



RESEARCH & DEVELOPMENT

Evaluating Corrosive Sites Policy for Concrete Bridges At the North Carolina Coast

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16. Abstract <p>This study was undertaken to determine whether the specified cover requirements, fly ash and silica fume addition rates, and corrosion inhibitor dosage rates described in the North Carolina Department of Transportation's Structures Design Manual are consistently and sufficiently achieving their objective of delaying the onset of corrosion in coastal concrete bridges. Additionally, this study was tasked with evaluating the current method of delineating the corrosive and highly corrosive zones. In order to evaluate the effectiveness of the current policy for corrosive sites, the research team used literature review, field evaluation, laboratory testing, and life-cycle modeling methods.</p> <p>The most significant active corrosion was detected in the tidal zone of bridge piers, although the corrosion rate was significantly reduced at locations outside of the tidal zone. Limited evidence of corrosion was detected or observed on the portions of the bridges that receive only atmospheric contact with chlorides through spray, splashing, and mist. On almost all elements tested, chloride concentration tests indicated a rapid decrease of chloride contamination between the surface and two inches below the surface. The significant contamination of chloride was limited mostly to the portions of the structure that are frequently exposed to brackish waters. Most concrete was found to have diffusion coefficients lower than the typical/target values proposed for 100-year service life by NCDOT. However, most structures located very close to the ocean were found to have more severe exposure than would be anticipated by those models. In most cases the calcium nitrite corrosion inhibiting admixture concentrations measured from powder samples collected in the field met or exceeded the specified NCDOT minimum addition rate of 3 gal/yd³. This finding confirmed that calcium nitrite is being utilized in the mixtures in approximately the correct dosage rates. However, the dosage rate did not have the expected effect of raising the corrosion initiation threshold of chloride concentration to above 9 lb/yd³.</p> <p>The findings of this research project suggest that the current corrosion policy is effective delaying the onset of corrosion in concrete components in most cases. It is effective at providing protection to concrete elements that are receiving chloride loading from splashing, spray, or atmospheric deposition. It is also effective at providing protection to bridges in locations where the concentration of chlorides in the brackish water is low. However, many bridges constructed under the current corrosion policy are not on track to have total maintenance-free service lives in excess of 75 or 100 years. Bridges that meet the current specified requirements of the corrosion policy to include silica fume, proper concrete cover, and corrosion inhibiting admixture, may have maintenance-free service lives of less than 50 years if they are located very close to the ocean.</p>					
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EXECUTIVE SUMMARY

Concrete structural elements of bridges constructed within highly corrosive areas of North Carolina (NC) are required to contain corrosion inhibiting admixtures and pozzolans in order to extend their service life in aggressive environments. The current North Carolina Department of Transportation (NCDOT) provisions provide prescriptive specifications for concrete mixtures used in certain elements of bridges built in corrosive regions. The intended outcome is that the use of pozzolans should slow the ingress of chloride by reducing the permeability of the concrete, and corrosion inhibitors should delay the onset of corrosion once chloride reaches corrosion initiation levels. The efficacy of the existing policy and materials specifications for concrete components designed for use in corrosive service locations has not been systematically verified. Since the existing policy went into effect in 2000, more than 200 bridges have been constructed. These structures have been in service for a number of years, and are currently available to observe and test. Their performance to date and remaining service life can be used to evaluate the impact of the corrosive sites policy and could also inform potential updates to the policy.

This study was undertaken to determine whether the specified cover requirements, fly ash and silica fume addition rates, and corrosion inhibitor dosage rates are consistently and sufficiently achieving their objective of delaying the onset of corrosion. Additionally, this study was tasked with evaluating the current method of delineating the corrosive and highly corrosive zones. In order to evaluate the effectiveness of the current policy for corrosive sites, the UNC Charlotte research team used literature review, field evaluation, laboratory testing, and life-cycle modeling methods. The scope of the research included the following five objectives:

1. Summarize the state of knowledge of NC bridge performance by reviewing previous NCDOT research projects related to corrosion and corrosion protection
2. Determine the current state of policies and specifications used by NCDOT and other state departments of transportation to prevent corrosion in concrete components
3. Evaluate the condition of bridges constructed within corrosive zones of North Carolina and link current condition to corrosive site conditions
4. Estimate typical service life of bridges constructed with current corrosion protection methods
5. Provide recommendations to retain or revise provisions of the existing corrosive sites policies

The team made the following primary observations during the study:

- Field measurements detected ongoing corrosion in all the structures that were sampled. The most significant active corrosion was detected in the tidal zone of bridge piers, although the corrosion rate was significantly reduced at locations outside of the tidal zone. Limited evidence of corrosion was detected or observed on the portions of the bridges that receive only atmospheric contact with chlorides through spray, splashing, and mist.
- Although most bridges did not exhibit significant visual signs of corrosion, at some locations the team identified exposed prestressing strand, concrete spalling and exposed steel bar of the bent cap, examples of consolidation problems, and efflorescence.
- On almost all elements tested, chloride concentration tests indicated a rapid decrease of chloride contamination between the surface and two inches below the surface. The significant contamination of chloride was limited mostly to the portions of the structure that are frequently exposed to brackish waters.
- Most concrete was found to have diffusion coefficients lower than the typical/target values proposed for 100-year service life by NCDOT (Rochelle 2000). However, most structures located very close to the ocean were found to have more severe exposure than would be anticipated by those models.
- In most cases the calcium nitrite corrosion inhibiting admixture concentrations measured from powder samples collected in the field met or exceeded the specified NCDOT minimum addition rate of 3 gal/yd³. This finding confirmed that calcium nitrite is being utilized in the mixtures in approximately the correct dosage rates. However, the dosage rate did not have the expected effect of raising the corrosion initiation threshold of chloride concentration to above 9 lb/yd³.
- Three of the bridges studied had expected service lives greater than 100 years, one just over 50 years, and three less than 50 years. The three bridges with expected service lives of less than 50 years were all characterized by high chloride exposure and loading (ie. less than 3 miles from the ocean) and problems with concrete quality that increased the diffusion coefficient.
- Evaluation of bridge deterioration rates (tracked as condition ratings in the NBI database) and the frequency of various flaws listed in NCDOT bridge inspection reports did not indicate that bridges constructed under the current policy are less susceptible to typical corrosion damage than bridges constructed under previous policies.

- The NC corrosion policy has similar features to many other states' policies but is more detailed than most in its specified requirements. Some states use site-based chloride sampling of coastal waters to determine the severity of the corrosive environment (rather than ocean proximity zones) and to set design guidelines based on severity. Several states allow the use of more corrosion resistant materials such as galvanized and stainless steel reinforcing.

The findings of this research project suggest that the current corrosion policy is effective delaying the onset of corrosion in concrete components in most cases. It is effective at providing protection to concrete elements that are receiving chloride loading from splashing, spray, or atmospheric deposition. It is also effective at providing protection to bridges in locations where the concentration of chlorides in the brackish water is low. However, many bridges constructed under the current corrosion policy are not on track to have total maintenance-free service lives in excess of 75 or 100 years. Bridges that meet the current specified requirements of the corrosion policy to include silica fume, proper concrete cover, and corrosion inhibiting admixture, may have maintenance-free service lives of less than 50 years if they are sited very close to the ocean.

As a result of the findings, the team proposes the following updates to the corrosion mitigation strategy:

- Create specifications for structures with extreme proximity to the coast that are likely to face very high chloride loading and fouling. These structures require either higher performance requirements for concrete mixtures or corrosion resistant steel.
- Enhanced provisions for elements exposed to wetting and drying are recommended to ensure the service life goals are met. These provisions could include increased concrete cover, protective jacketing, corrosion resistant steel, or other approaches.
- Continue to require and encourage use of pozzolans through binary and ternary cementitious material blends, but develop performance standards for these mixtures when used in tidal zones.
- Reduce reliance on corrosion inhibitors until their long-term efficacy can be verified.
- Use water samples to verify chloride exposure and to determine the aggressiveness of individual bridge sites. Revisit the boundaries of the highly corrosive and corrosive zones based upon the results of chloride content testing of the coastal waters.
- Add surface resistivity performance requirements to concrete mixture qualification processes. Add surface resistivity testing to the quality control protocol for concrete acceptance similar to those described in (Cavalline et al. 2020).

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LIST OF ABBREVIATIONS

AAR	alkali-aggregate reactivity
AASHTO	American Association of State Highway and Transportation Officials
AC	alternating current
ACI	American Concrete Institute
ASR	alkali-silica reactivity
ASTM	American Society for Testing and Materials
BME	Bridge Management Elements
BMS	Bridge Management System
DOH	Division of Highways
DOT	Department of Transportation
FHWA	Federal Highway Administration
ft	foot
kg	kilogram
k Ω	kilo-ohm
ID	identification
LaDOTD	Louisiana Department of Transportation and Development
lb	pound
LCA	lifecycle assessment
LCCA	lifecycle cost analysis
LTPP	Long Term Pavement Performance
MOE	modulus of elasticity
MOR	modulus of rupture
NBE	National Bridge Elements
NC	North Carolina
NCC	National Concrete Consortium
NCDOT	North Carolina Department of Transportation
Ω	ohm
OPC	ordinary portland cement
PEM	performance engineered (concrete) mixtures
PCC	portland cement concrete
pcf	pounds per cubic foot
pcy	pounds per cubic yard
PLC	portland limestone cement
PMS	pavement management system
psi	pounds per square inch
PLC	portland limestone cement
QA	quality assurance
QC	quality control
RCPT	rapid chloride permeability test
RP	Research Project
SCM	supplementary cementitious material
SHA	state highway agency
SG	specific gravity
SHRP	Strategic Highway Research Program
TRB	Transportation Research Board
US	United States
VDOT	Virginia Department of Transportation
w/cm	water to cementitious materials ratio

1. INTRODUCTION AND RESEARCH OBJECTIVES

Concrete structural elements of bridges constructed within highly corrosive areas of North Carolina are required to contain corrosion inhibiting admixtures and pozzolans in order to extend their service life in aggressive environments. The dosage rates prescribed in the specification are fixed, and were established following internal NCDOT research conducted by the Materials and Tests Unit. Bridges constructed since the specification was established have now experienced sufficient exposure to chloride rich environments that an assessment of their performance can serve as an evaluation of the effectiveness of the policy. Findings of this evaluation will enable NCDOT to confidently continue using the current specification, or will inform modifications to the specification in response to measured performance.

The current provisions provide prescriptive specifications for concrete mixtures used in certain elements of bridges built in corrosive regions. The intended outcome is that the use of pozzolans should slow the ingress of chloride by reducing the permeability of the concrete, and corrosion inhibitors should delay the onset of corrosion once chloride reaches corrosion initiation levels. This study was undertaken to determine whether the specified fly ash and slag addition rates and corrosion inhibitor dosage rates are consistently and sufficiently achieving their objective of delaying the onset of corrosion. The corrosive sites policy also links the construction location with required corrosion mitigation specifications. It must also verify that these requirements are being properly applied to all bridges that are substantially affected by corrosion.

The field study component of this project focused on a broad, representative sample of the more than 200 bridges that have been constructed during the fifteen years since the corrosive sites policy came into effect. The sample primarily included typical bridges (those not included in the High Value Bridge list) that utilized regularly implemented design considerations, materials, construction techniques, and QA and QC provisions. The research methodology employed field evaluation of bridges along with life-cycle modeling to assess whether the bridges are on track to achieve their service life expectations.

Research Need

The efficacy of the existing policy and materials specifications for concrete components designed for use in corrosive service locations has not been systematically verified. Benefits of the current policy, which is applied to two areas primarily delineated using selected roadways as boundaries, are that it is easy to apply with only the knowledge of the planned location of the bridge. Protective enhancements, such as addition of pozzolans and corrosion inhibiting admixtures, are prescriptive, and are therefore also easy to implement. However, whether these provisions are sufficiently reducing maintenance and extending the potential service life of bridges should be confirmed if the policy is to be continued. More than 200 bridges have been constructed under the provisions of the existing policy and are currently available to observe and test. Their performance to date and remaining service life can be used to evaluate the impact of the corrosive sites policy and could also inform potential updates to the policy.

Research Objectives

In order to evaluate the effectiveness of the current policy for corrosive sites, the UNC Charlotte research team used literature review, field evaluation, and life-cycle modeling methods. The scope of the research was encompassed by the following five objectives:

1. Summarize the state of knowledge of NC bridge performance by reviewing previous NCDOT research projects related to corrosion and corrosion protection
2. Determine the current state of policies and specifications used by NCDOT and other state departments of transportation to prevent corrosion in concrete components
3. Evaluate the condition of bridges constructed within corrosive zones of North Carolina and link current condition to corrosive site conditions
4. Estimate typical service life of bridges constructed with current corrosion protection methods
5. Provide recommendations to retain or revise provisions of the existing corrosive sites policies

1.2 Corrosive Sites

To delay the deterioration of structural concrete in areas of increased chloride concentrations, the North Carolina Department of Transportation (NCDOT) Structures Management Unit (SMU) created a Design Manual that includes multiple requirements and specifications related to the prevention of corrosion related damage. The SMU Design Manual

divides the coast into two areas, each requiring additional treatments to deter and/or slow the ingress of chlorides (NCDOT 2018). Structures lying within these areas and meeting additional criteria are identified as corrosive sites. The lines that demarcate the corrosive areas are shown below in Figure 1.1. All structures falling east of the highly corrosive line (red) are to follow all corrosive sites requirements and apply them to all bridge elements. Bridges falling east of the corrosive line (blue) and west of the highly corrosive line (red) are to apply all corrosive sites requirements only to those bridge elements located within 15 feet of the mean high tide.

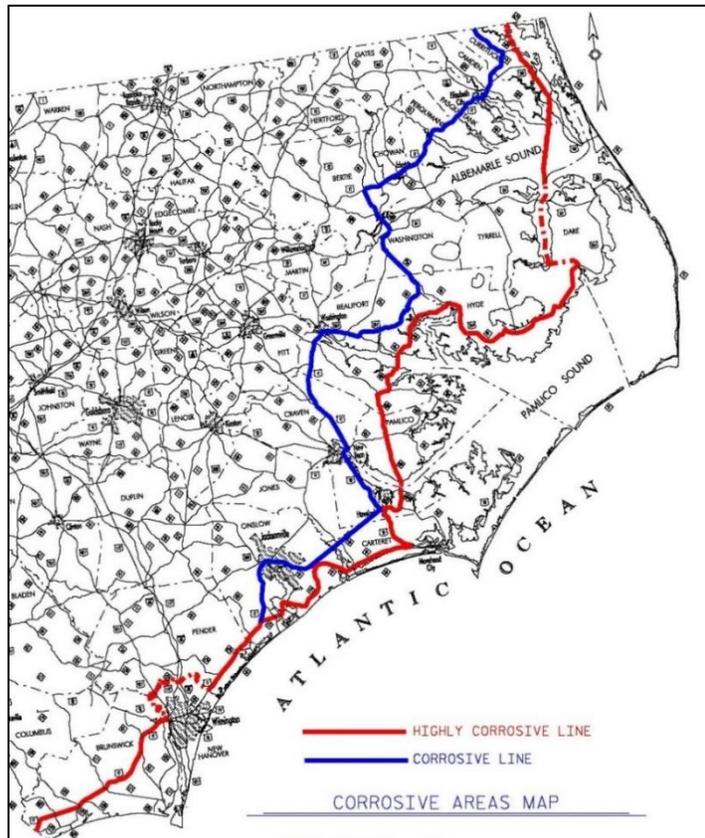


Figure 1.1. Map of corrosive sites dividing lines. (Originally from NCDOT SMU Design Manual, Figure 12-29 (NCDOT 2018))

1.3 NCDOT Corrosion Prevention Measures

The NCDOT SMU Design Manual provides guidelines for treating structures located within both of the corrosive site boundaries previously mentioned. A brief description of each corrosion prevention measure is presented below:

- Section 12-5 of the SMU Design Manual explains requirements related to when epoxy coated reinforcing steel is required. At corrosive sites, all cast-in-place (CIP) concrete elements should have epoxy coated reinforcing steel, bar supports, and incidental steel. Additionally, all precast and CIP culverts located east of the corrosive line should have epoxy coated reinforcing steel and bar supports.
- Section 12-12 (“Corrosion Protection”) of the SMU Design Manual describes the different measures that may be used as corrosion protection. At least one of the following measures is suggested for use:
 - Increased clear cover for reinforcing steel
 - Epoxy coated reinforcing steel
 - Addition of calcium nitrite corrosion inhibitor
 - Addition of silica fume
 - Addition of fly ash or granulated blast furnace slag
 - Specification of Class AA concrete in substructure elements
 - Limiting the use of uncoated structural steel

The selection of a measure and the degree of protection required varies depending on the location of the bridge. The factors that influence the selection of appropriate measures are whether the bridge is located east of the corrosive line or east of the highly corrosive line, and if the bridge falls within certain state divisions that experience significant levels of deicing agents. The SMU references the use of the flowchart shown in Figure 1.2 (Figure 12-30 in NCDOT SMU Design Manual) to determine the extent of corrosion protection required.

