3] Core broke at bar level during coring, very likely introducing damage to coating;
[4] Core on a crack, however, crack did not originate from bar;
[5] Core on a hollow-sounding area;
[6] Rust staining surrounded damaged areas;

Table 9. Measured concrete covers (in.) for the Ocean Isle Bridge

Measurement	Bent 9: South face	Bent 10: North face	Bent 11: Strut wall	Bent 11: Column
Drilled holes				3.5
Cores	4.6	4.3	4.8	5.0
	4.8	3.9	4.3	5.1
	2.3	4.1		
Average	**4.1**			**4.5**
Standard deviation	**0.8**			**0.9**
Design value	**4**			**3**[1]

[1] Design value assumed for the model.

Table 10. Chloride analyses results for the Ocean Isle Bridge

Core ID	Distance above high tide (in.)	Calculated C_S (wt%)	Calculated D (in.^{2}/y)	[Cl$^-$] @ bar top wt%[1]	[Cl$^-$] @ bar top Cover (in.)
9-2	-32	0.465	0.838	**0.199**[2]	4 3/4
				0.183[2]	5 1/8
9-1	-10	0.234	0.348	0.056	4 5/8
10-2	-8	0.248	0.277	0.066	3 7/8
11-4	6	0.428	0.169	0.036	4 3/4
11-5	6	0.452	0.176	0.057	4 1/4
9-4	18	0.335	0.220	**0.156**[3]	2 1/4
				0.104	3 1/8
10-1	22	0.235	0.133	0.021	4 1/4
9-3[4]	42	0.075	0.180	0.009	4 7/8
11-2	48	0.093	0.082	0.013	3[5]
10-3	50	0.067	0.129	0.007	4 1/2
11-1	69	0.063	0.063	0.008	3[5]

[1] Typical chloride threshold for uncoated bar is approximately 0.030 wt%;
[2] Epoxy-coated bar segment displayed active corrosion;
[3] Highest chloride concentration at which no active corrosion was observed; selected as C_T for modeling;
[4] The core contained a crack parallel to but not reaching the bar;
[5] Cover ≥ 5 in. (bar extracted not likely from the first layer of reinforcement), the design cover (3 in.) was used.

Table 11. Half-cell potentials (V vs. CSE) of the Holden Beach Bridge

Distance above high tide (ft.)	Bent 10: South side of west column					Distance to long edge (in.)	Bent 10: Top surface of the strut wall (14 in. above high tide)			
7.2	-0.031	-0.055	-0.007	-0.071	-0.084	10	-0.632	-0.635	-0.653	-0.658
6.2	-0.069	-0.081	-0.032	-0.072	-0.081	22	-0.631	-0.626	-0.698	-0.661
5.2	-0.053	-0.077	0.002	-0.072	-0.115	34	-0.624	-0.642	-0.643	-0.650
4.2	-0.045	-0.088	-0.021	-0.008	-0.070					
3.2	-0.069	-0.073	-0.017	0.000	-0.061					
2.2	-0.241	-0.232	-0.209	-0.092	-0.130					
1.2	-0.217	-0.215	-0.223	-0.217	-0.210					
Electrical continuity of bars	Connected					Electrical continuity of bars	Not tested (single ground)			

Note: Measurements were horizontally separated by 1ft.

Table 12. Half-cell potentials (V vs. CSE) of the Holden Beach Bridge

Distance above high tide (ft.)	Bent 12: West face of haunch				
5	-0.067	-0.099	-0.122	-0.144	-0.060
4	-0.193	-0.227	-0.153	-0.120	-0.062
3	-0.240	-0.248	-0.194	-0.121	-0.016
2	-0.190	-0.222	-0.154	-0.133	-0.039
1	-0.201	-0.252	-0.263	-0.189	-0.122
0	-0.357	-0.305	-0.313	-0.270	-0.270
Electrical continuity of bars	Not tested (single ground)				

Note: Measurements were horizontally separated by 1ft.

Table 13. Condition of cores and reinforcing steel segments extracted from the Holden Beach Bridge

Core ID	Bent	Element	Exposure	Above high tide (in.)	Length (in.)	Steel size and orientation (#)[1]	Cover (in.)	Coating thickness[2] (mils)	Coating Adhesion Rating	Coating Damage (%)	Active Corrosion Present?
10-1	10	Strut	Splash Zone	14	5 1/2	11V	4 3/4	10.7	2	0.3	No
10-2	10	Strut	Splash Zone	14	9	11V	4 1/2	10.0	5	2	No
10-3	10	Column	Splash Zone	26	3	4S	2 5/8	7.7	4	0	No
						4S	2 5/8	7.5	--	0	No
10-4	10	Column	Splash Zone	38	6	4S	3	9.3	3	0.3	No
10-5	10	Column	Splash Zone	62	6 1/2	4S	2 7/8	7.5	2.5	0	No
						4S	2 7/8	9.1	2.5	0.5	No
12-1	12	Pile Cap	Tidal Zone	-12	1 - 2 1/4	--	--	--	--	--	No
12-2	12	Pile Cap	Tidal Zone	-21	7 3/4	6H	6 3/4	6.5	4.5	1	No
						(11V)	7 5/8	N/A	N/A	N/A	No
12-3	12	Pile Cap	Splash Zone	28	8 1/2	6H	8 1/2	7.2	5	1	No
12-4	12	Pile Cap	Splash Zone	40	12 1/8	5V	8 5/8	8.0	5	0	No
12-5[3]	12	Pile Cap	Tidal Zone	0	4	(6H)	3 1/2	N/A	2	0	No
12-6	12	Pile Cap	Splash Zone	12	2 1/4 - 4	(5H)	3 5/8	N/A	N/A	N/A	No
12-7	12	Pile Cap	Tidal Zone	-11	16	7H	7	6.4	4	2	No

[1] H-horizontal bar; V-vertical bar; S-spiral bar; () bar left in the structure;
[2] Average of 6 or more readings;
[3] Bar left in the structure; inspection was performed at job site.

Table 14. Measured concrete covers (in.) for the Holden Beach Bridge

Measurement	Bent 10: West column	Bent 10: Strut wall	Bent 11: East face	Bent 12[1]: West face
Drilled holes	3.5	4.5	4.3	4.5
	3.3		3.5	
Cores	2.6	4.8		3.5
	2.6	4.5		3.6
	3			
	2.9			
	2.9			
Average	**3**	**4.1**		
Standard deviation	**0.3**	**0.5**		
Design value	**3**	**4**		

[1] Bent 12 has four cores with cover larger than 6 in. - these readings were excluded as these bars are not likely from the first layer of reinforcement.

Table 15. Chloride analyses results for the Holden Beach Bridge

Core ID	Distance above high tide (in.)	Calculated C_S (wt%)	Calculated D (in.^{2}/y)	$[Cl^-]$ @ bar	
				wt%[1]	Cover (in.)
12-2	-21	0.600	0.156	0.076	4[4]
12-7	-11	0.367	0.182	0.059	4[4]
12-5[3]	0	0.380	0.152	0.068	3.5
12-6[3]	12	0.281	0.118	0.034	3.6
10-1	14	0.380	0.133	0.023	4.8
10-2	14	0.393	0.162	0.039	4.5
10-3	26	0.204	0.324	**0.100**[2]	2.6
12-3[3]	28	0.073	0.331	0.025	4[4]
10-4	38	0.088	0.084	0.015	3
12-4	40	0.048	0.049	0.006	4[4]
10-5	62	0.032	0.068	0.008	2.9

[1] Typical chloride threshold for uncoated bar is approximately 0.030 wt%;
[2] Highest chloride concentration at which no active corrosion was observed; selected as C_T for modeling;
[3] The core contains crack(s); however, the bar did not corrode;
[4] Cover ≥ 5 in. (bar extracted not likely from the first layer of reinforcement), the design cover (4 in.) was used.