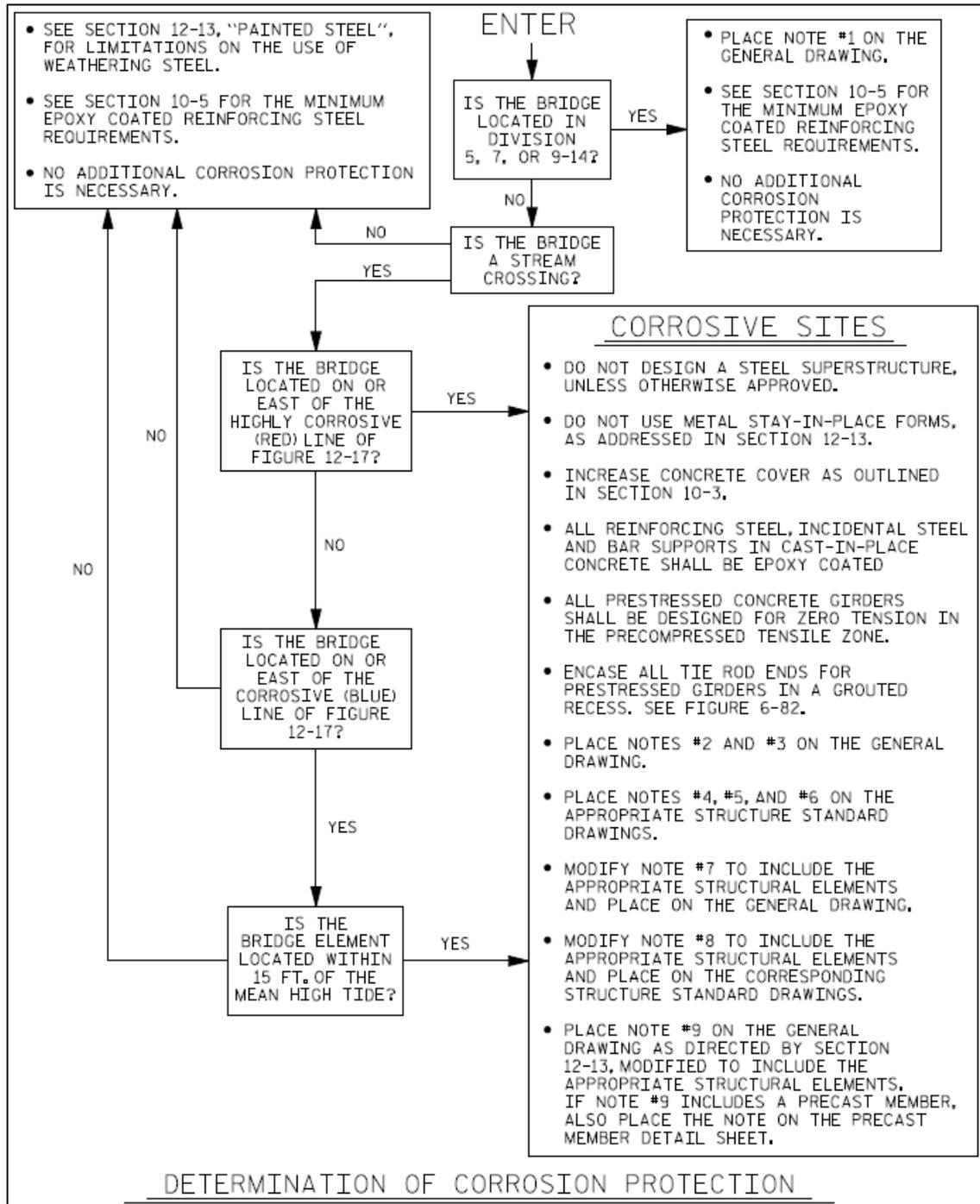


Figure 1.2. Flowchart to determine level of corrosion protection required. (Originally from NCDOT SMU Design Manual, Figure 12-30 (NCDOT 2018))

The research team used several strategies to study and evaluate the effectiveness of the corrosion policy. These included:

Review of other States' Policies

As a method of determining the state-of-the-art in terms of corrosion prevention policies, multiple states' DOT bridge design manuals were analyzed. For completeness, all states that have a coastal boundary were involved in this study. The states included were Alabama, California, Connecticut, Delaware, Florida, Georgia, Louisiana, Maine, Massachusetts, Mississippi, New Jersey, New York, North Carolina, Oregon, Rhode Island, South Carolina, Texas, Virginia, and Washington. The results of the review were summarized and compared to the policy used by NC. The review of other states' policies is summarized in Chapter 2, and more fully described in Appendix C.

Field Investigation

To obtain a clear picture of how these structures were performing, a representative subset of structures constructed under the current corrosion policy were selected and field inspections were performed to determine the current state of corrosion related deterioration. The structures selected were within the age range of 10 to 15 years old to ensure that they have had a significant amount of time to be exposed to conditions that cause weathering and potentially show early signs of corrosion. The investigation included corrosion rate and surface resistivity readings at multiple locations on each structure, along with a visual inspection and concrete specimen collection. The concrete specimens were evaluated in the lab at UNC Charlotte to determine the concentration of chlorides and concentration of corrosion inhibiting admixture that were present. The field investigation was intended to provide information to determine the current state of corrosion of typical structures near the North Carolina coastline. A full description of the methods and procedures used to collect data in the field are described in Chapter 3, and the results are presented in Appendix A.

Service Life Modeling

Modeling the effects of corrosion on bridge components selected for this analysis was performed using Life-365 software. Modeling provides an estimate of the service life a bridge will have before major repairs or reconstruction are necessary. This analysis is commonly known as a life cycle analysis (LCA). Life-365 allows a variety of corrosion mitigation techniques to be assessed on their impact to service life. The service life of a concrete structure is equal to the sum of the time it takes for corrosion to begin (initiation period) plus the time it takes for corrosion to reach an unacceptable level (propagation period) (Bentz and Thomas 2018). Data collected from the field investigation and laboratory testing were used as inputs to the modeling effort. Additional information on service life modeling and the findings of this effort are presented in Chapter 4.

Defect Mapping Investigation

An analysis of NCDOT maintenance records for structures located in the corrosive boundaries was performed to determine if structures constructed after the policy was enacted are performing better (i.e. longer maintenance-free service lives, delayed onset of corrosion, etc.) than structures constructed before the policy. The maintenance records were filtered to contain only defects that were potentially caused by corrosion of the embedded steel. In addition to determining whether structures are performing better following the enactment of the corrosion policy, this study was also intended to aid in determining whether the corrosive boundary lines are drawn in the correct location. The defect mapping procedure and results are described in Chapter 6 of this report.

Deterioration Modeling Investigation

Condition rating data sourced from the NBI database for structures located near the North Carolina coastline were used to create deterioration models. These models show the average time taken for a structure to deteriorate from condition rating 9 to condition rating 6 or 5 (not all structures reached condition rating 5 before receiving maintenance which increased the condition rating). Additionally, a statistical analysis of this same dataset was performed using Minitab to determine whether the differences shown on the deterioration models were statistically significant. This study was intended to determine whether structures constructed after the corrosion policy were performing better than those constructed before and to determine if structures constructed within the corrosive boundaries are performing equally to those constructed outside of the corrosive boundaries (i.e. west of the corrosive boundary line). This study also aided in determining whether the boundary lines are drawn in the correct location. Additional information and the findings of the deterioration modeling investigation are presented in Chapter 6?

This report summarizes the findings of a broad study. Greater detail is provided in three thesis documents that were generated by the Graduate Research Assistants that contributed to the project (Newsome 2020; Violette, 2020; Al-Salihi, 2022).

2. SUMMARY OF KEY LITERATURE FINDINGS

2.1 Overview of Corrosion

Corrosion in steel is an electrochemical process in which iron (Fe) is removed from the steel and dissolves into the surrounding pore solution. Once dissolved in the solution, the ferrous iron ions (Fe^+) are able to react with hydroxide ions (OH^-) and dissolved oxygen molecules (O_2) to form one of the many varieties of rust (ACI 2019). The presence of chlorides from winter maintenance materials, seawater, or other sources facilitates the onset of corrosion. This corrosion threshold of chloride ion concentration, also referred to as the critical chloride content, can vary greatly and is dependent on many factors such as the interface between steel and concrete, the chemistry of the pore solution, and the amount of oxygen that can make its way to the steel (Bertolini et al. 2013).

2.1.2 Mechanism

The process of corrosion occurs along the length of the reinforcing steel from an anodic site to a cathodic site. The process begins at the anode by the dissolution of iron into the pore solution, along with the loss of electrons (ACI 2019), as described by the following anodic reaction:



These electrons then flow through the reinforcement to the cathode, combining with the available water and oxygen which is present within the pores of the concrete. This combination leads to the creation of hydroxides (ACI 2019) as shown in the following cathodic reaction:



The creation of anodic and cathodic sites are the initial reactions in the formation of a rust by-product, with the actual formation of rust requiring several additional reactions. As there are many different ways to express the formation of rust, only one is detailed here. The following series of reactions describe how iron and hydroxide ions (OH^-) are formed at the cathode (Equation 2.2) and combine to create ferrous hydroxide. The ferrous hydroxide combines with available oxygen and water to create ferric hydroxide, which then dissolves into hydrated ferric oxide rust (Zhao and Jin 2016). These reactions are described in equations 2.3-2.5



2.1.3 Factors Affecting Corrosion Rate

The creation of anodic and cathodic sites leads to the accumulation of positive and negative charges. Hydroxide ions diffuse in the direction of the anode where they combine with the available ferrous ions. This combination, when the anodic and cathodic processes take the form of a corrosion cell with no additional electrons, causes electrical neutralization. If no source of electrons is present, then the oxidation rate at the anode must be equal to the reduction rate at the cathode. Therefore, the corrosion rate is reflected by the rate of electron flow (Zhao and Jin 2016). However, the rate at which corrosion occurs can also be influenced by many different factors such as concrete, steel, and environmental properties. One factor that can reduce the rate of corrosion is the availability of dissolved oxygen in the cathodic regions (Zhao and Jin 2016). As oxygen is consumed in the cathodic reaction as shown in Equation 2.2, the lack of a continuous supply of oxygen can significantly reduce the rate of corrosion. One way that oxygen is limited is by increased concrete cover.

Another factor that has a significant impact on the rate of corrosion is passivation. Passivation occurs when a thin, passive layer of insoluble metal oxide or hydroxide forms on the surface of the steel. This layer forms when the steel is exposed to an alkaline condition with a pH greater than 11.5 in an environment containing dissolved oxygen. Under these conditions, the steel can react with the oxygen to form the passive layer. With a passive layer surrounding the steel and given the same conditions required to create the passive layer, the rate of corrosion is effectively reduced to a negligible amount (ACI 2019). But even under continuous environmental exposure to alkaline conditions, the passive layer can be broken down by the ingress of chlorides.

2.2 Methods of Chloride Ingress

The four main mechanisms that allow the transport of chlorides into the concrete matrix include diffusion, capillary suction, permeation, and migration (Bertolini et al. 2013). While each mechanism can work alone, they often work simultaneously or in sequence with one another. Under different circumstances, one particular method is more likely to be the dominant transport mechanism. When the pores of the concrete are relatively dry, capillary suction is likely to be the driving mechanism, however, when the pores are relatively saturated, diffusion is likely to be the driving mechanism (Basheer et al. 2001).

2.3 Corrosion Mitigation Methods

Corrosion mitigation methods typically revolve around the protection of the reinforcing steel, as that is where the corrosive damage will begin. Selection of an appropriate mitigation method based on the susceptibility of individual structures to corrosion damage during their projected service life. This is handled differently by various coastal states. The methods covered in this section include minimum concrete cover requirements, reinforcing steel coatings, material selection, and common admixtures.

States with a coastal border have a variety of methods to identify which bridges are required to receive additional treatments or meet special specifications. States with the most extensive coastlines (California, Florida, etc.) typically have more specifically defined locations that require special treatments than states with less coastal exposure (Alabama, Mississippi, etc.). Some of the common ways to define a corrosive environment include a threshold content of chlorides measured in the water (California, Florida), the distance from the nearest coastline (Florida, North Carolina), marine water crossings (Louisiana, Maine), and defining specific counties as coastal counties (Georgia, South Carolina) (Caltrans 2003; FDOT 2019; GDOT 2018; LADOTD 2005; MEDOT 2003; NCDOT 2018; SCDOT 2006). Louisiana also includes areas with a history of corrosive damage (LADOTD 2005). Florida, California, and North Carolina all have different levels of corrosion prevention requirements based on parameters including distance from the nearest coastline or level of parts-per-million (ppm) chloride content (Caltrans 2003; FDOT 2019; NCDOT 2018).

Common mitigation methods are designated as either physical, passive, or active systems, and each can be used in conjunction with another. A physical method would be a matter of design materials and/or geometry, such as increased concrete cover and epoxy coating on the embedded steel or concrete surface. This acts as a way of increasing the time to corrosion initiation by providing a physical barrier between reinforcing steel and chlorides. A passive system would be the inclusion of admixtures, such as fly ash and other pozzolans. Passive methods work by decreasing the permeability of the concrete, therefore slowing the rate of chloride ingress. An active system would be the act of chemically raising the corrosion threshold of the steel. This is commonly achieved through the use of corrosion inhibitors (Rochelle 2000).

2.3.1 Minimum Concrete Cover

One of the simplest mitigation techniques used by many states is the specification of a minimum depth of concrete cover over the reinforcing steel, commonly defined for the top or bottom surface of the bridge deck. Some states also include a specification for exposed or submerged piers and bents. The most common minimum specification for top of bridge deck concrete cover is 2.5 inches, which is what North Carolina currently specifies (NCDOT 2018). Alabama, Louisiana, and Maine specify 2 inches for the minimum cover (ALDOT 2017; LADOTD 2005; MEDOT 2003). Florida and Georgia specify 2 inches if the bridge meets certain requirements, but typically specify a larger minimum value (FDOT 2019; GDOT 2018). While 2.5 inches is a common specification among many states, it appears that this is a moderately conservative value, as some states mentioned above specify a minimum value as low as 2 inches, and others, such as Delaware, specify a minimum value of up to 3 inches (DelDOT 2017).

2.3.2 Reinforcing Steel

The most common steel choices specified for high potential corrosion areas include epoxy coating, galvanized, and stainless steel. Based on our review of US coastal states' bridge design manuals, the most common type of reinforcing steel specified for corrosion resistance or mitigation is epoxy coated.

2.3.2.1 Epoxy Coated Steel.

Epoxy coated steel is one of the primary mitigation methods employed by North Carolina as well as many other states. This type of coating would be considered a physical mitigation method as it provides a physical barrier between the pore solution within the concrete and the reinforcing steel. The benefit of using epoxy coated steel is

that it has been used in both laboratory and field testing for decades, and has proven to be successful in delaying the onset of corrosion due to chlorides (ACI 2019).

A downside of using this method is that the coating can significantly lose its protective properties if it is damaged. This is a significant concern since damage can easily occur on a construction site. If damage occurs during construction, it is typically required to be repaired before moving forward. The concern is that it could be easy for damage to occur and go unnoticed or unreported.

At the time of this research, only 6 of the 19 states reviewed either do not specify or do not permit the use of epoxy coating. These states include Alabama, Florida, Mississippi, Oregon, South Carolina, and Virginia (ALDOT 2017; FDOT 2019; MDOT 2010; ODOT 2018; SCDOT 2006; VDOT 2011).

2.3.2.2 Galvanized Steel.

Galvanized, or zinc-coated, steel is another commonly specified protection for reinforcement found in environments designated as having a high corrosion potential. This type of coating is an example of a sacrificial protection. This means that the zinc coating undergoes corrosion and acts as the anode in the galvanic couple rather than the steel. An alternative protection method would be non-sacrificial, in which the coating protects the steel as long as it remains undamaged. Non-sacrificial coatings include copper and nickel, however, zinc-coated reinforcing bars are most commonly available (ACI 2019).

Similar to epoxy coated steel, galvanized steel has been used in structures for decades. However, unlike epoxy coated steel, the performance results of galvanized reinforcement under corrosive conditions have shown conflicting results. Some lab studies show an increase in the time required to crack (Cornet and Bresler 1996) while others show a reduction in the time required to crack (Griffin 1969). A field study of galvanized steel reinforcement that had been in service for many years did not show significant deficiencies. However, it was noted that the chloride concentrations at the depth of the reinforcing steel were fairly low so it could not be conclusively determined that the galvanized bars were working effectively (Cook and Radtke 1977). Additionally, marine and accelerated field studies showed that galvanized steel reinforcement was successful in delaying the onset of corrosion related damage, such as delaminations or spalling, but did not prevent them entirely (Arnold 1976; Sopler 1973). At the time of this research, very few coastal states specify or allow the use of galvanized steel, with Massachusetts, New York, and Texas being the exceptions (MassDOT 2013; NYDOT 2017; TXDOT 2018). Virginia explicitly states that galvanized steel should not be used (VDOT 2011).