Table 16. Half-cell potentials (V vs. CSE) of the Atlantic Beach Bridge

Distance above high tide (in.)	Bent 31: Center column							Distance to south edge (ft.)	Bent 31: Top surface of the center pile cap		
79	-0.094	-0.144	-0.155	-0.129	-0.055	-0.053	-0.009	0.9	-0.603	-0.583	-0.618
67	-0.053	-0.142	-0.122	-0.081	-0.045	-0.058	-0.082	1.9	-0.619	-0.610	-0.586
55	0.002	-0.150	-0.070	-0.025	0.000	-0.023	-0.060	2.9	-0.625	-0.599	-0.620
43	-0.004	-0.040	-0.053	0.009	0.019	0.003	0.011	3.9	-0.609	-0.626	-0.611
31	-0.030	-0.034	-0.037	-0.013	0.069	0.035	0.008				
19	0.083	0.075	0.063	0.056	0.085	0.086	0.073				
Electrical continuity of bars	Connected							Not tested (single ground)			

Note: Measurements were horizontally separated by 1ft.

Table 17. Condition of cores and reinforcing steel segments extracted from the Atlantic Beach Bridge

Core ID	Bent	Element	Exposure	Above high tide (in.)	Length (in.)	Steel size and orientation (#)[1]	Cover (in.)	Coating thickness[2] (mils)	Coating Adhesion Rating	Coating Damage (%)	Active Corrosion Present?
31-1	31	Pile cap	Tidal Zone	-8	3 1/4	None	--	--	--	--	--
31-2	31	Pile cap	Splash Zone	5	10	None	--	--	--	--	--
31-3	31	Pile cap	Splash Zone	17	9	6H	8 1/8	10.3	2	0.5	No
31-4	31	Strut Wall	Splash Zone	46	6 1/2 - 7	7V	6 5/8	6.5	2.5	1	No
						6H	5 7/8	8.0	5	1	No
31-5	31	Pile cap	Tidal Zone	-18	5 1/4 - 5 3/4	None	--	--	--	--	--
31-7	31	Pile cap	Splash Zone	22	2 3/4 - 3	None	--	--	--	--	--
31-8	31	Strut Wall	Splash Zone	35	4 1/2 - 5 1/4	4V	3 7/8	7.0	1	1	No
						4H	3 3/8	11.8	1	0	No
34-1	34	Column	Splash Zone	71	3	4S	2 7/8	6.0	5	0.5	No
						4S	2 7/8	7.0	5	0.5	No
34-3	34	Column	Splash Zone	31	3 1/2	4S	3 1/4	5.2	4.5	1	No
						4S	3 1/4	11.3	1	0.5	No
34-4[3]	34	Pile cap	Splash Zone	19	5	7V	4 3/4	5.2	5	0.5	No

[1] H-horizontal bar; V-vertical bar; S - Spiral;
[2] Typically average of 6 readings;
[3] Contains a full-depth tight crack, bar left in column and its coating was damaged while bar was extracted.

Table 18. Measured concrete covers (in.) for the Atlantic Beach Bridge

Measurement	Top surface of pile caps		Vertical surface of pile caps: Bent 31	Shaft and Strut: Bent 31	Columns: Bent 34
	Bent 31	Bent 34			
Drilled holes	5.8	5.5		3.4	3.5
				4.0	3.8
					3.8
Cores		4.8	8.1	3.4	2.9
			6.6	3.9	2.9
			5.9		3.3
					3.3
Average	**5.3**		**6.9**	**3.7**	**3.3**
Standard deviation	**0.5**		**1.1**	**0.3**	**0.4**
Design value	**4**[1]		**6**	**4**	**3**

[1] Design value assumed for model.

Table 19. Atlantic Beach Bridge chloride analyses results

Core ID	Distance above high tide (in.)	Calculated C_S (wt%)	Calculated D (in.^{2}/y)	[Cl$^-$] @ bar top	
				wt%[1,2]	Cover (in.)
31-5	-18	0.686	0.307	0.068[3]	6[4]
31-1	-8	0.471	0.234	0.029[3]	6[4]
31-2	5	0.437	0.072	0.003[3]	6[4]
31-3	17	0.461	0.029	0.003	6[4]
34-4	19	0.274	0.028	0.003	4 3/4
31-7	22	0.489	0.052	0.006[3]	4[4]
34-3	31	0.457	0.017	0.003	3 1/4
31-8	35	0.059	0.053	0.004	3 3/8
31-4	46	0.092	0.018	0.003	4[4]
34-1	71	0.342	0.014	0.003	2 7/8

[1] Typical chloride threshold for uncoated bar is approximately 0.030 wt%;
[2] Chloride concentration at all extracted bars were less than typical C_T for uncoated bars;
[3] No bar in core sample;
[4] Designed cover value was used.

Table 20. Characterization of coating

		MacKay River Bridge	Ocean Isle Bridge	Holden Beach Bridge	Atlantic Beach Bridge
Average thickness (mils)		6.2	8.0	8.5	7.7
Percentage of thickness measurements less than 5 mils		18	7	0	5
Percentage of thickness measurements less than 5.6 mils[1]		36	14	0	21
Adhesion Rating[2]	Tidal zone	5.0	5.0	4.3	n/a
	Splash zone	4.8	4.4	3.4	3.2

[1] The thickness criterion specified in ASTM A775-06. A coating with a single measurement under this value is considered to fail the thickness requirement;
[2] Rating scaled defined such that "1" corresponds to excellent and "5" corresponds to poor adhesion.

Table 21. Modeled structural elements of the MacKay River Bridge

Zone	Number of Columns	Height (in.)	Total area ($ft.^2$)
Tidal	20	14	450
	4	3	
Splash	24	60	2220

Table 22. Model inputs for the MacKay River Bridge

Zones		D ($in.^2/y$)	C_s (wt%)	cc (in.)
Tidal	Average	0.223	0.779	3.5
	Standard deviation	0.039	0.146	0.3
Splash	Average	0.045	0.471	3.5
	Standard deviation	0.025	0.148	0.3

Table 23. Model projected service lives (years)[1] for the MacKay River Bridge

	Uncoated bars	Epoxy-coated bar		
C_T (wt%)	0.030	**0.079**[2]	0.150	0.300
Tidal	17	**24**	28	40
Splash	24	**44**	73	>75

[1] Assume 10% damage as end of service life;
[2] The C_T for epoxy-coated bar estimated based on observation.

Table 24. Modeled structural elements of the Ocean Isle Bridge

Zone	Number of Columns (height)	Number of Pile caps	Total area (ft.^{2})
Tidal	4 (3 in.)	4 (top surface only)	553
Splash	4 (54 in.)	4 (top surface only)	959
	4 (60 in.)	None	

Table 25. Model inputs for the Ocean Isle Bridge

Zones		D (in.^{2}/y)	C_s (wt%)	cc (in.)	
				Columns	Top of pile caps
Tidal	Average	0.490	0.316	3	4.1
	Standard deviation[1]	0.163	0.129	0.6	0.8
Splash	Average	0.150	0.238	3	4.1
	Standard deviation	0.050	0.167	0.6	0.8

[1] Assumed to be 1/3 of the average value

Table 26. Model projected service lives (years)[1] for the Ocean Isle Bridge

	Uncoated bars	Epoxy-coated bar		
C_T (wt%)	0.030	0.079	**0.156**[2]	0.300
Tidal (Pile caps)	15	22	**29**	75
Tidal (Columns)	13	20	**23**	49
Splash (Pile caps)	15	30	**56**	>75
Splash (Columns)	9	19	**33**	>75

[1] Assume 10% damage as end of service life;
[2] The C_T for epoxy-coated bar estimated based on observation.

Table 27. Model structural elements for the Holden Beach Bridge (Bents 10 & 13)

Zone		Number of components	Height (in.)	Total area (ft.^{2})
Tidal	Pile cap	4	34	482
Splash	Pile cap	4	14	655
	Column	4	46	217

Table 28. Model inputs for the Holden Beach Bridge

Zones		D (in.^{2}/y)	C_s (wt%)	cc (in.) for Bents 10 & 13	
				Columns	Pile caps
Tidal	Average	0.163	0.449	3	4.1
	Standard deviation[1]	0.050	0.150	0.3	0.5
Splash	Average[2]	0.103	0.199	3	4.1
	Standard deviation	0.042	0.167	0.3	0.5

[1] Assumed to be 1/3 of the average value;
[2] Median value of the linear fit of all data collected from splash zone.

Table 29. Model projected service lives (years)[1] for the Holden Beach Bridge

Case	Uncoated bars	Epoxy-coated bar		
C_T (wt%)	0.030	**0.100**[2]	0.150	0.300
Tidal (pile caps)	21	**36**	45	>75
Splash (pile caps)	20	**53**	>75	>75
Splash (columns)	13	**33**	70	>75

[1] Assume 10% damage as end of service life;
[2] The C_T for epoxy-coated bar estimated based on observation.