2.3.2.3 Stainless Steel.

Stainless steel as reinforcement was first introduced in the 1930s, though it is not used often due to limited availability and high cost. It is currently being used more frequently as reinforcement for structures in environments with a higher corrosion potential due to its demonstrated ability to resist corrosion relative to conventional steel (ACI 2019). Stainless steel has also been used in conjunction with carbon steel in an effort to create a cost-effective design. This design methodology involves placing stainless steel in areas where corrosion is to be expected, such as the top layer of a bridge deck where water containing chlorides may sit for extended periods, and placing carbon steel where corrosion is less likely to occur (ACI 2019).

Currently, there are three states (New York, Oregon, and Texas) that use stainless steel as an option for reinforcement (NYDOT 2017; ODOT 2018; TXDOT 2018). New York allows the use of epoxy coated steel, stainless steel, or galvanized steel for bridge reinforcement (NYDOT 2017). Oregon specifically states that stainless steel should be used and epoxy coated steel is not permitted (ODOT 2018). Texas specifies the use of epoxy coated steel or galvanized steel, with stainless steel being required for areas of severe exposure (TXDOT 2018).

2.3.3 Admixtures

The most common admixtures specified in the different coastal states' bridge design manuals included calcium nitrite, fly ash, granulated blast furnace slag, and silica fume. These materials work in different ways and each fall under the category of either a corrosion inhibitor or a pozzolan.

2.3.3.1 Corrosion Inhibitors.

A corrosion inhibitor is a chemical admixture that reduces the rate of corrosion on the reinforcing steel without actually reducing the concentration of the corrosive species. Corrosion inhibitors effectively raise the corrosion threshold level, requiring that a higher level of chlorides be present on the surface of the reinforcing steel

in order for corrosion to initiate. Calcium nitrite is an example of a corrosion inhibitor and is currently used as a corrosion protection measure in Florida, Louisiana, Maine, New York, North Carolina, and Virginia (FDOT 2019; LADOTD 2005; MEDOT 2003; NCDOT 2018; NYDOT 2017; VDOT 2011).

2.3.3.2 Pozzolans.

A pozzolan is a supplementary cementitious material (SCM) that is used to refine and reduce the concrete porosity by filling in the smaller gaps found between aggregates. By filling these gaps with very fine minerals, concrete demonstrates significantly enhanced resistance to chloride penetration (ACI 2019). However, the use of SCMs can be counterproductive if too much is added. There are calcium hydroxides ($\text{Ca}(\text{OH})_2$) formed when portland cement hydrates which creates a buffer for the pore solution, helping to maintain a constant pH level. When too much of a mineral admixture is introduced, all of the calcium hydroxides will be used in the pozzolan reaction (ACI 2019). This eliminates its ability to act as a buffer and allows the pH to get to a lower level where the steel will no longer be passivated. Fly ash, granulated blast furnace slag, and silica fume are pozzolans that are currently being used as a corrosion protection measure in California, Florida, Louisiana, Maine, North Carolina, Oregon, and Rhode Island (Caltrans 2003; FDOT 2019; LADOTD 2005; MEDOT 2003; NCDOT 2018; ODOT 2018; RIDOT 2007).

2.4 Test Methods for Corrosion-Related Processes

A portion of this project included performing field investigations of structures located within the NCDOT defined corrosive zones. These field visits included the collection of corrosion rate data, surface resistivity data, and concrete powder samples from bridge elements.

2.4.1 Corrosion Measurement Device Selection

To obtain data about ongoing corrosion during field visits the iCOR by Giatec was used. The iCOR is a non-destructive testing (NDT) device (no direct contact with the reinforcing steel is required) that includes a user-friendly tablet-based interface that is wirelessly connected to the handheld transducer. Additionally, testing indicated that the iCOR has a smaller margin of error and reported more precise measurements than the GalvaPulse (Poe 2019).

2.4.2 CEPR Method of the Giatec iCOR

The iCOR makes use of Giatec Scientific's patented technology called Connectionless Electrical Pulse Response Analysis (CEPRA). CEPRA is what differentiates the iCOR from other corrosion measurement devices that were commercially available at the time of this study because it is not necessary to expose the reinforcing steel to take measurements. Using four electrodes, the electrical response of reinforcing steel in concrete can be determined. This is depicted in Figure 2.1. An AC current is applied to the surface of the concrete between the two outer electrodes while the voltage between the two inner electrodes is measured.

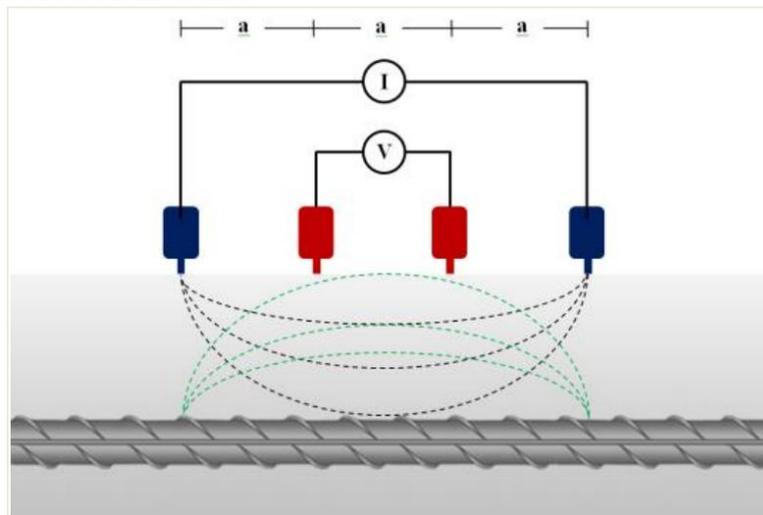


Figure 2.1. Layout of electrodes utilized by Giatec iCOR. (Originally Figure 3 from Giatec iCOR Manual (Giatec 2019))

During testing, the applied AC current is varied between a low frequency and a high frequency while the voltage at the inner electrodes is recorded. The use of varying frequencies is key due to the difference in response between a corroding bar and a non-corroding bar. The voltage response of a non-corroding bar increases when a lower frequency is applied. The voltage response remains nearly constant with varying frequencies for a bar that is corroding (Giatec 2019). This relationship is depicted in Figure 2.2.

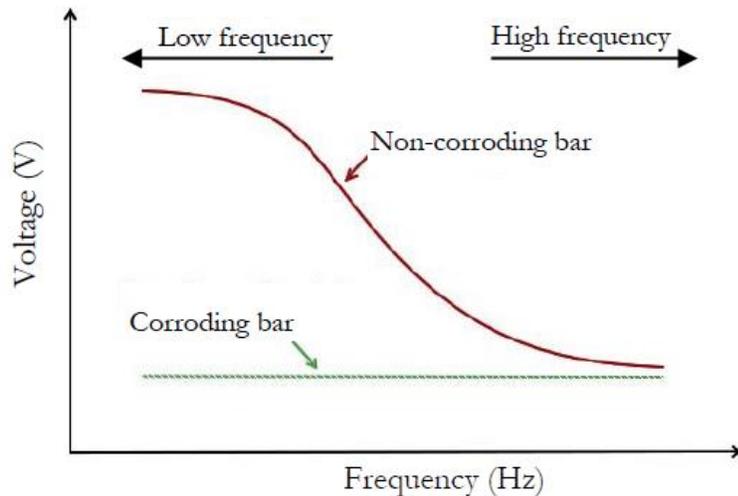


Figure 2.2. Graph demonstrating different voltage responses between a corroding and non-corroding bar (Originally Figure 4 from Giatec iCOR Manual (Giatec 2019)).

While the voltage response varies between a corroding and non-corroding bar under the application of various frequencies, direct measurement of this low-frequency response is time consuming and is vulnerable to noise corruption (Giatec 2019). This makes it impractical to apply this technique to measure the corrosion rate of steel embedded in concrete in a field setting. The Giatec iCOR avoids these issues by applying a narrow DC/AC current pulse over a short time period (6-10 seconds) while using a high sampling rate (3 samples per second) to record the voltage response (Giatec 2019).

The limitations of the device include its inability to test components with epoxy coated steel, galvanized steel, or prestressed post tensioned tendons that are placed in a protective tube. Additionally, the results can be affected by temperature, moisture, concrete cover thickness and material properties, and the availability of oxygen.

The interpretation of results is largely driven by the coefficient of determination, or R-squared value, which is the proportion of observed variation explained by the regression model (Devore 2010). The R-squared value ranges from 0.0 meaning no correlation to 1.0 meaning a perfect correlation. After each individual test is performed, the software plots a graph of change in voltage over the total test period and generates a best fit curve along with the corresponding R-squared value. At this stage, the data can either be saved if the R-squared value is acceptable or deleted so that the test may be performed again. The R-squared values that were accepted in the field are discussed in Chapter 3: Field Investigation.

2.4.3 Corrosion Rate

The corrosion rate is usually described as rate of steel loss and is typically expressed in $\mu\text{m}/\text{year}$ (micrometers per year). There are many factors that can influence the magnitude of the corrosion rate, such as temperature or humidity (Bertolini et al. 2013). Bertolini et al. (2013) suggest the following ranges for interpreting corrosion rate measurements:

- Negligible: Less than 2 $\mu\text{m}/\text{year}$
- Low: 2 – 5 $\mu\text{m}/\text{year}$
- Moderate: 5 – 10 $\mu\text{m}/\text{year}$
- Intermediate: 10 – 50 $\mu\text{m}/\text{year}$
- High: 50 – 100 $\mu\text{m}/\text{year}$
- Very high: Greater than 100 $\mu\text{m}/\text{year}$

Alternatively, the Giatec iCOR User Manual suggests the ranges in table 2.1 (Giatec 2019). These ranges were used in the field work in this study to create heat maps in the

Table 2.1 Classification of corrosion rate results (Giatec 2019).

Color Code	Corrosion Rate (µm/year)	Classification
Green	< 10	Passive/Low
Yellow	10 - 30	Moderate
Orange	30 - 100	High
Red	> 100	Severe

The iCOR is capable of calculating the corrosion rate by first determining the polarization resistance of rebar, or R_p , by making use of the following equations (Giatec 2019):

$$R_p = A_p \times R_{cA} \quad (2.6)$$

$$i_{corrosion} = \frac{B}{R_p} \quad (2.7)$$

In Equations 2.6 and 2.7, A_p is the polarized area of steel, R_{cA} is the charge transfer resistance of the steel (defined previously in Figure 2.3), $i_{corrosion}$ is the corrosion rate of the steel, and B is a constant parameter that is determined experimentally (Giatec 2019). It should be noted that the variables described above are all determined by iCOR's companion tablet and the equations are solved automatically after testing a particular grid point.

2.4.4 Surface Resistivity

Surface resistivity is a measure of the ability of an electrical current to flow within a material and can be used as a parameter to describe the ability of concrete to resist the ingress of chloride ions (Lataste 2010). Resistivity can be influenced by several factors such as, cement type, water-to-cement ratio (w/c), and the presence of chlorides. While some factors work to decrease the resistivity and therefore increase the risk of corrosion, other factors such as a low w/c ratio or the addition of a pozzolan work to increase the resistivity, therefore decreasing the risk of corrosion (Bertolini et al. 2013). It has been shown that the addition of a pozzolan, such as silica fume (a common pozzolan used by the NCDOT), can reduce the electrical conductivity of concrete by more than 90% when compared to a conventional mixture with portland cement (Ramezaniapour et al. 2011; Shi et al. 1998). Surface resistivity is inversely related to electrical conductivity, meaning that as the conductivity is reduced, the resistivity is increased.

The resistivity of concrete can range in value from less than ten to hundreds of kilohm-centimeters (tens to thousands of ohm-meters) with lower values indicating a higher risk of chloride ingress and higher values indicating a lower or negligible risk of chloride ingress (Bertolini et al. 2013). The following table (Table 2.2) presents global reference values for the surface resistivity of mature (age > 10 years) dense-aggregate concrete measured at 20°C (68°F) (Cox et al. 1997; Polder et al. 2000).

Table 2.2. Global reference values at 20°C for resistivity of mature (>10 years) concrete. (Cox et al. 1997; Polder et al. 2000)

Environment	Concrete resistivity (Ωm)	
	Ordinary portland cement	Blast furnace slag or fly ash cement including silica fume
Very wet, submerged, splash zone	50-200	300-1000
Outside, exposed	100-400	500-2000
Outside, sheltered, coated, hydrophobized (not carbonated)	200-500	1000-4000
Outside, sheltered, coated, hydrophobized (carbonated)	1000+	2000-6000+
Indoor climate (carbonated)	3000+	4000-10000+

There have been multiple recent projects completed at UNC Charlotte in which surface resistivity specifications were recommended for use by the NCDOT (Biggers 2019; Cavalline et al. 2020). However, recommendations from Biggers (2019) and Cavalline et al. (2020) are not directly applicable to the study of concretes in this project due to differences in testing conditions (structural elements in the field vs. laboratory cured specimens), cement type, concrete moisture content, chloride contamination, specimen shape, etc. The previous research at UNCC was also primarily related to early age surface resistivity while all structures investigated for this thesis were within 10 to 15 years old at the time of testing. For the purpose of the field work, the ranges in Table 2.3 were used to interpret the surface resistivity data. This table has been suggested for use by Feliu et al. (1996) and Polder et al. (2000).

Table 2.3. Classification of surface resistivity results (Feliu et al. 1996; Polder et al. 2000)

Color Code	Resistivity(kohm.cm)	Classification
Green	> 100	Very High
Yellow	50 - 100	High
Orange	10 - 50	Moderate
Red	< 10	Low

The Giatec iCOR is capable of determining the surface resistivity of concrete by making use of the following equation (Giatec 2019):

$$\rho = 2\pi a \times R \quad (2.8)$$

In Equation 2.8, ρ is the surface resistivity, a is a constant parameter based on the geometry of the electrodes, and R is the equivalent resistance of the concrete. R is calculated from R_{c2} , R_{c3} , and R_{c4} which were previously defined in Figure 2.3. The iCOR user manual notes that by using this approach the effect of rebar would be minimized whereas other AC measurement techniques will have inherent error (Giatec 2019). It should be noted that the variables described above are all determined by the iCOR's companion tablet and the equation is solved automatically after testing a particular grid point.

2.4.5 Variability Associated with Surface Resistivity

Field testing of surface resistivity, regardless of testing device, has inherent variance due to constantly changing conditions such as fluctuations in the weather and tides. Presuel-Moreno et al. (2010) performed a field study of more than 60 new and old bridges located in coastal environments in Florida. The study included the creation of surface resistivity profiles at varying elevations of partially submerged reinforced concrete substructures at and above the mean water elevation. The field testing of surface resistivity was performed using a commercial Wenner probe with electrode spacings of 3 centimeters (cm) and 5 cm. The profiles demonstrated a surface resistivity gradient from low to high starting within the submerged zone and extending upwards to an elevation outside of the splash zone, which Presuel-Moreno et al. (2010) attributed to the elevation dependence of moisture saturation (higher moisture content at lower elevations for example).

Concrete has an outer surface layer that can have a surface resistivity value different than that of the bulk material. This layer could be the result of chloride ingress, carbonation, or differential moisture content and can range in depths from a fraction of a millimeter to a few centimeters (Presuel-Moreno et al. 2010). It was found that the relative humidity at depths of 3 centimeters or greater is constant, which would result in a constant surface resistivity unless chlorides have penetrated deeper than 3 centimeters (McCarter et al. 2000; Pruckner and Gjorv 2001; Saleem et al. 1996).

In addition to the field study, Presuel-Moreno et al. (2010) collected nominal 2-inch cores in the same vicinity as the surface resistivity profiles were tested and performed surface resistivity measurements in a controlled laboratory setting. Prior to performing the laboratory measurements, the cores were placed in a high humidity chamber for a few weeks (exact timeframe not specified). These resistivity measurements were considered to be wet while the resistivity measurements made in the field were considered to be dry. The wet and dry measurements were correlated, and researchers observed that the dry (field tested) resistivity values were about 3 times higher than the wet (laboratory conditions) resistivity values (Presuel-Moreno et al. 2010).

2.5 Chloride Concentration Measurements in Concrete Samples

A common way to test hardened concrete for chloride content is the rapid chloride test (RCT). This test requires that a powder sample be taken by drilling into hardened concrete. It is then mixed with an acidic extraction liquid to liberate the chloride ions from the concrete matrix. The separated ions can then be measured with a calibrated electrode as a function of chloride percentage by mass of concrete (Germann Instruments 2020). Powder samples taken at various depths at the same location allow for a plot to be generated that depicts the chloride profile, or changes in concentration with depth. This test method provides equivalent results to other typical methods such as AASHTO T260, ASTM C114 and others published by international agencies (AASHTO 2021; ASTM 2018). A diffusion coefficient can be calculated using these measurements of chloride concentrations at various depths. This diffusion coefficient is a key variable to service-life prediction models, such as Life-365, and quantifies the rate at which chlorides can move along a concentration gradient.

2.6 Techniques to Verify the Presence of Corrosion Inhibitors

To limit the impact of corrosion on reinforced structures, the use of corrosion inhibiting admixtures has become common. Many types of corrosion inhibiting admixtures are commercially available, but the most common ones utilize calcium nitrite as the active ingredient (Jeknavorian et al. 1995). Calcium nitrite acts as a corrosion inhibitor by preventing the creation of expansive corrosion products while repairing oxidization damage on the steel surface (Jeknavorian et al. 1995). In North Carolina, these admixtures are utilized in specified quantities based upon individual contracts to optimize their performance against different levels of chloride penetration (Jeknavorian 2005, Jeknavorian et al. 1995). Testing methods have been developed to assess if the specified dosages of nitrite corrosion inhibitor have been added to the mixture while it is in the fresh or hardened state. Most highway agencies use these methods as a form of QC for new construction as well as to ensure the levels of calcium nitrite have not depleted below the level needed for sufficient protection over time (Jeknavorian et al. 1995). Several test methods have been developed that allow for detection of the calcium nitrites in both fresh and hardened concrete.

Detecting calcium nitrite presence in fresh concrete is challenging due to the difficulty of bringing analytical instruments into the field. The W. R. Grace corporate research laboratory developed the first method of detecting chloride in hardened concrete in 1980 (Jeknavorian et al. 1995). It involves collecting a powdered sample of concrete and treating it with several reagents before analyzing calorimetrically (Jeknavorian et al. 1995). This method is still widely utilized, but a method utilizing Colorimetry has been adopted due to its greater ease of use and less specialized equipment. In this method, concrete extract is treated with sulfanilic acid that is diazotized by the nitrite. It is then coupled with ethylenediamine to produce a purple color that is measured by a spectrophotometer.

2.7 Corrosion Modeling

Since corrosion is known to be a primary deterioration mechanism in coastal environments, service life prediction models that focus specifically on its impacts are often used. These models allow a quick and cost-effective means to predict how reinforced concrete structures will perform with various design features, concrete mixtures, environmental conditions, and mechanical loads (ACI 2017). Most methods utilize diffusion coefficients and chloride concentration thresholds for onset of corrosion to determine the rate at which chloride ingress occurs and the time for corrosion to initiate and propagate (Bhattacharjee 1998). Most models use the error function solution of Fick's Second Law of Diffusion to quantify the time it takes for chloride ions to penetrate concrete to the depth of the reinforcing steel, but other methods are also used.

The Life-365 consortium developed Life-365 as a modeling software to analyze and estimate the service life and lifecycle costs of concrete structures subjected to failure by chloride ingress induced corrosion (Bentz and Thomas 2018). The consortium was supported by Master Builders Technologies, GCP Applied Technologies, and the Silica Fume Association, with the intention to develop a standardized model to predict corrosion deterioration. This failure mechanism is the primary one modeled and the software does not predict other deleterious processes, such as carbonation or sulfate attack (Bentz and Thomas 2018). It also assumes that the concrete is in ideal condition which is saturated and uncracked (Ehlen et al. 2009). The presence of cracks in concrete allow for chlorides to access the reinforcing steel and cause the early onset of corrosion. The software is often utilized to predict the benefits that specific corrosion prevention measures have on the service life of a structure.

Like many models, its analysis is based upon the error function solution of Fick's Second Law of Diffusion. It assumes that corrosion due to chloride ingress is the failure mode from which to determine the end of the service life. The software allows for user inputs for the geometry of the structure, materials utilized, and chloride exposure conditions. These are all factors of known importance in the longevity of reinforced concrete. It accounts for some of the complex phenomena

associated with corrosion and diffusion by making several assumptions to simplify analysis while being applicable to a wide array of situations (Bentz and Thomas 2018).

Life-365 defines the service life of a structure by the total time necessary for chlorides to penetrate and for corrosion to first begin and then produce damage to a structure. These two stages of the service life are respectively known as the initiation period and the propagation period (Andrade et al. 2012). Factors specific to concrete mix design that impact the time to onset of corrosion are w/cm , use of pozzolans, and corrosion inhibitors. Additional factors related to the design and construction are increased concrete cover, the use of epoxy coated, galvanized, or stainless-steel rebar, and membranes or sealers (Ehlen et al. 2009). Life-365 allows for the input of all these corrosion prevention measures and estimates a service life using the error function of Frick's second law of diffusion (Hodhod and Ahmed 2013). The Fickian diffusion model demonstrates that the initiation period increases with greater concrete cover and corrosion threshold values and it decreases with larger chloride concentration and diffusion coefficients (Hodhod and Ahmed 2013). Because of the way this modeling system estimates service life, determining an accurate diffusion coefficient is critical to achieving the best results from it.

3. FIELD TESTING PROGRAM AND RESULTS

A total of 90 bridges have been constructed in NC within the highly corrosive zone and corrosive zone since 1999, when the corrosion policy was initiated. A subset of these structures were selected for field assessment in consultation with the NCDOT Steering and Implementation Committee (STIC). In the selection of bridges, effort was made to include bridges at sites distributed throughout both corrosive zones and bridges that have been in service for a duration sufficient to demonstrate performance. The research team then visited the structures to determine their current condition and to collect concrete specimens for subsequent lab testing. The following activities were typically performed during the field visit to each bridge:

- Visual assessment to locate and map evidence of existing corrosion, such as cracks, spalls, stains, and efflorescence
- Collection of concrete powder samples from relevant components to measure chloride content and to determine the amount of corrosion inhibitor present in the concrete
- Measurement of corrosion current to detect the onset and relative severity of corrosion
- Measurement of surface resistivity to characterize concrete’s permeability and resistance to chloride ingress

The original project proposal indicated that concrete cores would be collected for water permeability testing, however, on the advice of the NCDOT STIC the value of the results did not warrant the potential damage to the structure. The selection of representative bridges was a first step in undertaking the review of NCDOT’s current corrosion mitigation policy. The selection criteria generally included bridges:

- Located within corrosive or highly corrosive zones
- Crossing a brackish river or creek
- Designed with current corrosion policy specifications
- In service with at least 10 years of coastal exposure
- Safely accessible by the research team using either canoe, rowboat or waders
- With typical secondary road bridge design (ie. no high value or unusual bridge designs)

3.1 Description of Bridges Selected

The locations of the bridges selected for field visits are shown in Figure 3.1. The final selection of bridges was approved by the NCDOT STIC.



Figure 3.1: Location of bridges that were visited

Nine bridges were selected for evaluation, as shown in Table 3.1. Depending on access, the sampled locations are described as either atmospheric or tidal. The contamination of the concrete in the atmospheric zone comes from chloride ions in the marine air, delivered by spraying and splashing. In the tidal zone, the chloride is delivered by long daily wetting and drying cycles (Sun, Xiao, Jian Guo, & Zhao, 2019).

Table 3.1: Selected Bridge Characteristics

Structure #	Zone		Bridge Element		Age at time of site visit
	Atmospheric	Tidal	Pier Cap	Pier	
90056	X	X	X		16
660019	X	X		X	12
660021	X	X		X	15
640010	X	X	X	X	14
90061		X		X	15
90206		X		X	12
150026	X		X		14
260007	X		X		13
150020		X		X	12

3.2 Field Testing Procedures

The work performed during each field visit included field testing and observations. These included a visual survey, NDT methods to determine the current corrosion rate and concrete resistivity, and collection of powder samples for further analysis at the UNC Charlotte laboratory.

3.2.1 Visual Observations

A visual survey for corrosion-related deterioration was completed at each bridge visited. The primary signs of deterioration from corrosion include discoloration or staining, cracking, and spalling. Construction defects that could lead to an increased risk of corrosion or chloride ingress were also of interest and recorded if noticed. Due to the relatively young age of the bridges selected (10 to 15 years old), the team acknowledged that it would be unlikely that many signs of corrosion would be sufficiently manifested to create visual distress at the time of this study.

3.2.2 Corrosion Rate and Concrete Resistivity

Testing of the current corrosion rate and concrete resistivity were completed simultaneously with the Giatec iCOR NDT device. To begin this testing procedure, a flat reinforced concrete surface was selected to map the corrosion rates and concrete resistivity. A pachometer was also utilized to identify the location of reinforcing steel. A testing grid was drawn in chalk over the area to be evaluated and marked with an identifying code (as shown in Figure 3.3). Testing locations were strategically selected if they incorporated multiple accessible elements and areas of worst-case exposure (i.e. not rain-washed, as close to the water-line as possible). In several cases, the locations of greatest interest were fouled by oysters and other marine life. In each of these instances, the team removed the fouling with shovels and freed the testing area from debris as much as possible before making measurements.

3.2.3 Powder Sample Collection

Powder samples were removed from several locations on each bridge using a rotary hammer drill in a manner consistent with the method described in ASTM C1152 and AASHTO T260 (ASTM 2020, AASHTO 2021). Sampling locations were determined based on proximity to water, bridge elements, accessibility, and areas with a high corrosion rate as determined by the Giatec iCOR NDT device. Reinforcing steel locations were mapped with a pachometer prior to drilling

to ensure that holes drilled into the concrete avoided the reinforcing steel. At each location, powder samples were obtained at three to five depths in one-inch increments ranging from a depth of one inch to five inches. The powder samples obtained for each one-inch drill depth are comprised of the concrete ½-inch above and below the representative depth. To avoid the possibility of contamination from previously drilled depths, powder samples were taken using one drill bit to sample and a larger bit to clear the previous sampling hole. A 1 ¼-inch diameter drill bit was utilized as a pilot bit, while a drill bit with a ¾-inch diameter was utilized as the sampling bit. The procedure at each location began by using the larger 1 ¼-inch bit to drill a half inch into the surface of the concrete. The newly drilled pilot hole was cleaned of all loose powder with a vacuum or blast of compressed air before the smaller ¾-inch bit is used to drill from a depth of ½-inch to 1 ½-inches in the center of the pilot hole. To prevent cross-contamination of powder from different depths, each drill bit was cleaned using compressed air and isopropyl alcohol after drilling each hole segment.

Powder created while drilling the one-inch sample was collected in a clean powder collection pan held underneath the drilling location. The powder sample was transferred to a polyethylene bag and labeled with its location and depth. The larger pilot bit was then used to enlarge the hole from the depth of ½-inch to 1 ½-inches along the bore of the sampling hole. The remaining dust was cleaned away with the vacuum. Researchers continued this drilling process until the desired sampling depth was reached. The process is illustrated in Figure 3.2. After all powder samples were acquired the sampling hole was cleaned and filled with a quick setting repair mortar. Special care was taken to ensure that the entire volume of the drill hole was filled with the mortar to ensure that the site of the drill hole would not compromise the integrity of the structure.

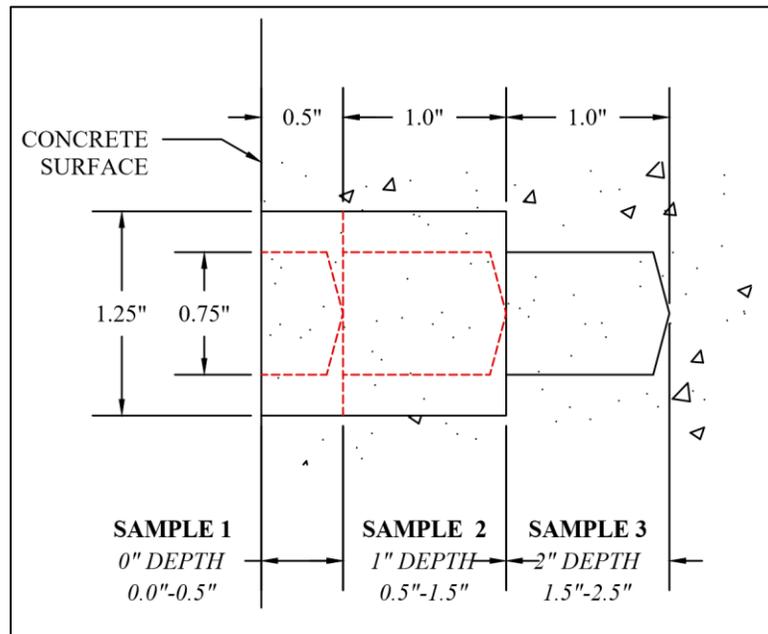


Figure 3.2: Powder sample acquisition process

3.3 Laboratory Testing Procedures

Laboratory testing of concrete powder samples included analysis of powder samples taken during the field-testing portion of the project. These powder samples were returned to the UNC Charlotte laboratory to be tested for chloride concentration and corrosion inhibitor content.

3.3.1 Rapid Chloride Test

Rapid Chloride Tests (RCT), developed and manufactured by Germann Instruments, were used to determine the concentration of chlorides at various depths in the concrete elements. The tests were conducted in duplicate on separate powder samples from each depth at each location. A full summary of the laboratory testing results of all RCT tests conducted is provided in Appendix B. An example of the rapid chloride test results is presented for Structure number 660021, which

provides results that could be considered to represent those found at many of the structures included in the study. This structure is located in the highly corrosive zone crossing Bear Creek. The prestressed pile substructure of Bent 1 was evaluated. Powder samples were acquired from two piles at two zones: the tidal zone, where daily tides inundate and expose the concrete, and the atmospheric zone, where chlorides are deposited in the concrete by splashing, misting, and other forms of atmospheric deposition. The two locations are identified as L1 and L2 for both zones and are labeled in Figure 3.3.

The RCT results indicating the chloride concentrations detected are shown in Table 3.2 and the resulting chloride profile is shown in Figure 3.4. As is apparent in these results, the chloride concentrations from the tidal zone are substantially greater than the atmospheric zone at shallow depths. At greater depths, the chloride concentrations become nearly zero, or background levels of the small amounts of naturally occurring chlorides in the aggregates and concrete mixing water. Another common observation is the considerable variability of surface chloride concentrations between different locations. The tidal concentration at L1 (8.4 lb/yd³) is much less (nearly half) the tidal concentration at L2 (15.7 lb/yd³). This is likely attributable to the fact that L1 is on an exterior pier that may be washed by rain, whereas L2 is in an interior area that is protected from freshwater washing. Also shown in Figure 3.4 is an indication of the depth of steel for the pier. In the tidal zone, the chloride concentrations are around 6 lb/yd³, which would typically be associated with a very elevated probability of corrosion. The steel-depth chloride concentrations in the atmospheric zone are barely above the background level. This observation demonstrates the degree to which exposure is highly variable at different locations on the same element.



Figure 3.3: RCT test locations L1 and L2 on bridge 660021

Table 3.2: RCT Results for test locations L1 and L2 on bridge 660021

Structure #	Zone	Location	Chloride Content		
			Depth (in)	% Conc. by Wt.	Chlorides (lb/CY)
660021	Tidal	L1	0	0.220	8.397
			1	0.243	9.252
			2	0.150	5.738
			3	0.094	3.592
			4	0.018	0.705
			5	0.012	0.443
	Atmospheric	L1	0	0.115	4.404
			1	0.023	0.887
			2	0.004	0.156
			3	0.003	0.109
			4	0.003	0.105
			5	0.003	0.102
	Tidal	L2	0	0.412	15.718
			1	0.297	11.327
			2	0.161	6.157
			3	0.030	1.159
			4	0.006	0.247
			5	0.005	0.185
	Atmospheric	L2	0	0.183	6.987
			1	0.043	1.628
2			0.004	0.145	
3			0.004	0.142	
4			0.003	0.112	
5			0.003	0.116	

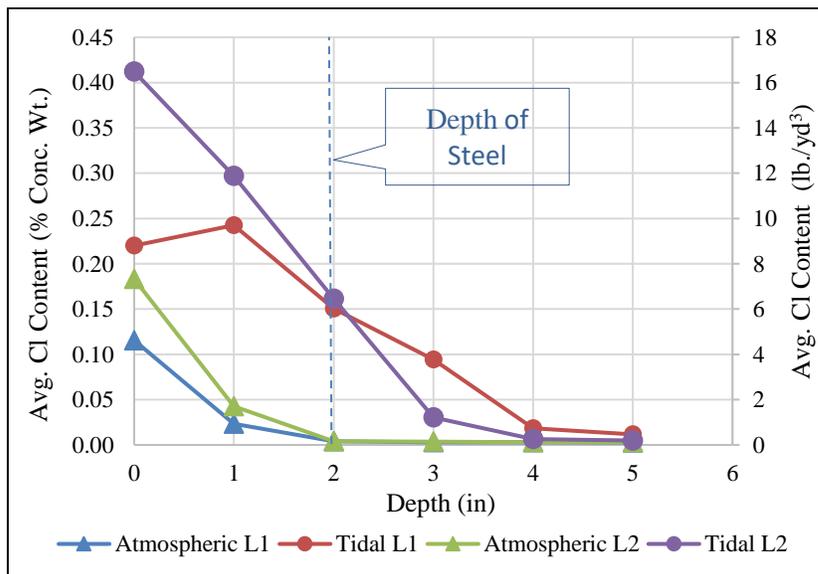


Figure 3.4: Chloride profile for locations L1 and L2 on bridge 660021

3.3.2 Surface Resistivity and Corrosion Rate Testing

The Giatec iCOR was used to measure the rate of ongoing corrosion and surface resistivity of the concrete. The device is capable of performing both tests simultaneously. The testing locations were selected to obtain data at different levels of exposure. Data was collected at elevations within both the atmospheric and tidal exposure zones of the piers and pier caps. If accessible, data was also collected from the underside of the cored slabs, which typically receives very low exposure depending on the structure’s height above the water. To ensure consistency of results at each structure, the team attempted to obtain a similar number of readings from each element of interest at each structure. This was not always possible due to accessibility challenges, such as excessive height above the water or piers that were mostly submerged. Due to the results being deemed typical of many structures included in the study, structure number 660021 was again selected to demonstrate the corrosion and concrete resistivity rates for tidal and atmospheric zones, which are presented in Table 3.3. The corrosion rate found in the atmospheric zone was relatively low and indicated a passive condition by the instrument manufacturer’s guidelines. In the tidal zone, measurements indicate substantial ongoing corrosion. A summary of the results collected from all structures is provided in Table 3.4.