Table 30. Model structural elements for the Atlantic Beach Bridge

Zone		Number of components	Total area (ft.^{2})
Splash	Pile cap	63	5552
	Column	63	2367

Table 31. Model inputs for the Atlantic Beach Bridge

Zones		D (in.^{2}/y)	C_s (%)	cc (in.) for Bents 10 & 13	
				Columns	Pile caps
Splash[1]	Average[2]	0.035	0.276	3.3	4[3]
	Standard deviation	0.021	0.184	0.4	0.8[4]

[1] Tidal zone was excluded because of large cover (designed value is 6 in.) ;
[2] Median value of the linear fit of all data collected from splash zone;
[3] Assumed values based on designed cover;
[4] Assumed to be 20% of the average.

Table 32. Model projected service lives (years)[1] for the Atlantic Beach Bridge

Case	Uncoated bars	Epoxy-coated bar	
C_T (wt%)	0.030	**0.150**[2]	0.300
Splash (pile caps)	38	**>75**	>75
Splash (columns)	30	**>75**	>75

[1] Assume 10% damage as end of service life;
[2] A C_T for epoxy-coated bar similar to that observed in the Ocean Isle Bridge was used.

Table 33. Model projected service life extension

Bridge Name	Epoxy-coated bar threshold (wt%)	Service life extension (years)	
		Tidal	Splash
MacKay River	0.079	7	20
Ocean Isle	0.156	10	24
Holden Beach	0.100	15	20
Atlantic Beach[1]	0.150	N/A	>45

[1] Prediction based on 8% crack-affected area

FIGURES

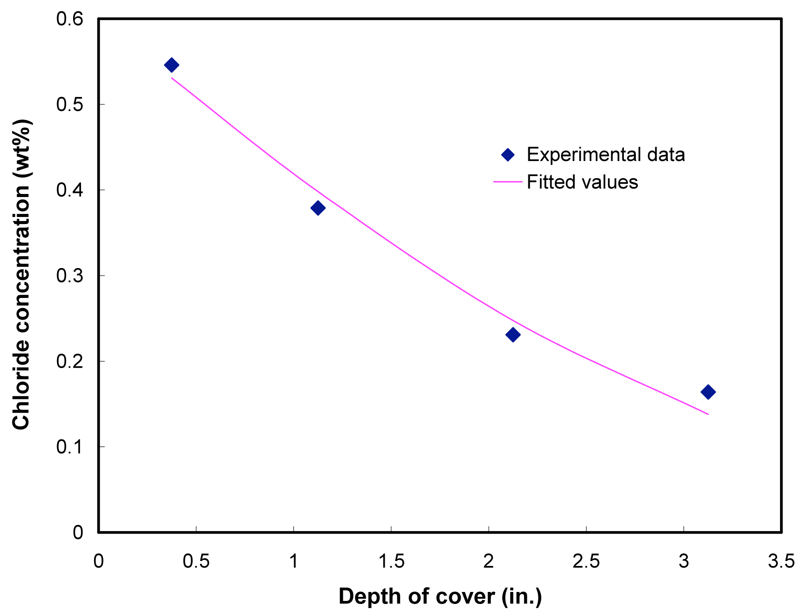


Figure 1. Typical chloride analysis result. Core chloride profiles were fitted using a least squares fit method to obtain estimation of effective chloride diffusion coefficient D and surface chloride concentration C_s.

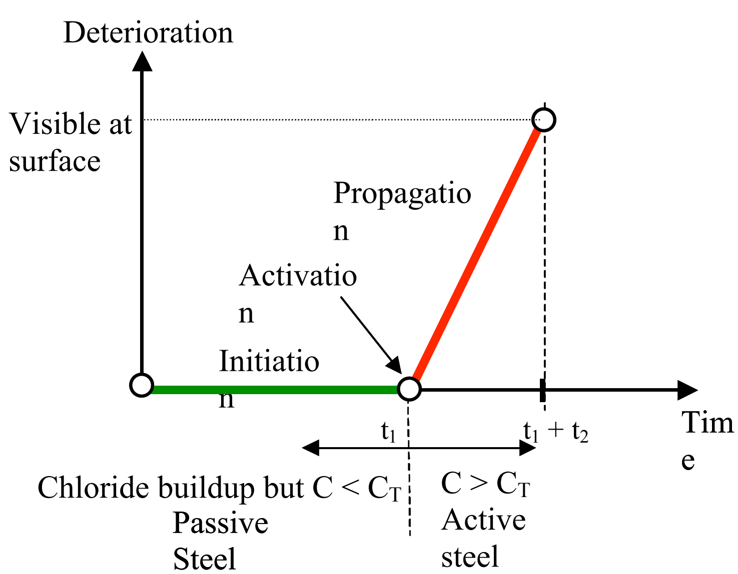


Figure 2. Corrosion sequence (Tuutti 1982)

Figure 3. A view of the MacKay River Bridge near Brunswick, Georgia.

Figure 4. Surface condition of the MacKay River Bridge Bents: east face of Bent 12.

Figure 5. Surface condition of the MacKay River Bridge Bents: west face of the North column of Bent 9 and adjacent strut wall.

Figure 6. Surface condition of the MacKay River Bridge Bents: west side of Bent 14 (some vertical cracks are visible on the strut wall).

(a)

(b)

Figure 7. Surface condition of the MacKay River Bridge Bents: a) south face of the south column of Bent 17; b) east face of the south column of Bent 15 (A long vertical crack is highlighted with white crayon).

Figure 8. Carbonation test holes on the strut wall of the MacKay River Bridge, Bent 17.

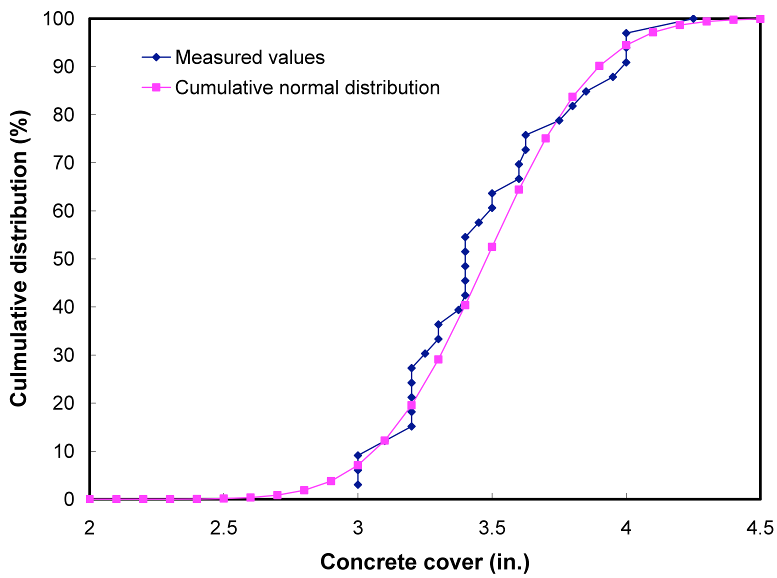
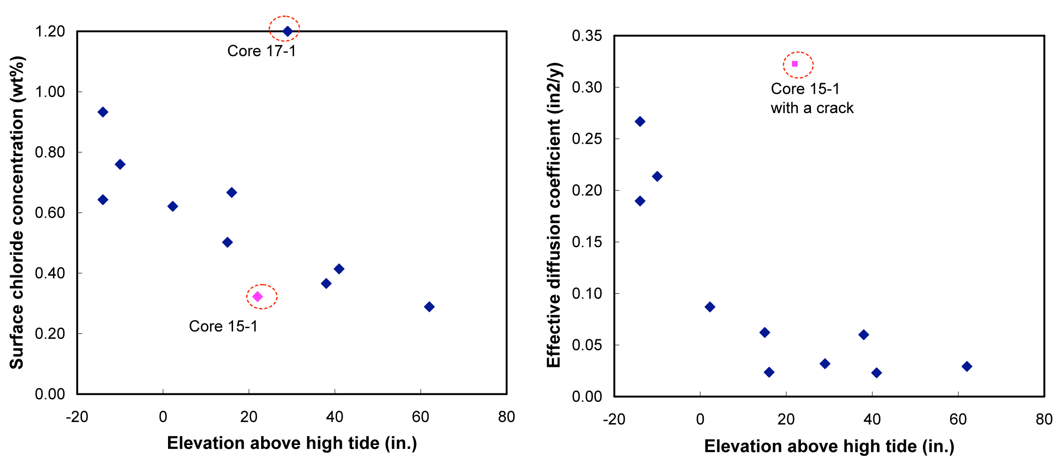


Figure 9. Measured values and fitted cumulative cover thickness normal distribution of the MacKay River Bridge substructure.