Table 3.3: Example of surface resistivity and corrosion rate data collected from structure # 660021

Structure #	Location	Atmospheric Zone				Tidal Zone			
		Corrosion Rate (µm/year)	Avg. Corrosion Rate (µm/year)	Concrete Resistivity (kΩ·cm)	Avg. Concrete Resistivity (kΩ·cm)	Corrosion Rate (µm/year)	Avg. Corrosion Rate (µm/year)	Concrete Resistivity (kΩ·cm)	Avg. Concrete Resistivity (kΩ·cm)
660021	L1	4.2	4.3	119	179.5	403.1	197.1	15	15.0
		5.4		112		13.2		28	
		4.6		208		230.5		11	
		1.1		204		335.8		9	
		4.1		234		122		10	
		6.5		200		181.8		17	
						153.2		22	
			162.4	12					
			171.8	11					
	L2	0.23	3.7	257	197.8	15.17	88.3	74	27.5
		2.9		226		135.9		23	
		3.2		178		113.6		14	
		3		191		78.24		12	
		2.6		177		46.31		56	
10		158		51		21			
				147.3		10			
		119.3	10						

Table 3.4: Summary of surface resistivity and corrosion rate data collected from all bridges visited

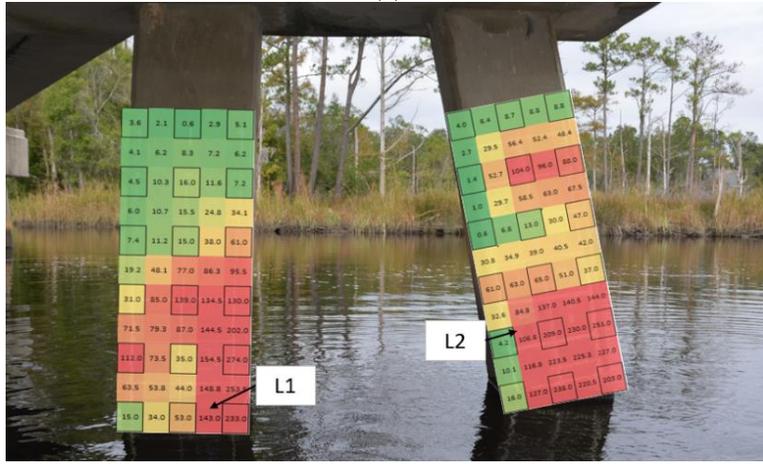
Structure #	Location	Bridge Element	Pozzolans	Zone	Corrosion Rate ($\mu\text{m}/\text{year}$)		Surface Resistivity ($\text{k}\Omega\cdot\text{cm}$)	
					Avg.	Max.	Avg.	Min.
660019	L1	Prestressed Pile	Silica Fume	Atmospheric	56	90	81	49
				Tidal	78	191	32	10
	L2	Prestressed Pile	Silica Fume	Atmospheric	38	136	96	51
				Tidal	64	218	57	7
090061	L1	Prestressed Pile	Silica Fume	Tidal	120	274	28	7
	L2	Prestressed Pile	Silica Fume	Tidal	154	251	23	7
640010	L1	Bent 1 Cap	Fly Ash/Silica Fume	Atmospheric	3	7	361	172
	L2	Prestressed Pile	Silica Fume	Atmospheric	42	77	118	91
				Tidal	26	45	61	32
	L3	Bent 3 Cap	Fly Ash/Silica Fume	Atmospheric	3	5	103	91
	L4	Prestressed Pile	Silica Fume	Atmospheric	-	-	-	-
				Tidal	30	40	52	32
660021	L1	Prestressed Pile	No Pozzolans	Atmospheric	4	7	212	200
				Tidal	197	403	28	9
	L2	Prestressed Pile	No Pozzolans	Atmospheric	5	10	176	158
				Tidal	88	147	35	10
090206	L1	Prestressed Pile	Fly Ash/Silica Fume	Tidal	83	120	35	25
	L2	Prestressed Pile	Fly Ash/Silica Fume	Tidal	103	138	30	13
090056	L1	Bent 2 Cap	Fly Ash/Silica Fume	Atmospheric	28	99	171	46
				Tidal	81	155	45	16
	L2	Bent 3 Cap	Fly Ash/Silica Fume	Tidal	30	87	90	24
150026	L2	Bent 1 Cap	Fly Ash	Tidal	-	-	-	-
	L3	Bent 1 Cap	Fly Ash	Tidal	9	28	305	199
260007	L1	Bent 3 Cap	Slag	Atmospheric	22	52	187	67
	L2	Bent 3 Cap	Slag	Atmospheric	31	52	137	67
150020	L1	Prestressed Pile	Fly Ash	Atmospheric	5	7	190	109
	L2	Prestressed Pile	Fly Ash	Atmospheric	4	11	518	407

"- " Indicates data was not collected or technical issues at time of testing

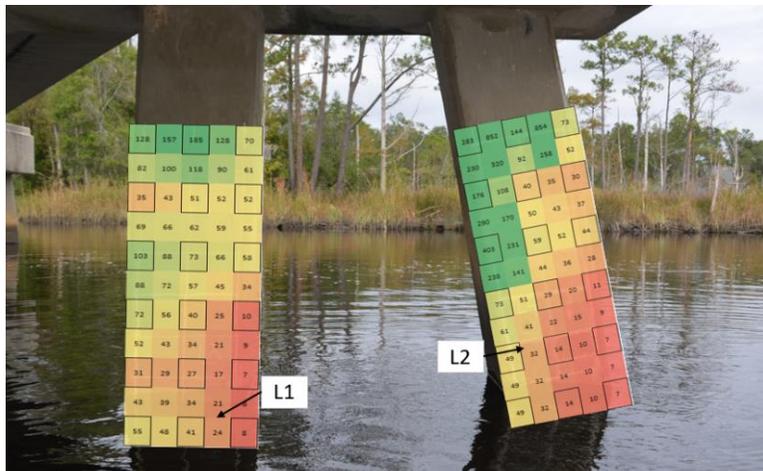
The heat maps shown in Figure 3.5 are a representation of how corrosion rates and surface resistivity were typically distributed on elements. As could be expected, the highest rates of ongoing corrosion were often discovered within the splash or tidal zone where daily wetting cycles occur. Outside of the splash zone, values tended to be much lower. This same trend was seen with regard to surface resistivity. Because the moisture content of the concrete was not controlled during the field testing, there was often considerable variability between wet and dry locations.



(a)



(b)



(c)

Figure 3.5. (a) Perspective on high tide location, (b) Piers: corrosion rate heat map (c) Piers: Surface resistivity heat map

3.3.3 Diffusion Coefficient and Surface Concentration Calculations

The diffusion coefficient is a parameter in Fick’s second law of diffusion that describes the permeability of a medium to a particular species or agent. The chloride concentration profiles that were collected from the field provide datapoints that can be used to fit a solution of Fick’s law. One solution, that relates the concentration of chloride at a particular depth after a particular time of exposure, given a known concentration at the surface, is shown as eq 3.1 .

$$C(x, t) = C_0 \left\{ 1 - \operatorname{erf} \left(\frac{x}{2\sqrt{D_c \times t}} \right) \right\} \quad (3.1)$$

where C is the concentration of chlorides at depth x and time t , C_0 is the surface chloride concentration, erf is the error function, and D_c is the chloride diffusion coefficient in in^2/year . Since the bridge’s age and exposure duration, t , is known and the concentration at depth x is known, the parameters that require solving by curve fitting are D_c and C_0 . Fitting the parameters was undertaken using the Solver package in Microsoft Excel. A set of optimization constraints were created within Solver to estimate a value for D_c so that it would be 1) consistent between locations sampled on similar elements (i.e. those cast at the same time and with the same concrete mixtures, such as two locations on the same pier), and 2) accurate to predict chloride concentration at depth x given C_0 . The optimization routine arrived at values for D_c and C_0 by nominating values that minimized the squared difference between the field-measured data point and the modeled datapoint. The process the Solver function uses to fit the data is iterative non-linear least-squares regression, and it accepts user input of the model non-linear function (in this case, Eq. 3.1).

The iterative non-linear least-squares regression model output tables can be seen in Appendix B. A summary of the surface concentration and diffusion coefficient information calculated for the iterative non-linear least-squares regression model can be seen in Table 3.5.

Table 3.5: Summary of measured diffusion coefficients and surface concentrations

Structure #	Location	Bridge Element	Pozzolans	Zone	Diffusion Coefficient (in^2/yr)	Chloride Surface Concentration (lb/yd^3)	
						Model Estimated	Field Measured
660019	L1	Prestressed Pile	Silica Fume	Atmospheric	0.0443	10.56	11.10
				Tidal	0.0443	11.81	11.53
	L2	Prestressed Pile	Silica Fume	Atmospheric	0.0443	12.61	13.00
				Tidal	0.0443	17.87	17.43
090061	L1	Prestressed Pile	Silica Fume	Tidal	0.2587	18.48	15.89
	L2	Prestressed Pile	Silica Fume	Tidal	0.2587	28.42	28.97
640010	L1	Bent 1 Cap	Fly Ash/Silica Fume	Atmospheric	0.0439	0.23	-
	L2	Prestressed Pile	Silica Fume	Atmospheric	0.0454	8.15	8.04
				Tidal	0.0454	11.13	11.24
	L3	Bent 3 Cap	Fly Ash/Silica Fume	Atmospheric	0.0439	4.38	4.40
	L4	Prestressed Pile	Silica Fume	Atmospheric	0.0454	15.71	14.53
				Tidal	0.0454	21.68	22.42
660021	L1	Prestressed Pile	No Pozzolans	Atmospheric	0.1686	3.15	4.40
				Tidal	0.1686	10.77	8.40
	L2	Prestressed Pile	No Pozzolans	Atmospheric	0.1686	5.06	6.99
				Tidal	0.1686	15.95	15.72

090206	L1	Prestressed Pile	Fly Ash/Silica Fume	Tidal	0.0513	57.25	57.24
	L2	Prestressed Pile	Fly Ash/Silica Fume	Tidal	0.0513	66.85	67.23
090056	L1	Bent 2 Cap	Fly Ash/Silica Fume	Atmospheric	0.0695	10.08	12.13
				Tidal	0.0695	26.43	23.61
150026	L2	Bent 3 Cap	Fly Ash/Silica Fume	Tidal	0.0695	30.10	28.37
	L3	Bent 1 Cap	Fly Ash	Tidal	0.0365	18.38	18.06
260007	L1	Bent 3 Cap	Slag	Atmospheric	0.0472	6.01	6.19
	L2	Bent 3 Cap	Slag	Atmospheric	0.0472	3.90	3.61
150020	L1	Prestressed Pile	Fly Ash	Atmospheric	0.0261	10.69	10.78
	L2	Prestressed Pile	Fly Ash	Atmospheric	0.0261	19.10	11.82

The feasibility of the values generated by the curve fitting routine were verified by two benchmarking techniques. Figure 3.6 provides a comparison of the surface chloride concentration measured in the field with the value predicted by the modeling parameters determined by curve fitting. As is evident in Figure 3.6 as well as by comparing the rightmost two columns of Table 3.5, the values are very similar, indicating the reliability of the model to relate surface concentration with diffusion coefficient and predict the current conditions. Table 3.6 provides the range of values determined for concretes containing various SCMs determined from this study. These can be compared with a second set of reference point values that were developed by Rochelle (2000) for NCDOT for use with coastal bridge service life modeling. The model was used in the design of the Manteo Bypass bridge. For comparative purposes, the values estimated from field data collected during this project are shown alongside the typical values based on Rochelle's work. Most values are either consistent with the ranges that Rochelle prepared for various concrete elements, or slightly lower. Only two were found to be higher, however these structures had either physical defects or did not contain pozzolans.

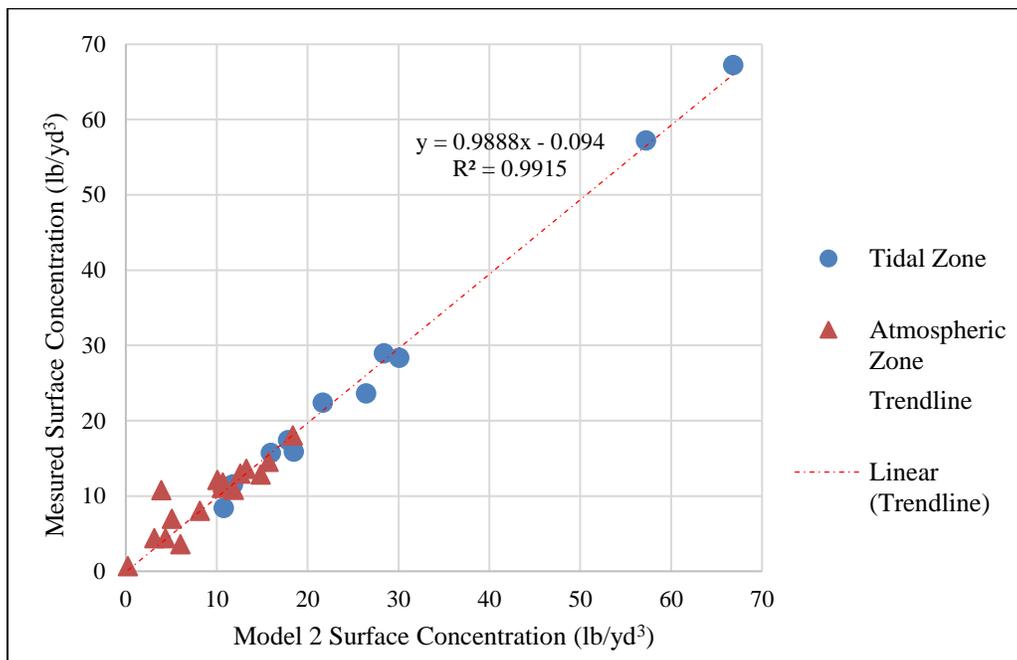


Figure 3.6: Measured vs modeled surface concentration

Table 3.6: Diffusion coefficient range for various concrete mixtures determined by testing field samples

Pozzolans	Diffusion Coefficient (in ² /yr)
Silica Fume	0.025-0.045
Fly Ash	0.025-0.045
Slag	0.045-0.065
Silica Fume/Fly Ash	0.045-0.075
No Pozzolans	>0.2

Table 3.7: Estimated tidal zone diffusion coefficients compared to Manteo Bypass service life model (Rochelle, 2000)

Structure #	Bridge Element	Iterative Non-linear Least-Squares Regression					Manteo Bypass Durability Design Model	
		Diffusion Coefficient (in ² /yr)	Corrosion Inhibitor (gal/yd ³)	Fly Ash %	Slag %	Silica Fume %	Diffusion Coefficient (in ² /yrs.)	Value from RP 2019-22 corresponding with Manteo Bypass model?
660019	Prestressed Pile	0.044	3	0	0	5	0.0392 - 0.1225	In Range
090061	Prestressed Pile	0.259	3	0	0	5	0.0392 - 0.1225	Above Range (High)
640010	Bent Cap	0.045	3	25	0	5	0.0784 - 0.147	Below Range (Low)
	Prestressed Pile	0.045	3.5	0	0	5	0.0392 - 0.1225	In Range
660021	Prestressed Pile	0.169	3.5	0	0	0	0.0392 - 0.1225	Above Range (High)
090206	Prestressed Pile	0.051	3	0	0	5	0.0392 - 0.1225	In Range
090056	Bent Cap	0.070	3	25	0	5	0.0784 - 0.147	Below Range (Low)
150026	Bent Cap	0.037	3	30	0	0	0.0784 - 0.147	Below Range (Low)
260007	Bent Cap	0.047	3	0	43	0	0.0784 - 0.147	Below Range (Low)
150020	Prestressed Pile	0.026	3	30	0	0	0.0392 - 0.1225	Below Range (Low)

3.3.4 Corrosion Inhibitor Concentration

Once concrete has hardened, the concentration of corrosion inhibitor can be determined from field-collected powder samples using chemical analysis. This technique is sometimes performed for quality assurance to ensure the correct dosages of the inhibitor were added and that the admixture was thoroughly mixed into the concrete. The method utilized for determining the corrosion inhibitor concentration in the bridge elements included in this study was adopted from the W. R. Grace chemical procedure #C-20.0 for Determination of Nitrite in Hardened Concrete (Jeknavorian 2005), and is described in Appendix B.

To verify that the minimum dosage rates and uniform dispersion of the calcium nitrite admixture were achieved, powder samples from two different depths at one location from each bridge were selected for testing to measure the remaining concentration of corrosion inhibitor. Each bridge location selected included a test of the powder taken from two inches (which is the depth of the outermost steel) and of the powder taken at the deepest depth sampled (which in most cases was five inches).

The calcium nitrite extraction procedure utilized has known limitations in recovery rate, with expected recovery rates between 85% and 96% of the theoretical calcium nitrite concentration within the concrete published in the literature (Jeknavorian 2005). To acknowledge the expected range of extraction efficiency, two values for the theoretical

concentration are reported beside the Lab Measured concentration in Table 3.8. The first theoretical concentration assumes a recovery rate of 96%. This is a conservative value as it is assuming that all but 4% of the calcium nitrite was extracted and allowed to be measured. The second is the theoretical concentration assuming a recovery rate of 85%. This is a more realistic value which assumes that 15% of the nitrite was not extracted.

Table 3.8: Theoretical calcium nitrite concentrations of varying recovery rates

Bridge-Location	Sample Depth (in)	Calcium Nitrite Concentration (gal/CY)		
		Lab Measured	Theoretical (96% Recovery Rate)	Theoretical (85% Recovery Rate)
150020-L2	2	2.98	3.11	3.51
	5	3.03	3.15	3.56
660019-L1	2	3.62	3.77	4.26
	3	4.28	4.46	5.03
260007-L1	2	3.14	3.27	3.69
	5	1.27	1.32	1.49
640010-L1	2	2.38	2.48	2.8
	5	2.84	2.96	3.35
660021-L1	2	3.87	4.03	4.55
	5	3.2	3.33	3.76
090061-L1	2	1.96	2.04	2.3
	5	3.05	3.18	3.59
150026-L2	2	2.79	2.9	3.28
	5	3.13	3.26	3.68

The theoretical concentrations for both the minimum theoretical concentration (with a recovery rate of 96%) and the maximum theoretical concentration (with a recovery rate of 85%) are displayed in Figure 3.7. This figure illustrates that in most cases the theoretical calcium nitrite concentration from the powder samples met or exceeded the NCDOT minimum required. This verifies that calcium nitrite is being utilized in the mixtures at approximately the correct dosage rates. In all samples where the minimum was not met, a powder sample acquired from a different depth tested positively for a calcium nitrite concentration that meets specifications. This phenomenon is most likely explained by variations in the powder samples due to concrete being a composite material. The testing procedure calculations account for an anticipated portion of the powder sample being comprised of coarse or fine aggregate which will not contain any calcium nitrite, and an anticipated portion of the sample being paste which will contain calcium nitrite. If a drilled powder sample happened to contain a larger than average proportion of coarse aggregate powder it would cause a test result indicating an artificially low concentration of calcium nitrite. This is most likely the cause of lower than expected concentrations of calcium nitrite on a few of the tests. None of the samples tested indicated lower than required corrosion inhibitor concentration at both depths sampled.

The calcium nitrite detected at the depth of steel (2-inches) and deeper into the concrete element (3 to 5-inches) were found to be either similar concentrations or show no discernable trend between where the calcium nitrite is most concentrated. This suggests that the corrosion inhibitor admixture was generally mixed uniformly within the concrete. This is desirable to ensure that all embedded steel is afforded the same level of corrosion protection by the admixture.

The NCDOT Materials and Tests unit has used the same procedure (W.R. Grace Chemical Procedure #C-20.0) to verify proper dosing of corrosion inhibiting admixture. Table 3.9 provides a comparison of measurements taken by both UNC Charlotte and NCDOT at four locations. The NCDOT measurements were taken during construction and the UNC Charlotte measurements were taken after 11-14 years of service. There is no discernible trend that indicates that the concentration was impacted by time. It is also important to note that the NCDOT measured values reported in Table 3.9 are the average of several tests conducted at a variety of locations on the concrete element. Amongst these test results it was common to see a wide range of results of both acceptable and unacceptable nitrite concentrations within the same element.

The average of these results in some cases fell below the minimum specified concentration of 3.0 gal/CY, but all locations had at least one test indicating that the concentration was above the limits, and therefore all elements tested were ultimately accepted for use by the NCDOT. The variability of these test results, and the acceptance of even lower than expected concentrations, indicate that this procedure for determining nitrite concentration is utilized as a simple validation that calcium nitrite is present within the concrete mixture.

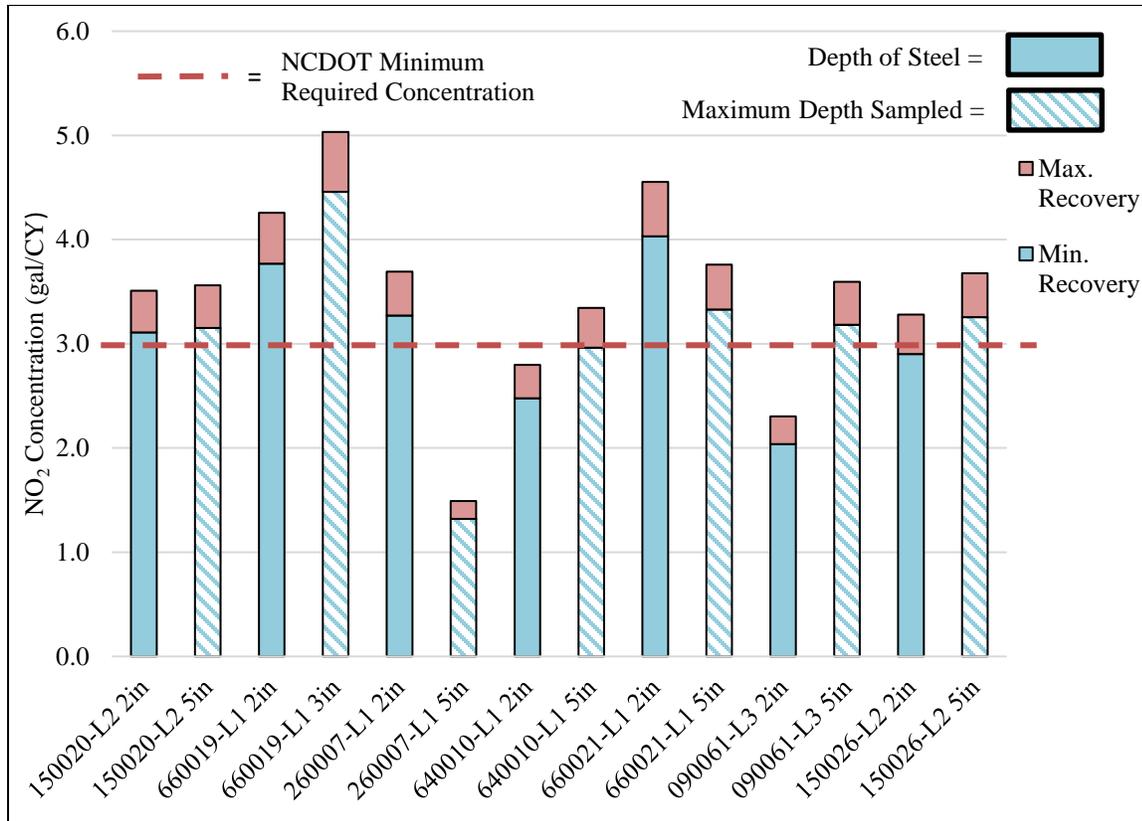


Figure 3.7: Theoretical calcium nitrite concentrations

Table 3.9: Calcium nitrite concentration as measured by UNC Charlotte and NCDOT

Bridge-Location	UNC Charlotte Measured		NCDOT Measured	Test Status	% Difference NCDOT Measured vs. UNC Charlotte Measured
	Time Since Acceptance Tested (yrs)	Average Concentration (gal/CY)	Average Concentration (gal/CY)		
150020-L2	≈12	3.01	2.87	Accepted	-5%
660019-L1	≈14	3.95	2.61	Accepted	-51%
090061-L1	≈14	2.51	2.96	Accepted	15%
150026-L2	≈11	2.96	3.49	Accepted	15%

4. SERVICE LIFE MODELING

Service life modeling was completed using Life-365 software. The modeling strategy for this project used information gathered from the field study to determine the concrete performance characteristics. The information and variables utilized to complete the model and the results of the modeling process are provided in this chapter and further elaborated in Appendix A. The service life models were used to assess whether concrete structures were made sufficiently durable by use of the provisions currently included in the corrosion policy in the Structural Design Manual.

4.1 Model Inputs

The Life-365 software estimates service life based on material properties, exposure, and component geometry. For this study, field data regarding concrete material properties and exposure was collected for prestressed piles and bent caps (as described in Chapter 3). The type of element, its dimensions, and the concrete cover used on each component modeled are shown in Table 4.1. The exposure conditions were inputted based upon as many field measured datapoints as possible. These conditions, which can be seen in Table 4.2, include the estimated surface concentration and the buildup period which is the age of the bridge. The average monthly temperature values were populated with values for the region surrounding one of three North Carolina cities: Jacksonville, Wilmington, and Nags Head (whichever was closest to the structure). The vertical distance from high tide elevation that is reported for each location is an approximate value identified using the approximate elevation where high tide was observed on the day of testing. Many factors impact the elevation of the high tide on a day to day basis, so the reported value is only approximated to the nearest half foot. Negative values indicate that the sampling location was below the high tide mark on the structure. The team attempted to sample all the structures in the tidal zone at least one foot below the high tide markings. The value, *m*, is a diffusion decay modifier that is based on the quantity of fly ash or silica fume present in the mixture.

Table 4.1: Geometry and element type inputs of modeled locations (tidal zone)

Structure Number	Location	Vertical Distance from High Tide Elevation (ft)	Bridge Element	Dimensions (in)	Cover (in)
660019	L1	-1	Prestressed Pile	16 x 16	2
	L2	-1	Prestressed Pile	16 x 16	2
090061	L3	-1	Prestressed Pile	20 x 20	2
	L4	-1	Prestressed Pile	20 x 20	2
640010	L1*	5	Bent 1 Cap	30 x 33	2
	L2	-1	Prestressed Pile	16 x 16	2
	L3*	5	Bent 3 Cap	30 x 33	2
	L4	-1	Prestressed Pile	16 x 16	2
660021	L1	-1	Prestressed Pile	12 x 12	2
	L2	-1	Prestressed Pile	12 x 12	2
090206	L1	-1	Prestressed Pile	16 x 16	2
	L2	-1	Prestressed Pile	16 x 16	2
090056	L1	-1	Bent 2 Cap	30 x 33	2
	L2	-1	Bent 3 Cap	30 x 33	2
150026	L2	-1	Bent 1 Cap	42 x 44	2
	L3	-1	Bent 1 Cap	42 x 44	2
260007*	L1*	2	Bent 3 Cap	33 x 50	2
	L2*	2	Bent 3 Cap	33 x 50	2
150020*	L1*	-1	Prestressed Pile	20 x 20	2
	L2**	-1	Prestressed Pile	20 x 20	2

* Atmospheric

** Approximated

Table 4.2: Exposure condition inputs of modeled locations (tidal zone)

Structure Number	Location	Vertical Distance from High Tide Elevation (ft)	Age (yrs.)	Surface Con. (% wt. conc)	Temperature Region
660019	L1	-1	12	0.309	Jacksonville
	L2	-1		0.468	
090061	L3	-1	15	0.485	Wilmington
	L4	-1		0.745	
640010	L1*	5	14	0.006	Wilmington
	L2	-1		0.292	
	L3*	5		0.115	
	L4	-1		0.568	
660021	L1	-1	15	0.282	Jacksonville
	L2	-1		0.418	
090206	L1	-1	12	1.501	Wilmington
	L2	-1		1.752	
090056	L1	-1	16	0.693	Wilmington
	L2	-1		0.789	
150026	L2	-1	14	0.482	Jacksonville
	L3	-1		0.347	
260007*	L1*	2	13	0.158	Nags Head
	L2*	2		0.102	
150020*	L1*	-1	12	0.280	Jacksonville
	L2**	-1		0.501	
*	Atmospheric				
**	Approximated				

Table 4.3: Corrosion protection in concrete mixtures

Structure Number	Location	Corrosion Inhibitor (gal/cy)	Fly Ash	Slag	Silica Fume
660019	L1	3	-	-	5%
	L2	3	-	-	5%
090061	L3	3	-	-	5%
	L4	3	-	-	5%
640010	L1*	3	25%	-	5%
	L2	3.5	-	-	5%
	L3*	3	25%	-	5%
	L4	3.5	-	-	5%
660021	L1	3.5	-	-	-
	L2	3.5	-	-	-

090206	L1	3	-	-	5%
	L2	3	-	-	5%
090056	L1	3	25%	-	5%
	L2	3	25%	-	5%
150026	L2	3	30%	-	-
	L3	3	30%	-	-
260007*	L1*	3	-	43%	-
	L2*	3	-	43%	-
150020*	L1*	3	30%-	-	-
	L2*	3	30%-	-	-

* Atmospheric

Table 4.4: Concrete property input values of modeled locations

Structure Number	Location	Constant Values			Estimated Values	
		Hydration (yr.)	Propagation Period (yr.)	Ct (% wt. conc.)	Diffusion Coefficient at 28 days (in ² /in ² /sec)	m
660019	L1	25	6	0.24	2.33E-09	0.26
	L2				2.33E-09	
090061	L3			0.24	1.45E-08	0.26
	L4				1.45E-08	
640010	L1*			0.24	4.89E-09	0.40
	L2			0.28	2.49E-09	0.26
	L3*			0.24	4.89E-09	0.40
	L4			0.28	2.49E-09	0.26
660021	L1			0.28	2.15E-08	0.26
	L2				2.15E-08	
090206	L1			0.24	7.65E-09	0.26
	L2				7.65E-09	
090056	L1			0.24	8.17E-09	0.40
	L2				8.17E-09	
150026	L2			0.24	1.14E-08	0.44
	L3				1.14E-08	
260007	L1*			0.24	1.47E-08	0.45
	L2*				1.47E-08	
150020	L1*			0.24	7.65E-09	0.44
	L2*				7.65E-09	

* Atmospheric

The results output from the service life modeling process using both methods are summarized in Table 4.5. The total maintenance free service life from the time of construction was estimated based on exposure conditions for each individual element modeled. Because these can vary substantially based on individual conditions, the total maintenance-

free bridge service life is reported as the minimum service life of any of the components modeled on the bridge. Using the field measured current conditions as inputs, 67% of structural concrete bridge elements observed were predicted to have a maintenance-free service life greater than 50 years, and 33% of structural concrete bridge elements observed were predicted to have a maintenance-free service life less than 50 years.

Table 4.5: Summary of service life modeling (tidal zone)

Structure Number	Location	Corrosive Zone	Bridge Element	Vertical Distance from High Tide Elevation (ft)	Distance from open water (mi)	Component Service life (yr)	Total Maintenance Free Life (yr)
660019	L1	Corrosive	Prestressed Pile	-1	4.95	337.	141
	L2		Prestressed Pile	-1		141	
090061	L3	Highly Corrosive	Prestressed Pile	-1	3.88	31	24
	L4		Prestressed Pile	-1		24	
640010	L1*	Highly Corrosive	Bent 1 Cap	5	3.87	506	111
	L2		Prestressed Pile	-1		506	
	L3*		Bent 3 Cap	5		506	
	L4		Prestressed Pile	-1		111	
660021	L1	Highly Corrosive	Prestressed Pile	-1	6.42	506	33
	L2		Prestressed Pile	-1		33	
090206	L1	Highly Corrosive	Prestressed Pile	-1	2.07	22	21
	L2		Prestressed Pile	-1		21	
090056	L1	Highly Corrosive	Bent 2 Cap	-1	1.15	57	52
	L2		Bent 3 Cap	-1		52	
150026	L2	Highly Corrosive	Bent 1 Cap	-1	7.29	-	139
	L3		Bent 1 Cap	-1		134	
260007	L1*	Highly Corrosive	Bent 3 Cap	2	0	506	506
	L2*		Bent 3 Cap	2		506	
150020	L1*	Corrosive	Prestressed Pile	-1	2.5	409	104
	L2*		Prestressed Pile	-1		104	

* Atmospheric

** Approximated

Since the components that were considered in this study primarily featured very similar types of concrete mixtures due to the requirements of the corrosion policy, most had very similar diffusion coefficients. The level of exposure, and therefore, the surface concentration was the most important variable that impacted the expected duration of service life. Based on the results of the modeling it appears that the main factor impacting the service life is the frequency with which the concrete element is exposed to the chloride-rich water. In all cases of modeling locations in the atmospheric zone, the maximum service life of 506 years was estimated by both modeling methods (this is the maximum timeframe considered

by the software). This indicates that in locations where concrete elements do not experience heavy chloride loading from regular exposure to chloride rich waters, the risk of corrosion-related deterioration is low. In tidal areas exposed to frequent wetting and drying cycles, the predicted service lives were markedly shorter, with some structures having less than 50 years of predicted maintenance free service life. Frequent wetting and drying of the concrete is known to enhance chloride ingress due to the absorptive properties of the concrete capillary pore structure (ACI 2010).