(a) (b)

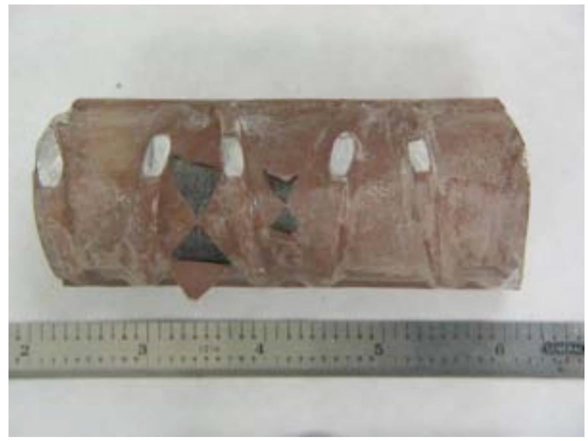
Figure 10. Calculated surface chloride C_s and effective diffusion coefficient D for the MacKay River Bridge.

Figure 11. On-site adhesion test on the MacKay River Bridge Core 17-1 (29 in. above high tide, bar-level chloride concentration is 0.009 wt%).

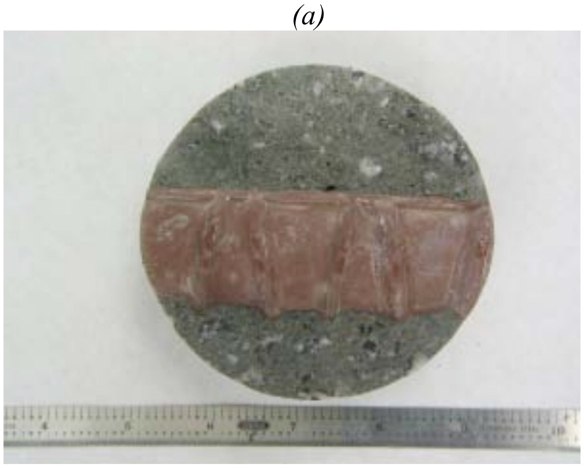
(a) *(b)*

Figure 12.Condition of the bar contained in the MacKay River Bridge Core 17-5 (54 in. above high tide, bar-level chloride concentration is 0.023 wt%).

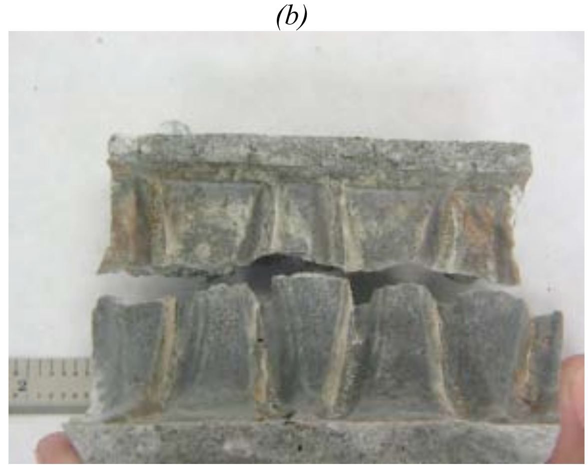
(a)



(b)

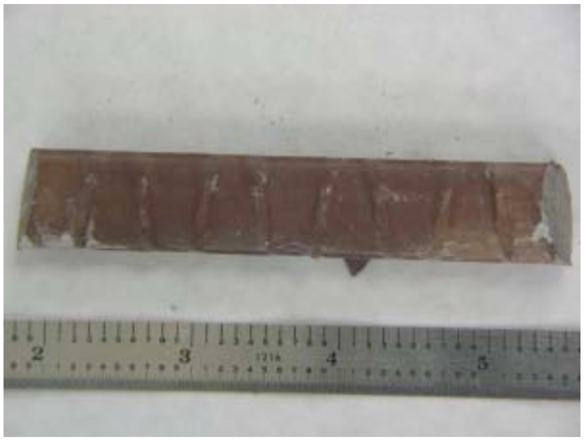


(c)



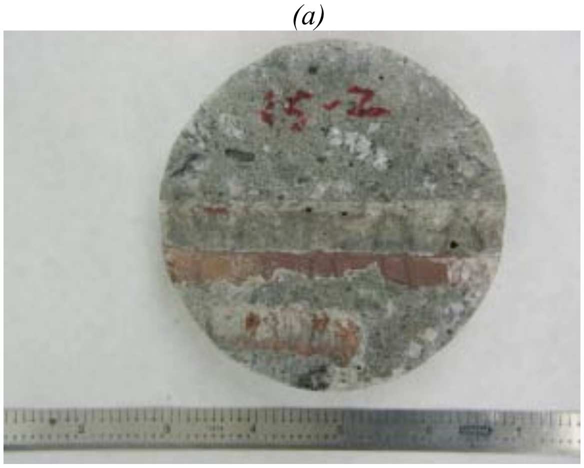
(d)

Figure 13. Condition of the bar contained in the MacKay River Bridge Core 15-1 (22 in. above high tide, a vertical crack on top of the bar, bar-level chloride concentration is 0.079 wt%): a) top surface (coating damaged during concrete slicing in laboratory); b) adhesion test result (rating=5); c) bottom surface (no corrosion); d) bar imprint of top surface (no corrosion stains).



(a)

(b)



(c)

Figure 14. Condition of the bar from MacKay River Bridge Core 15-2 (2 in. above high tide, bar-level chloride concentration is 0.079 wt%): a) surface condition; b) adhesion test result (rating=5); c) bar imprints (no corrosion stains).

Figure 15. Condition of steel under coating of the MacKay River Bridge Core 12-4 (62 in. above high tide, bar-level chloride concentration is 0.002 wt%).

(a) *(b)*

Figure 16. Condition of the Bar contained in the MacKay River Bridge Core 12-1 (tidal zone, bar-level chloride concentration is 0.251 wt%): a) top surface showing corrosion staining; c) adhesion test result (rating=5).

Figure 17. A view of the Ocean Isle Bridge, Ocean Isle, North Carolina.

(a) (b)

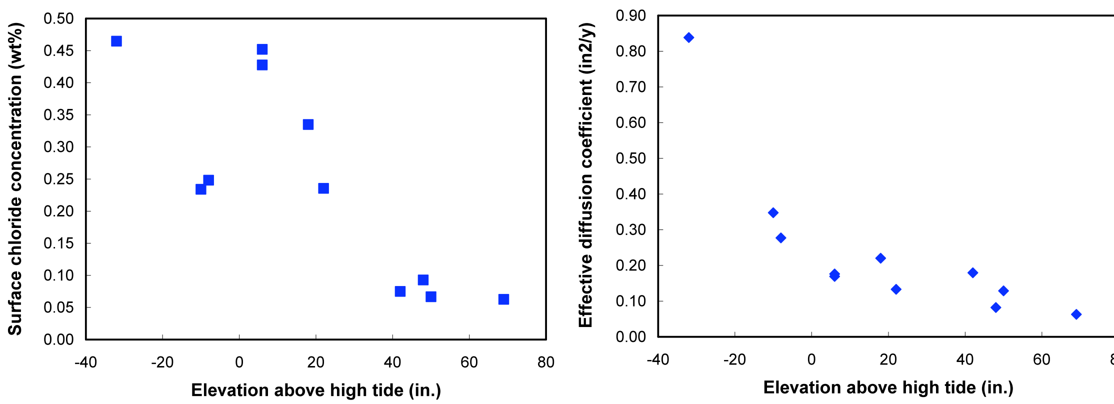
Figure 18. Condition of the Ocean Isle Bridge bents: a) east face of Bent 9; b) east face of Bent 10.