The variation in surface concentration between multiple locations for the same structure can be seen in Table 4.6. The chloride concentration is a direct measurement of the severity of exposure to chloride. Rochelle (2000) estimated that severe exposures at the NC coast would have chloride concentrations in the range of 15 to 25 lb/yd³. In fact, during this study, chloride concentrations up to approximately 31 lb/yd³ were determined. However, the average surface concentration in the tidal zone locations sampled was 0.515% by weight of concrete or 19.6 lb/yd³.

As expected, the diffusion coefficient is an important variable related to the ingress of chlorides and the tendency to corrode. Figure 4.1 relates the diffusion coefficient estimated from concrete samples and existing exposure, with the modeled service life of the bridge. The three structures with a predicted service life greater than 100 years had diffusion coefficients less than 0.05 in²/year. Structures predicted to have service life of less than 50 years were associated with diffusion coefficients greater than 0.15 in²/year. Although service life is a function of both permeability and exposure, the importance of low permeability to chloride ions is demonstrated by this relationship. Note that the structure with heavy fouling and a construction defect (and notably lower expected remaining service life) was excluded from this model.

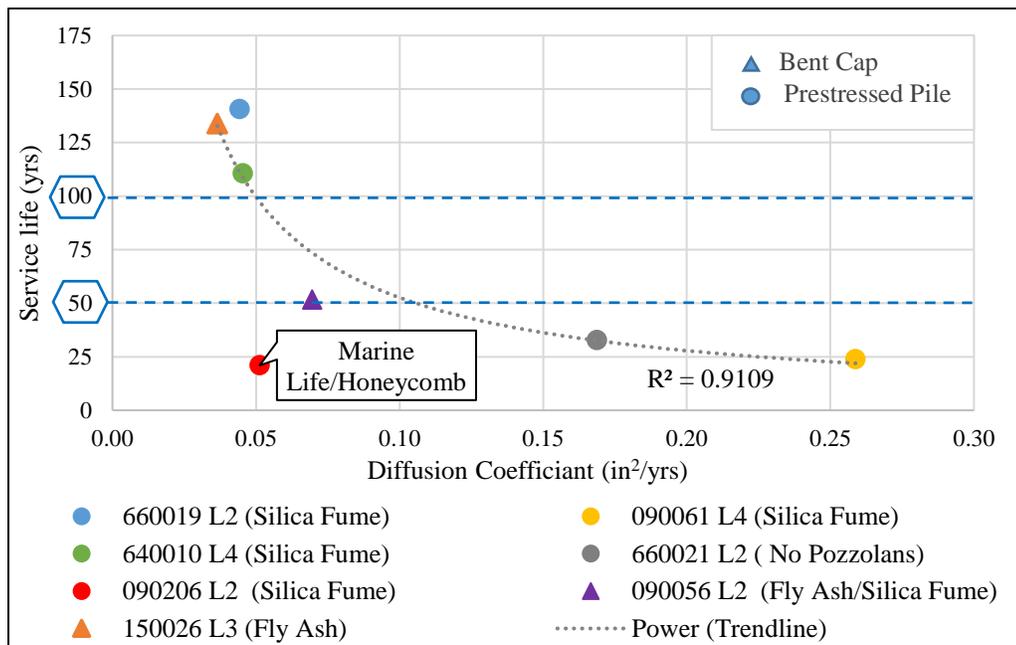


Figure 4.1: Relationship between diffusion coefficient and predicted service life

Table 4.6: Summary of surface concentration and service life

Structure Number	Location	Surface Con. (lb/yd ³)	Diffusion Coefficient at 28 days (in ² /sec)	Component Service life (yrs.)	Total Maintenance Free Life (yrs.)	Diffusion Coefficient at 28 days (in ² /sec)	Component Service life (yrs.)
660019	L1	11.805	2.33E-09	337	141	4.09E-09	143
	L2	17.866	2.33E-09	141		4.09E-09	63
090061	L3	18.484	1.45E-08	31	24	9.34E-09	61
	L4	28.419	1.45E-08	24		9.26E-09	42
640010	L1*	0.233	4.89E-09	506	111	7.85E-09	506
	L2	11.125	2.49E-09	506		7.85E-09	506.
	L3*	4.382	4.89E-09	506		7.85E-09	137
	L4	21.684	2.49E-09	111		7.85E-09	63
660021	L1	10.765	2.15E-08	506	33	7.91E-09	506
	L2	15.952	2.15E-08	33		7.91E-09	53
090206	L1	57.248	7.65E-09	22	21	4.83E-9	25
	L2	66.845	7.65E-09	21		4.83E-9	230
090056	L1	26.433	8.17E-09	57	52	4.57E-09	94
	L2	30.095	8.17E-09	52		5.40E-09	84
150026	L2	18.375	1.14E-08	-	139	9.87E-09	294
	L3	13.257	1.14E-08	134		9.87E-09	132
260007	L1*	6.012	1.47E-08	506	506	9.34E-09	506
	L2*	3.900	1.47E-08	506		9.34E-09	506
150020	L1**	10.686	7.65E-09	409	104	9.87E-09	319
	L2**	19.097	7.65E-09	104		9.87E-09	82

* Atmospheric
 ** Approximated based on data from locations with similar proximity to open water

5. ANALYSIS OF EXPOSURE STUDY AND SERVICE LIFE MODELING RESULTS

During the field portion of this research project, material samples were collected from several coastal NC bridges that were built in corrosive and highly corrosive designated areas. These samples enabled the team to estimate current levels of exposure and chloride permeability (diffusion coefficients) for locations on the piers and bent caps that are within the tidal zone. The team also measured rates of ongoing corrosion. This chapter presents an analysis that combines the findings from data collected in the field with the results of the service life modeling in order to describe correlations between exposure and durability.

5.1 Minimum service life vs. Maximum surface concentration (Tidal Zone)

High chloride surface concentrations were consistently associated with shorter predicted service life. This relationship can be seen in Figure 5.1. Although a total expected maintenance-free service life was predicted for about one third of the bridges sampled in the tidal zone, the remainder of the bridges in the tidal zone have a predicted total maintenance-free service life of less than 100 years. The linkage between service life and exposure showed that bridges with a chloride surface concentration greater than 25 lbs/yd³ were predicted to have a service life of less than 75 years on their current track. Structure number 660021 was excluded from the relationship because its mixture design did not include pozzolans. Structure number 090206 was found to have an unusually high surface chloride concentration (66.8 lb/yd³), which indicates that its exposure conditions may have been unique. In this case heavy fouling from oysters was present, as well as several honeycombs on the surface of the piers. The effects of marine organisms, such as algae, mollusks, bacteria, and crustaceans, on marine structures can increase exposure to chlorides (BS 6349-1-1 2013). Some marine mollusks living in warm coastal waters bore into the concrete surface and lessen the protective capacity of concrete cover (PIANC 1990).

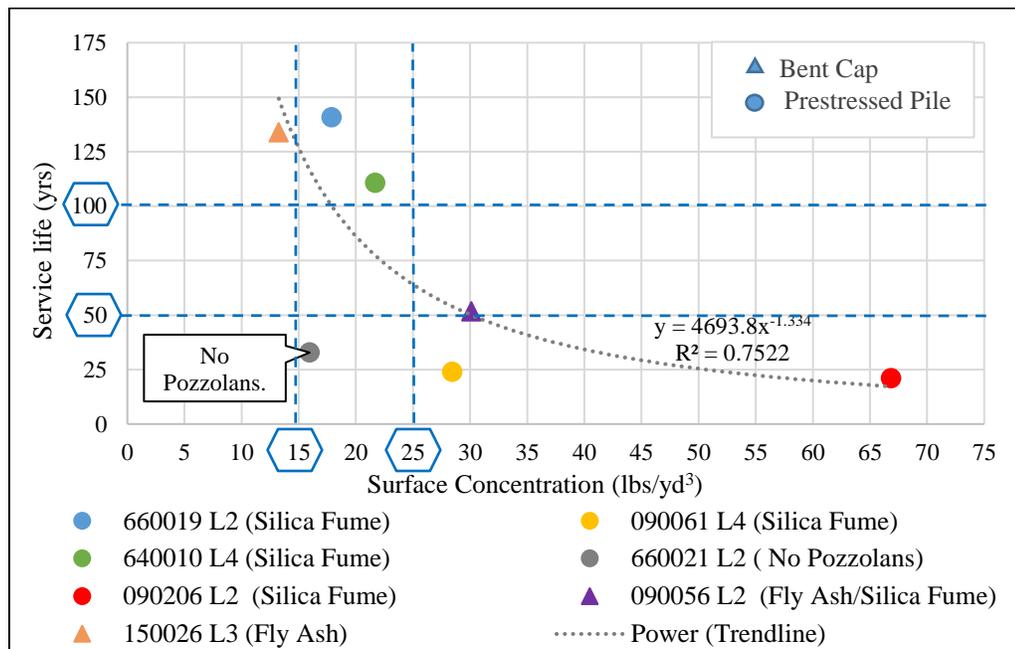


Figure 5.1: Relationship between maintenance free service life and surface concentration

5.2 Distance from open water (Tidal Zone)

The distance of structures from the coastline significantly impacts the surface chloride concentration of locations within the tidal zone. Bridges that were further from open water had a much lower surface chloride concentration than those closer to open water. This relationship can be seen in Figure 5.2. Structure number 090206 was an outlier due to its very high chloride surface chloride concentration. However, the cause of this seemed to again be related to the presence of heavy fouling by marine life and honeycombing in the concrete.

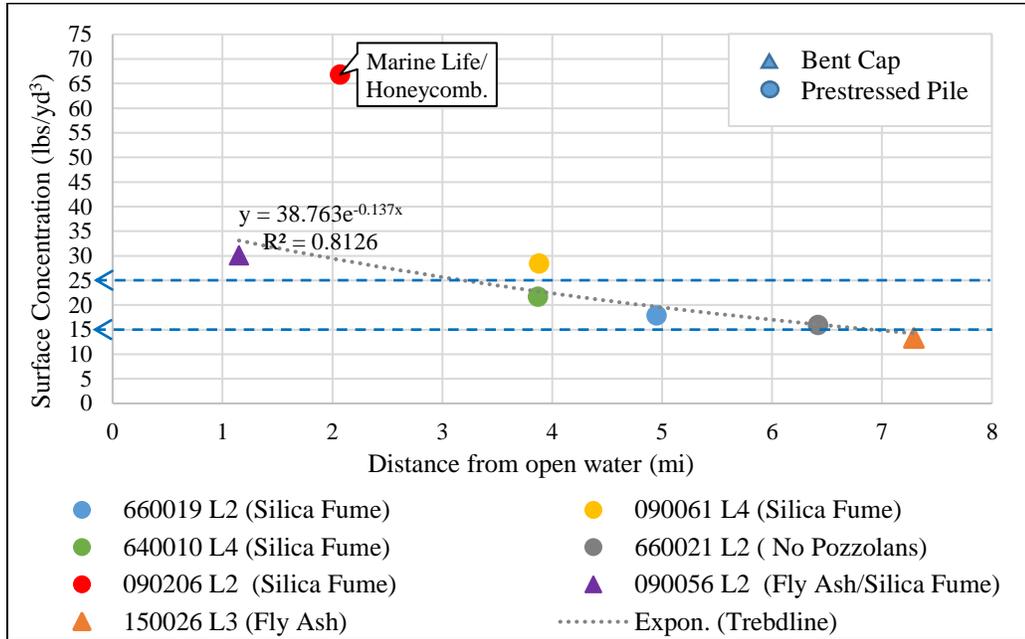


Figure 5.2: Relationship between surface concentration and distance from open water

The data also show a strong relationship between the distance from open water (proximity to the coastline) and service life, which can be seen in Figure 5.3. The expected service life increases with an increase in distance from open water. Two bridges with outlying conditions were excluded from the relationship. Structure number 660021 did not follow the trend because its concrete did not contain pozzolans like the other structures, and its diffusion coefficient ($0.169 \text{ in}^2/\text{yr.}$) was substantially higher than those typical of bridges designed in accordance with the provisions included in the corrosion policy. Structure number 090061 was an outlier to this relationship. Although it contained silica fume, this structure's diffusion coefficient was exceptionally high.

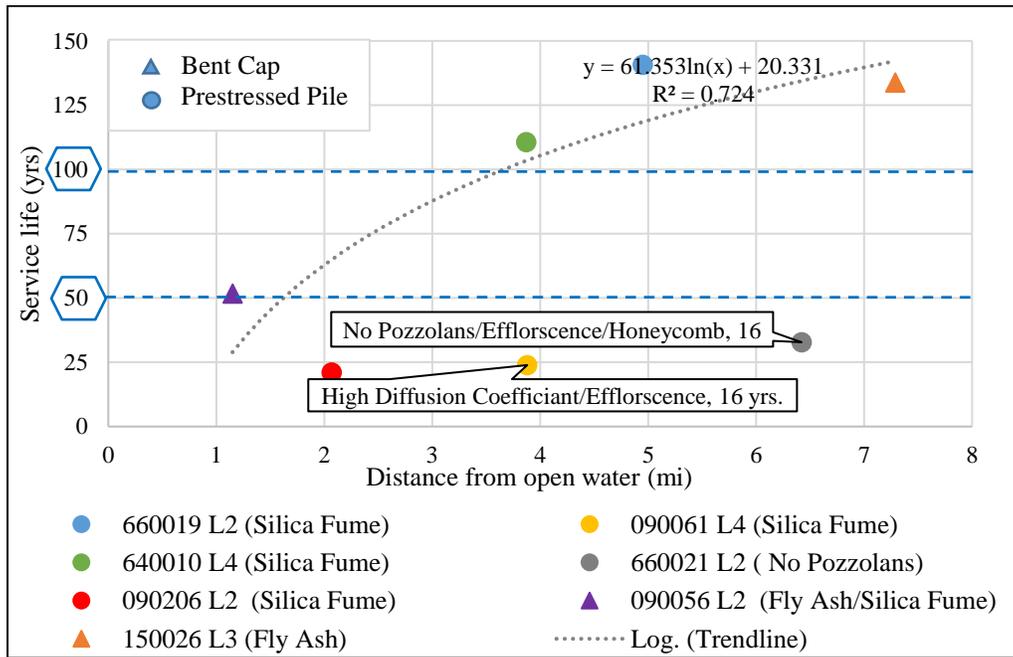


Figure 5.3: Minimum service life vs. distance from open water (Tidal Zone)

5.3 2-in Chloride Concentration vs. Avg. Corrosion Rate

Figure 5.4 shows a strong correlation between the chloride concentration at a two-inch depth and the rate of ongoing corrosion. Low corrosion rates were associated with chloride concentration below the threshold of approximately 1.4 lb/yd³. In previously published information about corrosion modeling, NCDOT has utilized a chloride threshold for non-carbonated concrete at a steel depth (2-in) of 1.4 lb/yd³ for concrete not containing a calcium nitrite corrosion inhibiting (CNI) admixture and 9 lbs/yd³ for concrete mixtures with 3 gal/yd³ of calcium nitrite corrosion-inhibiting admixture (Rochelle 2000). Each of the concrete elements that were tested contained corrosion-inhibiting admixture dosed at rates between 3-3.5 gal/yd³, yet ongoing corrosion was detected in several structures with a chloride concentration above 1.4 lb/yd³ and less than 9.0 lb/yd³. This finding indicates that the corrosion inhibiting admixture dosed at 3-3.5 gal/yd³ may not be providing adequate protection to bridge components with heavy chloride loading, such as those in the tidal zone.