(a) (b)

(c) (d)

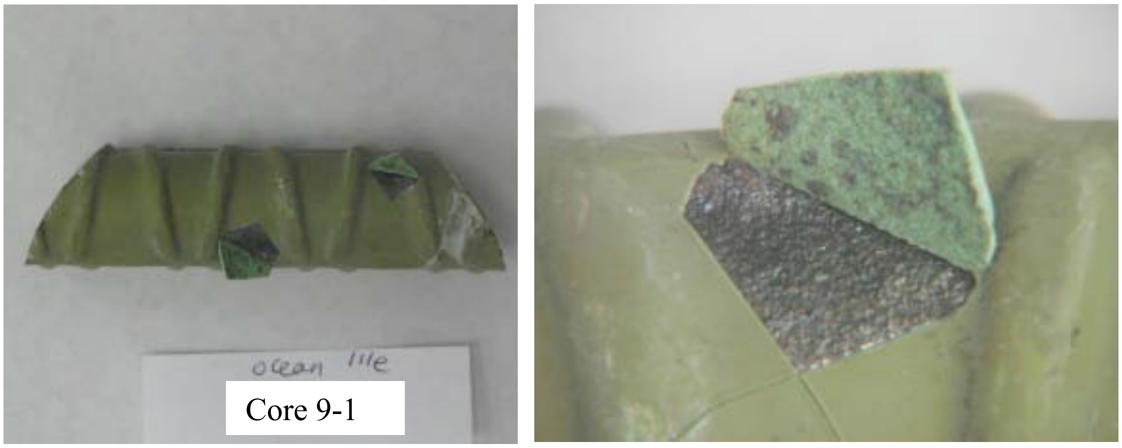
Figure 19. A core extracted from a cracked zone on Bent 9 in the Ocean Isle Bridge: a) a crack in concrete; b) the core showing side view of crack; c) external surface condition of the core; d) the epoxy-coated bar segment contained in the core was corrosion-free. Bar-level chloride concentration is 0.009 wt%.

Figure 20. Several columns in the Ocean Isle Bridge developed spalls (far above high tide) due to inadequate cover over uncoated bar spirals.



(a) *(b)*

Figure 21. Calculated surface chloride C_s and effective diffusion coefficient D for the Ocean Isle Bridge.



(a) (b)

Figure 22. Typical condition of corrosion-free epoxy-coated bar extracted from the Ocean Isle Bridge: a) overall appearance; b) undercoating steel often has a dark color but is corrosion-free. The bar is from Core 9-1, 10 in. below high tide. Bar-level chloride concentration is 0.056 wt%.

(a) (b)

Figure 23. Two bars contained in Core 9-2 from the Ocean Isle Bridge show corrosion staining: a) a horizontal No. 6 bar with a cover of 4.75 in. Bar-level chloride concentration is 0.199 wt%; b) a vertical No. 8 bar with a cover of 5.1 in. and bar-level chloride concentration is 0.183 wt%.

(a)

(b)

(c)

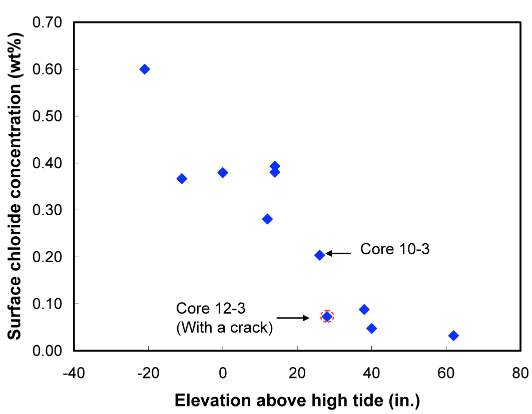
Figure 24. Uncoated bars from the Ocean Isle Bridge: a) a No. 3 round bar from Core 10-1, bar cover is 4.3 in. and bar-level chloride concentration is 0.021 wt%; b) a No. 4 deformed bar from Core 10-3, cover is 4.125 in. and bar-level chloride concentration is 0.009 wt%; c) a No. 4 deformed bar from Core 10-3, cover is 4.5 in. and bar-level chloride concentration is 0.007 wt%.

Figure 25. A view of the Holden Beach Bridge, Holden Beach, North Carolina.

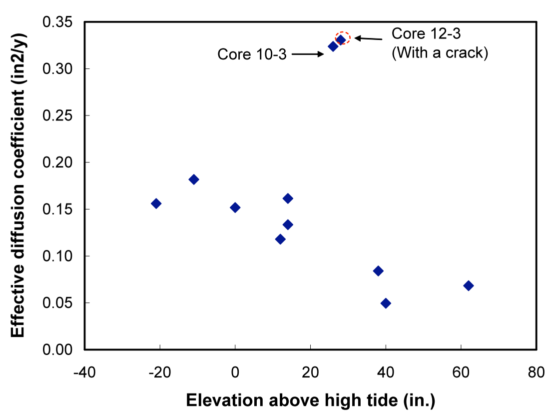
Figure 26. Surface condition of the Holden Beach Bridge Bents: the south face of Bent 11.

Figure 27. Surface condition of the Holden Beach Bridge Bents: the west face of Bent 12. Cracks are highlighted with yellow crayon.

Figure 28. A spalled area observed on the east column of Bent 11of the Holden Beach Bridge. The spall is about 10 ft. above high tide, and concrete cover for the uncoated bar is approximately 1 in.

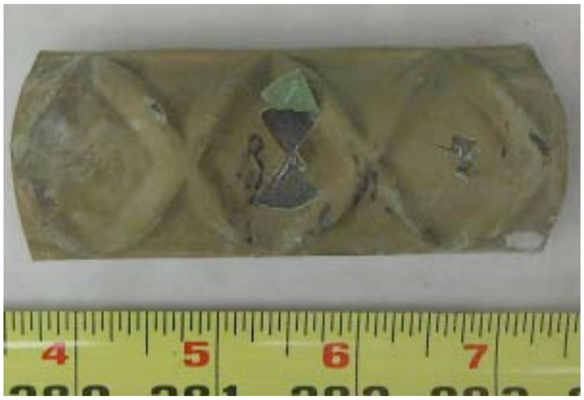


(a)



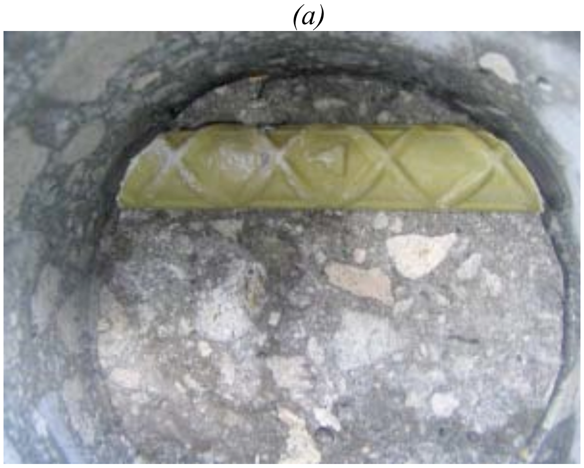
(b)

Figure 29. Calculated surface chloride C_s and effective diffusion coefficient D for the Holden Beach Bridge.

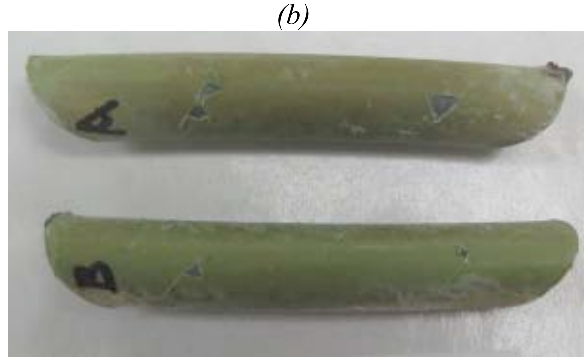


(a)

(b)



(c)



(d)

Figure 30. Condition of bars extracted from the Holden Beach Bridge: a) a bar from Core 10-2, from the top of the strut wall of Bent 10 (14 in. above high tide). Bar-level chloride is 0.039 wt%; b) a bar from Core 10-3, from the west column of Bent 10 (26 in. above high tide). Bar-level chloride concentration is 0.100 wt%; c) a bar from Core 12-5, from the north face of Bent 12 (at high tide line). Bar-level chloride concentration is 0.068 wt%; d) two bars from Core 10-5, from the west column of Bent 10 (62 in. above high tide). Bar-level chloride concentration is 0.008 wt%.

Figure 31. A view of the Atlantic Beach Bridge, Atlantic Beach, North Carolina.

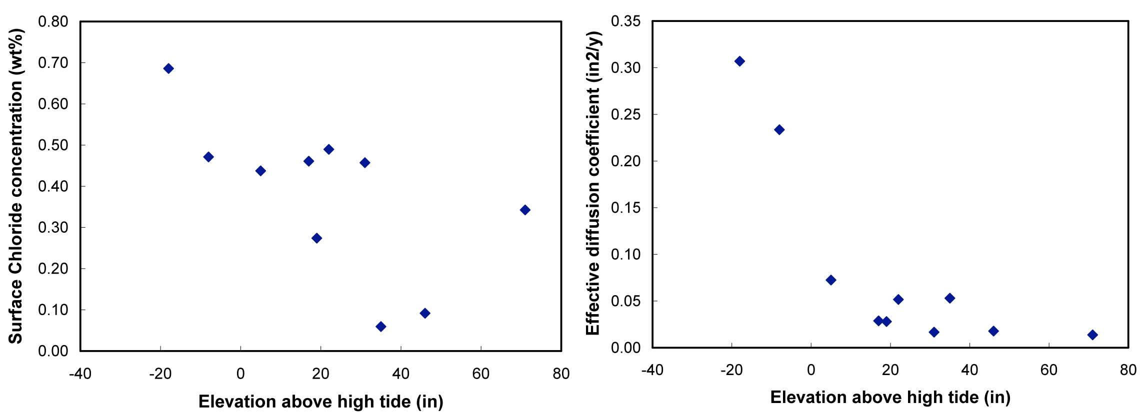
(a) (b) (c) (d)

Figure 32. Atlantic Beach Bridge substructure configuration: a) prestressed piles (Bent 1-14 and 44-48); b) pile caps with thin strut (Bents 15-25 and 34-43); c) pile caps with shafts (Bents 26-28 and 31-33); d) pile cap with a haunch (Channel Bents 29 and 30).

Figure 33. Surface condition of the Atlantic Beach Bridge Bents: the pile cap of Bent 38 has two horizontal cracks (highlighted by arrows).

Figure 34. Surface condition of the Atlantic Beach Bridge Bents: a delamination observed on the top of the pile cap of Bent 36. The delaminated concrete has a thickness of approximately 1 in.

Figure 35. Surface condition of the Atlantic Beach Bridge Bents: a delaminated area and several cracks were found on the top surface of the east pile cap of the Bent 34. The delaminated area has very thin cover (approximately 1/2 in.) and appears to be a secondary concrete placement (overlay).



(a) *(b)*

Figure 36. Calculated surface chloride C_s and effective diffusion coefficient D for the Atlantic Beach Bridge.

Figure 37. Condition of epoxy-coated bar extracted from the Atlantic Beach Bridge: 1) Core 34-4; 2) Core 31-8, bar imprint with no sign of active corrosion, uncoated steel ties were used.

(a) *(b)*

Figure 38. Surface condition and adhesion test results of bars extracted from the Atlantic Beach Bridge: a) a bar from Core 31-4, poor adhesion; b) a bar from Core 31-3, good adhesion. Bar-level chloride concentrations were both 0.003 wt%.

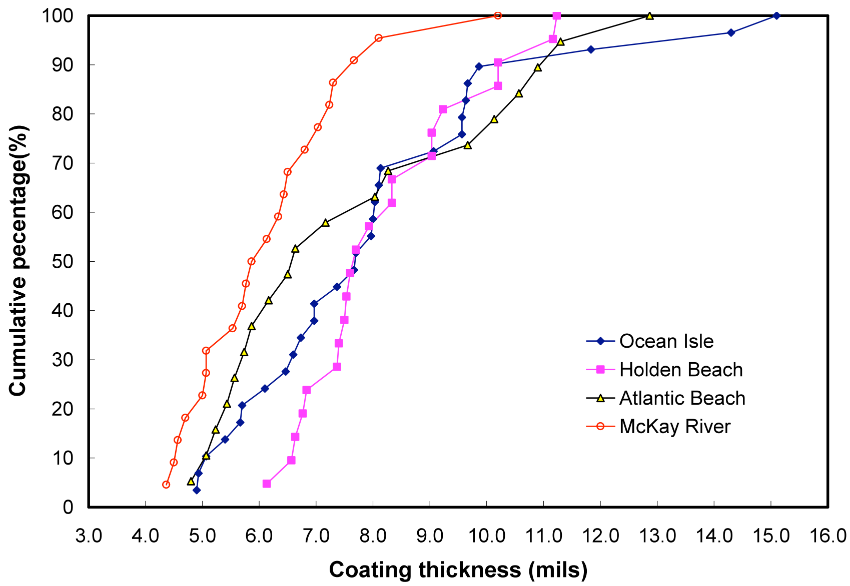


Figure 39. Measured coating thickness measurements of all the extracted epoxy-coated bars.

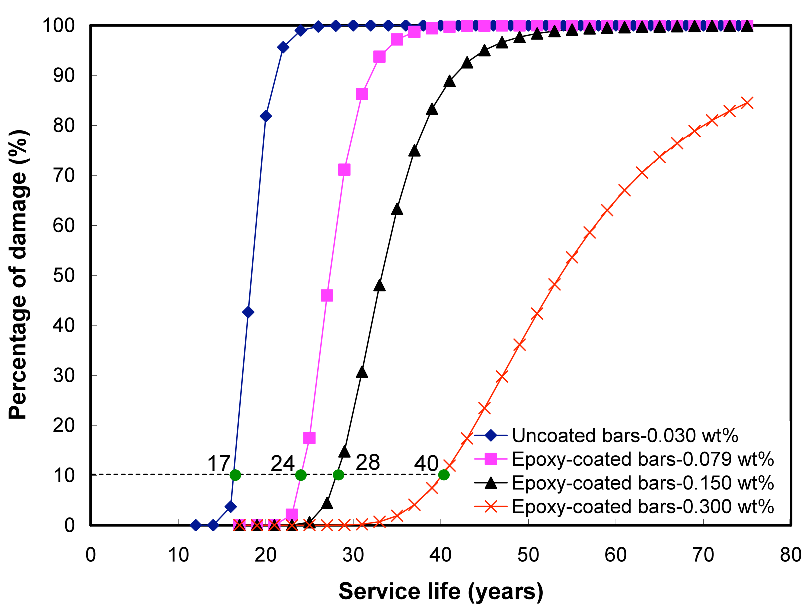


Figure 40. Model outputs for tidal zone: damage function for the MacKay River Bridge columns without cracks. Threshold based on field specimens: 0.079 wt%; thresholds included for comparison: 0.150 wt%, 0.300 wt%.

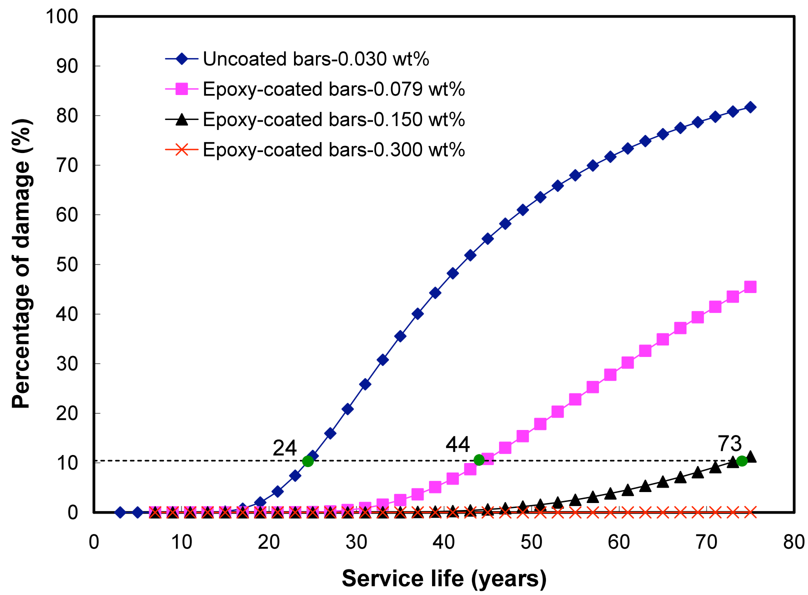


Figure 41. Model outputs for splash zone: damage function for the MacKay River Bridge columns without cracks. Threshold based on field specimens: 0.079 wt%; thresholds included for comparison: 0.150 wt% , 0.300 wt%.

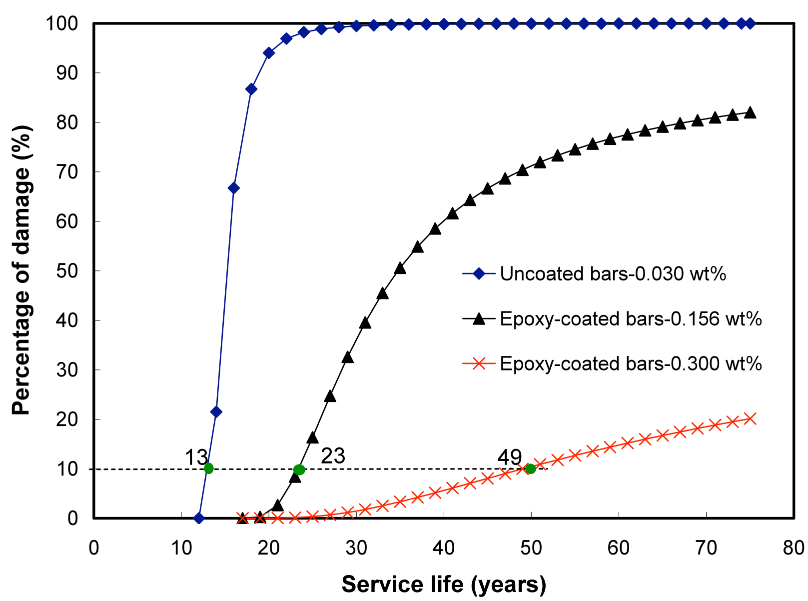


Figure 42. Model outputs for tidal zone: damage function for the Ocean Isle Bridge columns without cracks. Threshold based on field specimens: 0.156 wt%; threshold included for comparison: 0.300 wt%.

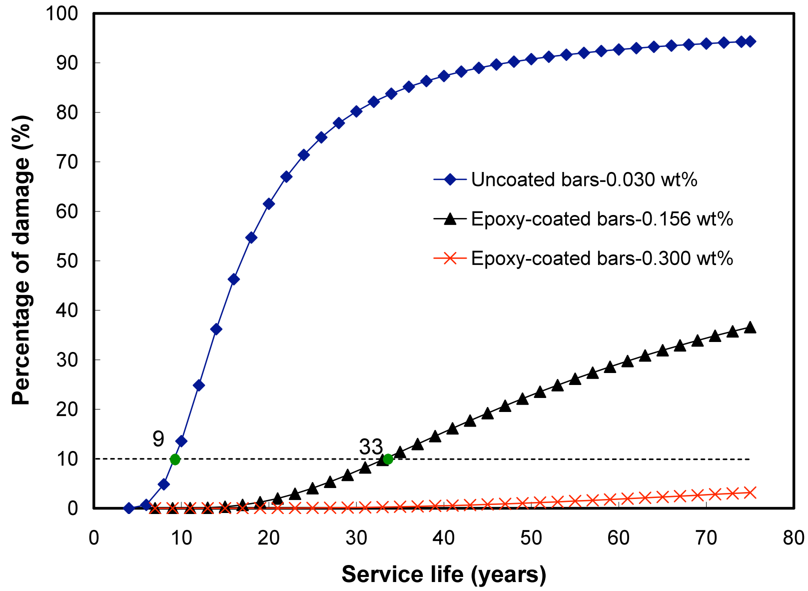


Figure 43. Model outputs for splash zone: damage function for the Ocean Isle Bridge columns without cracks. Threshold based on field specimens: 0.156 wt%; and threshold included for comparison: 0.300 wt%.

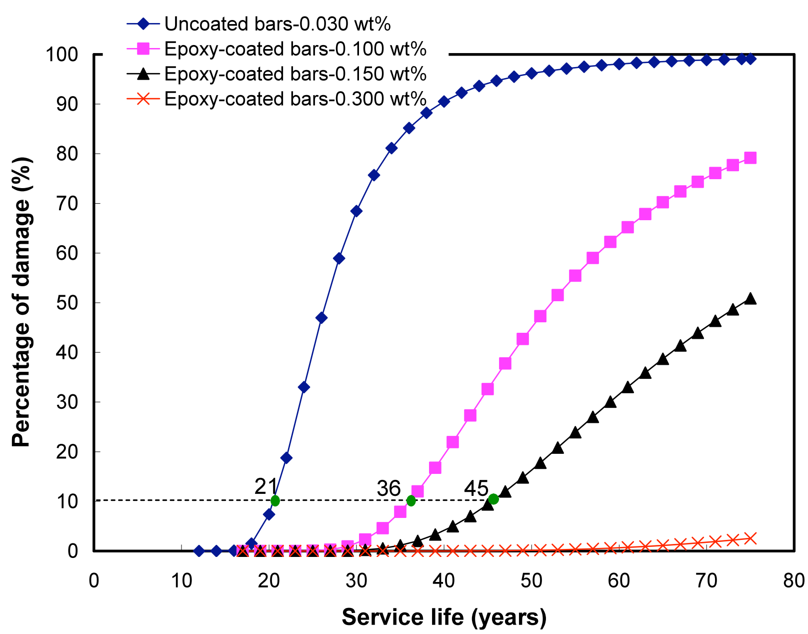


Figure 44. Model outputs for tidal zone: damage function for the Holden Beach Bridge pile caps without cracks. Threshold based on field specimens: 0.100 wt%; thresholds included for comparison: 0.150 wt% , 0.300 wt%.

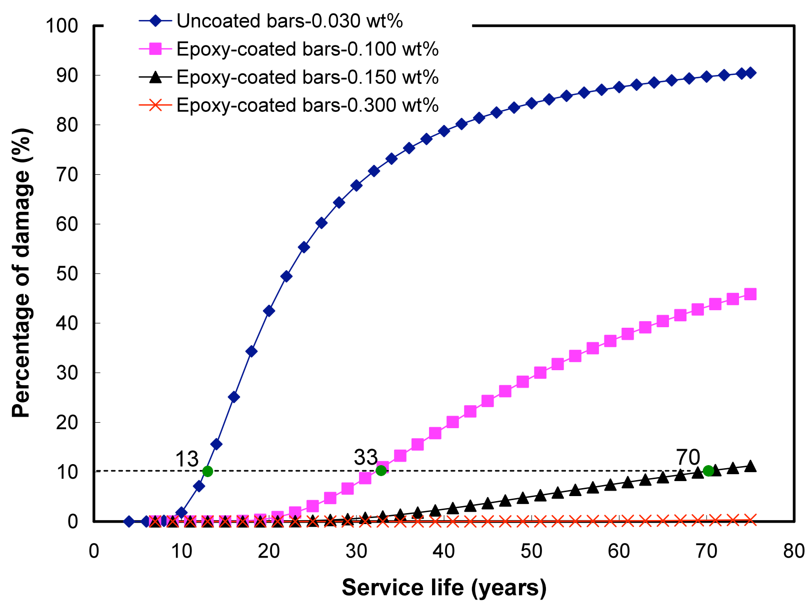


Figure 45. Model outputs for splash zone: damage function for the Holden Beach Bridge columns without cracks. Threshold based on field specimens: 0.100 wt%; thresholds included for comparison: 0.150 wt%, 0.300 wt%.

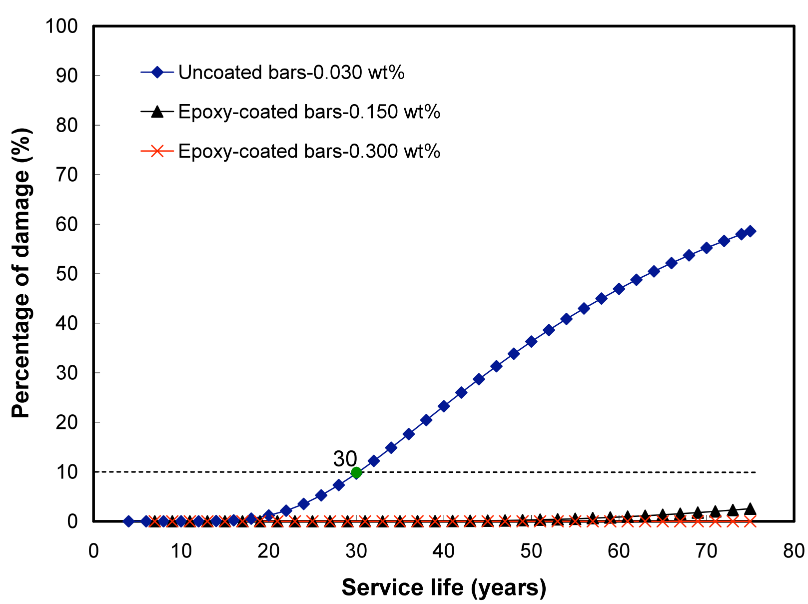


Figure 46. Model outputs for splash zone: damage function for the Atlantic Beach Bridge columns without cracks. Threshold based on field specimens of the Ocean Isle Bridge: 0.150 wt%; threshold included for comparison: 0.300 wt%.

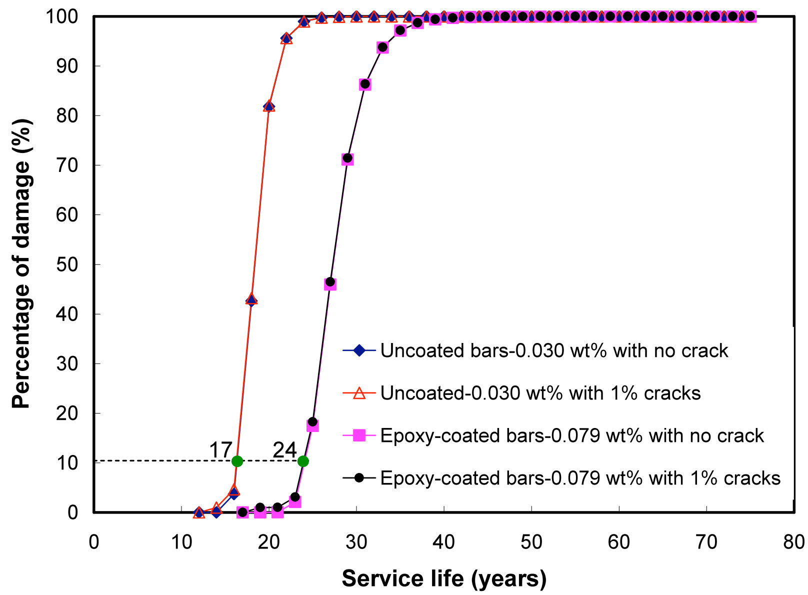


Figure 47. Model outputs for tidal zone: damage function for the MacKay River Bridge columns with 1% area affected by cracking. Threshold based on field specimens: 0.079 wt%.

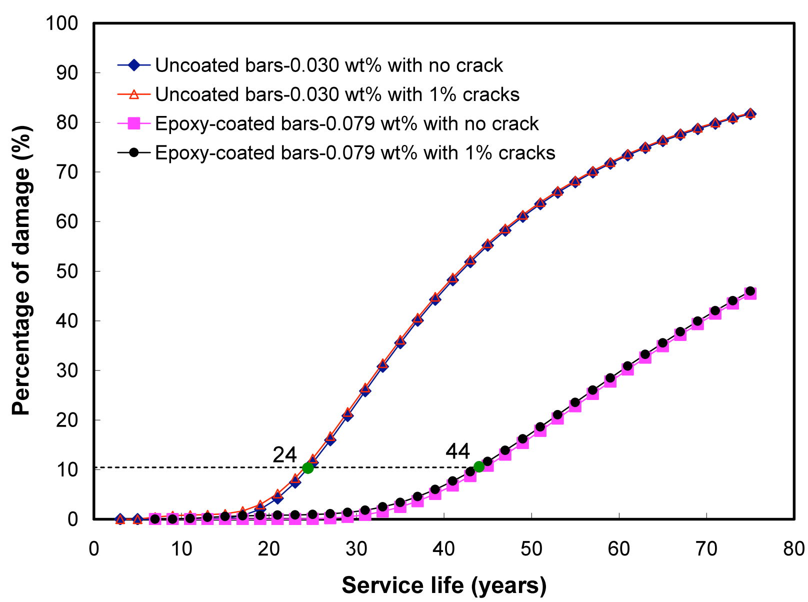


Figure 48. Model outputs for splash zone: damage function for the MacKay River Bridge columns with 1% area affected by cracking. Threshold based on field specimens: 0.079 wt%.

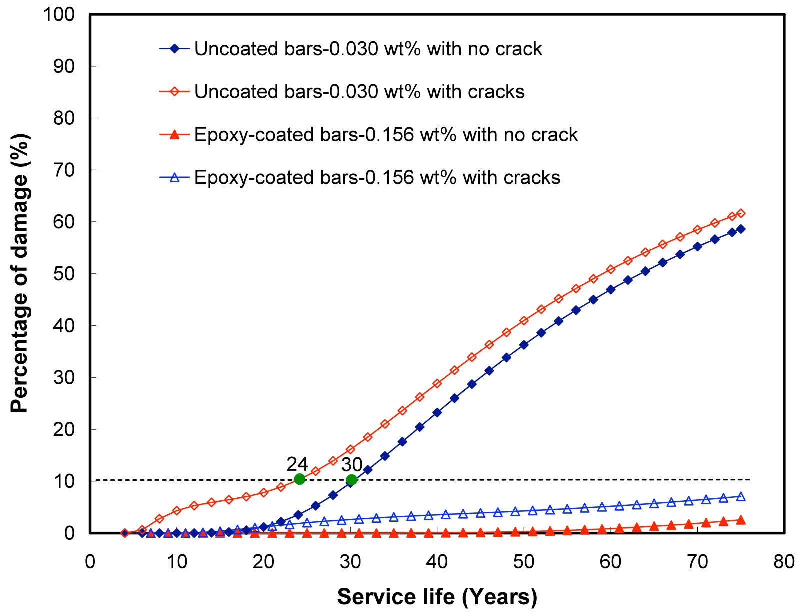


Figure 49. Model outputs for splash zone: damage function for the Atlantic Beach Bridge columns with 8% area affected by cracking. Threshold based on field specimens of the Ocean Isle Bridge: 0.150 wt%.

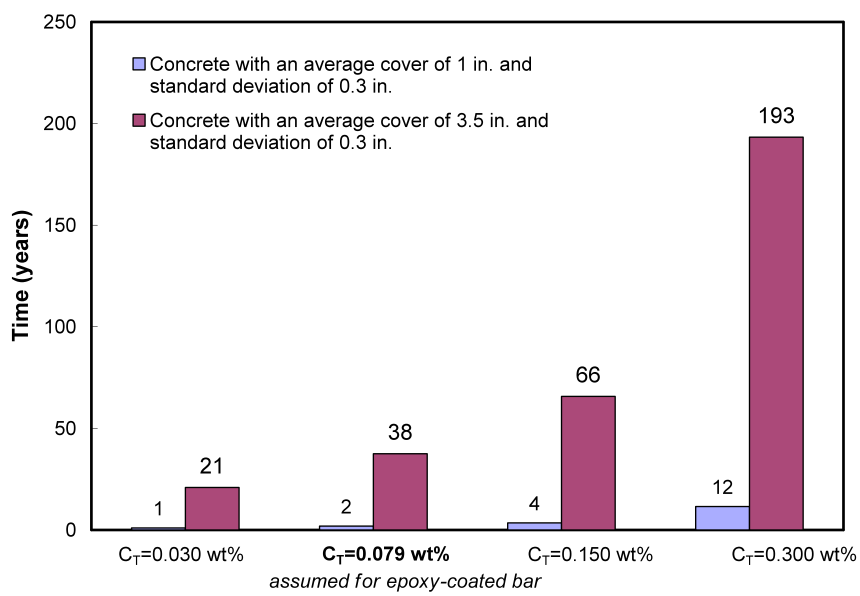


Figure 50. The effect of concrete cover on time to 10% reinforcement corrosion (Based on diffusion and surface chloride data from the splash zone of the MacKay River Bridge.)