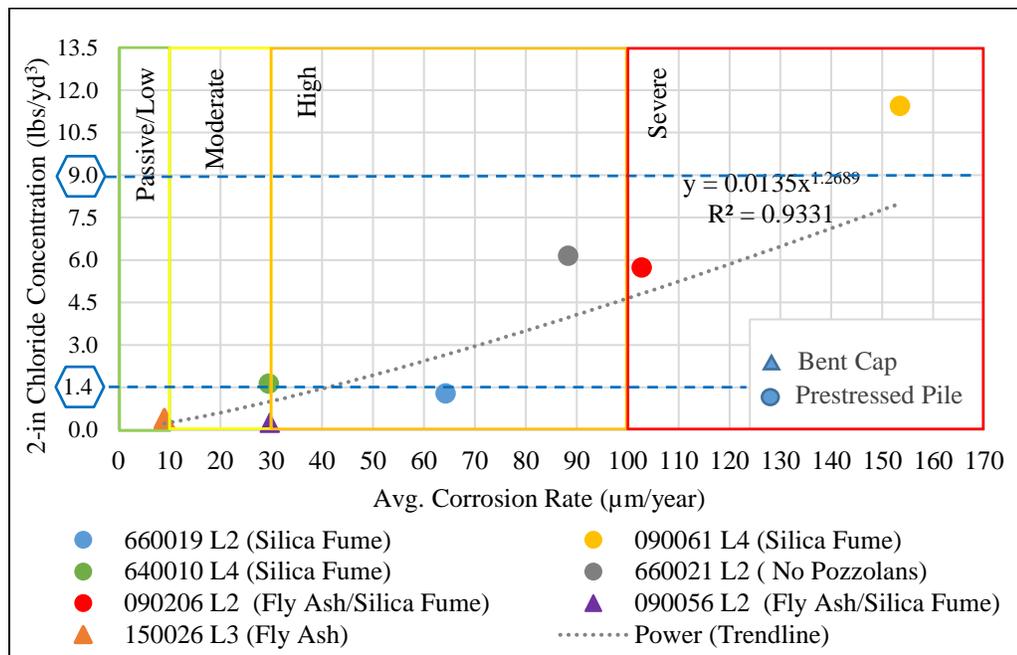


Figure 5.4: Relationship between average corrosion rate and two-inch chloride concentration

Studies on CNI have found that calcium nitrite increases the chloride concentration threshold that depassivates steel reinforcing, and that the strength of this effect is strongly linked to the dosage amount (Ann, 2007). The concentration of CNI around the steel is also related to the rate of corrosion once it begins. However, some of the early research that established commonly used dosage rates was based on test methods that have since been abandoned due to occasionally misleading results. Berke and Hicks (2004) proposed the protective CNI dosage rates summarized in Table 5.1, which were based on direct observations of corrosion from field samples of bridge decks. Sufficient protection is associated with the ratio of chloride to nitrite (Cl:NO₂) at the depth of the steel being less than the values in the rightmost column of Table 5.1. The FHWA has also conducted research (with different methods) that established this protective ratio as between 0.9 and 1.0 (Paul and Clemena, 1998). The notion of a NO₂:Cl ratio of between 0.34 to 0.66 being associated with the onset of corrosion was also described by Ann et al. (2006). Although the ratio is presented as the inverse of other researchers' notation, the upper end of this range (0.66) is similar to the Cl:NO₂ ratios protective ratio proposed by Berke and Hicks in Table 5.1. There is wide variation in the ratios that have been proposed by various researchers.

The Cl:NO₂ ratios of bridges in the NC study are summarized in Figure 5.5 and compared with the rate of ongoing corrosion. The Cl:NO₂ ratio is based on the prescribed CNI dosage rates (3 or 3.5 gal/yd³, as shown on accepted mix specification sheets) and the 2" depth chloride concentrations measured in the field. The average corrosion rates were also measured in the field as part of the experimental phase of this study. There is a clear relationship between the Cl:NO₂ ratio and the average amount of ongoing corrosion. Giatec, the corrosion measurement device manufacturer, proposes that corrosion rates in excess of 10-30 μm/year can be considered as significantly high (below 10 μm/year is considered passive). Therefore, from the data shown in Figure 5.5, it seems that a lower ratio of Cl to NO₂ would be required to delay the onset and progress of corrosion in NC bridges. Berke et al (2004) suggests that low w/c ratio, low permeability concretes

containing mineral admixtures may require approximately 5 gal/yd³ to achieve maintenance free service life of 100 years. The current documentation from the manufacturers of CNI suggest that up to 6 gal/yd³ may be required in order to achieve corrosion protection in the most severe environments. These dosages are generally 60%-100% greater amount than are currently required in the NCDOT corrosion specification.

Table 5.1: Protective CNI dosages (Berke and Hicks 2004)

Calcium Nitrite [l/m ³ , (gal/yd ³)]	Chloride Concentration [kg/m ³ , (lb/yd ³)]	Cl/NO ₂
10 (2)	3.6 (6)	1.3
15 (3)	5.9 (9.9)	1.5
20 (4)	7.7 (13)	1.4
25 (5)	8.9 (15)	1.3
30 (6)	9.5 (16)	1.2

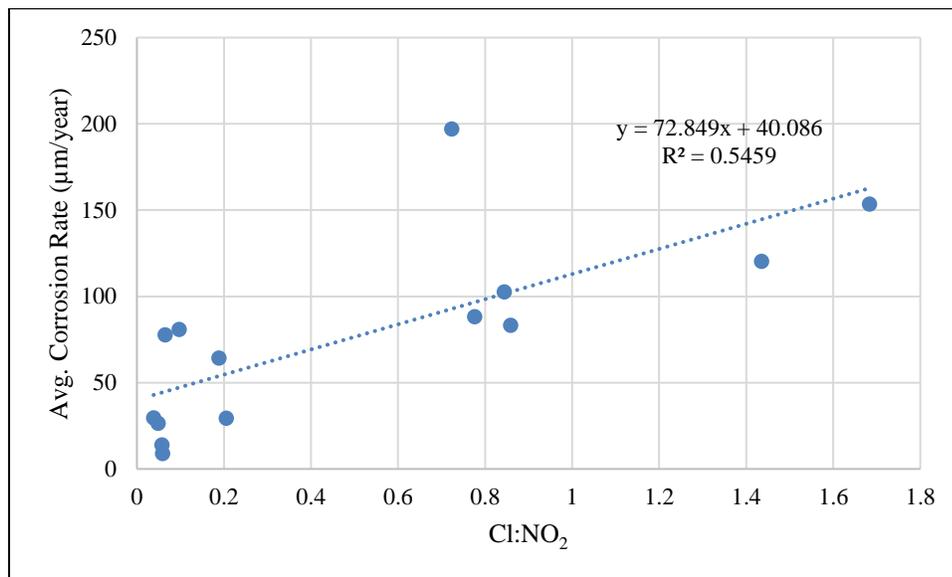


Figure 5.5: Relationship of corrosion rates and Cl:NO₂ ratios for NC bridges sampled during study

5.4 Concrete Diffusion Coefficient and Resistivity

The diffusion coefficient describes the permeability of concrete to chloride ions. It is used in models to estimate corrosion-related service life for bridges that are exposed to chlorides. As was described in Chapter 3, the diffusion coefficient was estimated by collecting samples of concrete from the field and measuring the existing chloride concentration at various depths within the concrete. A second, indirect measurement of permeability and general concrete quality is resistivity. Although resistance to electron flow is a surrogate for permeability to chloride ions, there is a strong relationship between the two. In the following plots, these quantities are related to each other and to ongoing corrosion that was measured in-situ in the structures. Measured values for corrosion rate, surface resistivity, and diffusion coefficient were assigned to a color-coded classification system based upon published information. This classification system is displayed in Table 5.1 (Erdogdu, et al. 2004, GiatecScientific Inc. 2020).

In Figure 5.5, the diffusion coefficient is related to the predicted service life of the bridges in the study. With strong correlation, higher diffusion coefficients were related to shorter service life. Bridges with a predicted service life greater than 100 years were associated with diffusion coefficients of less than 0.05 in²/year. These concrete mixtures were achieved by using silica fume or fly ash in the mixtures. They were also achieved in cast-in-place bent cap mixtures (e.g. in structure 150026).

Table 5.1: Interpretation of corrosion rate and surface resistivity measurements

Corrosion Rate ($\mu\text{m}/\text{yr}$)	Classification	Surface Resistivity ($\text{k}\Omega\cdot\text{cm}$)	Classification	Diffusion Coefficient (in^2/yr)	Classification
<10	Passive/Low	>100	Very High	<0.3	Low
10-30	Moderate	50-100	High	0.3-0.6	Moderate
30-100	High	10-50	Moderate	>0.6	High
>100	Severe	<10	Low		

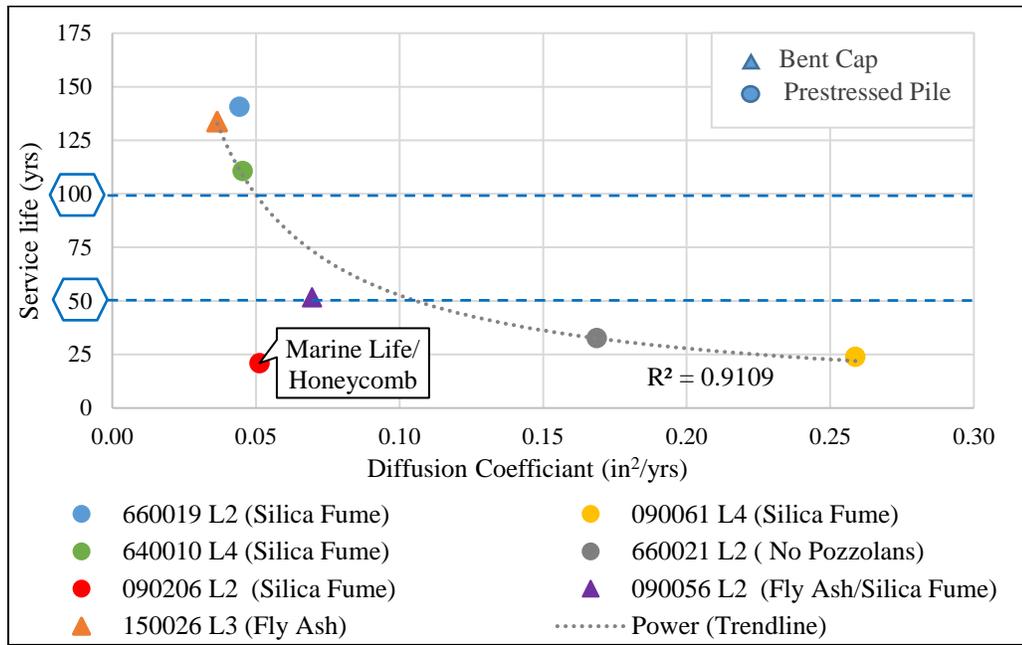


Figure 5.5: Relationship between the diffusion coefficient and predicted service life

As was previously described in Figure 5.4, the rate of ongoing corrosion was strongly related to the chloride concentration at the two-inch depth within the concrete. This depth is typically close to the first layer of steel within reinforced concrete components. High levels of chlorides were found at the two-inch depth within concrete components with low surface resistivity. This relationship is shown in Figure 5.6. Figure 5.7 similarly relates the rate of active corrosion with the surface resistivity measured in the field. Low corrosion rates were associated with high surface resistivity and high corrosion rates were associated with low surface resistivity. Not only is the concrete resistivity an indication of low permeability to chloride ions, it also indicates that the progress of corrosion is impeded once chlorides build up near the steel reinforcement. Although this correlation is strong, the values of surface resistivity measured in the field are not standardized to methods that would be used in the lab to measure resistivity. Due to the field conditions, the concrete was not tested at a standard temperature or moisture content, and because elements had spent years in service, was also heavily contaminated with chlorides. Therefore, although these results are a good justification to incorporate surface resistivity into the corrosion polity specification for concrete, the appropriate threshold may not be determined from this data.

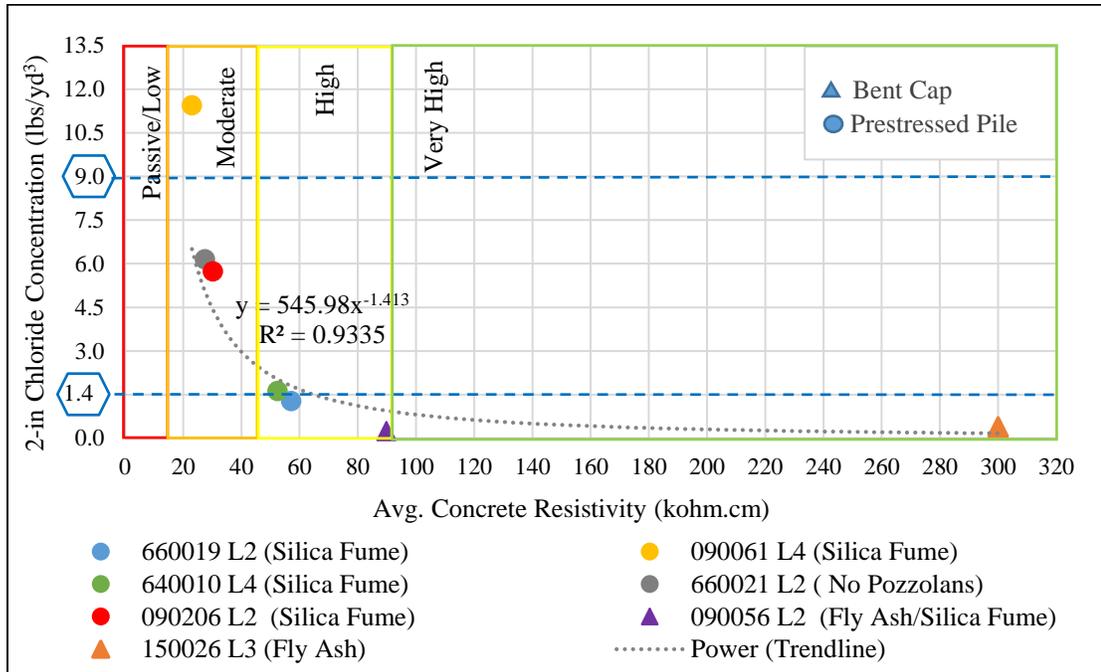


Figure 5.6: Relationship between concrete surface resistivity and two-inch depth chloride concentration

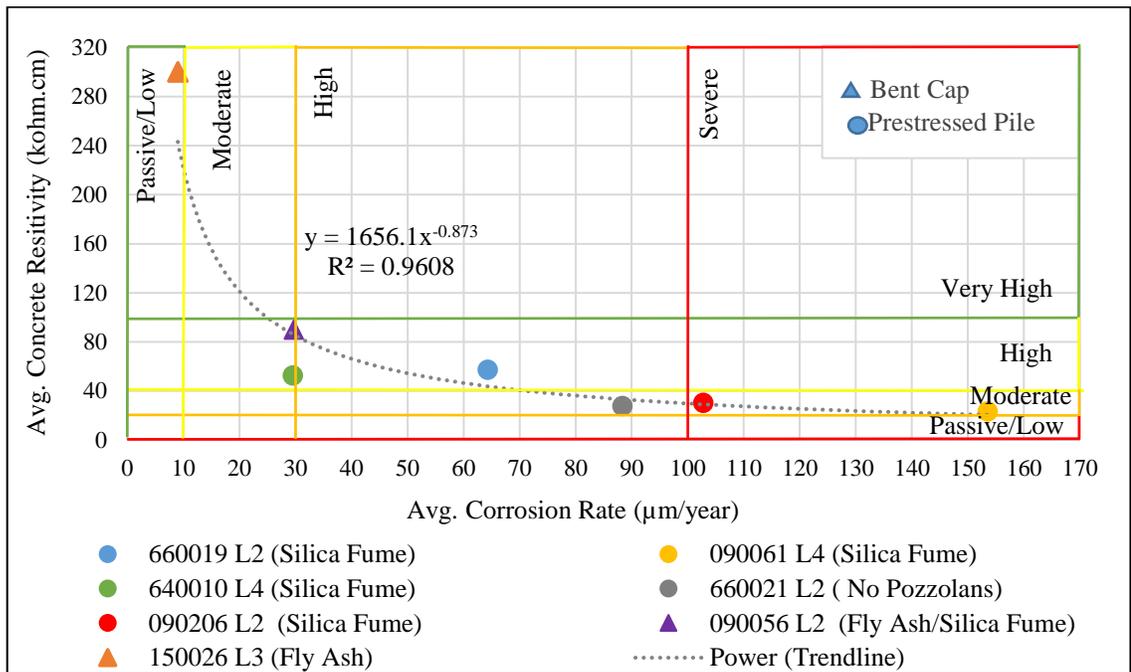


Figure 5.7: Relationship between average concrete resistivity and average corrosion rate

DEFECT MAPPING AND DETERIORATION MODELING

In order to perform a broader survey of bridge conditions and deterioration within the corrosive zones, defect mapping and deterioration modeling were used to supplement the field visits that were made by the team. A query of NCDOT maintenance records from 2016 was used to determine whether there are relationships between proximity to the coast (i.e. within corrosive zones) and corrosion related damages such as delamination and spalling, concrete cracking around reinforcement, and efflorescence. The structures included were limited to bridges near the North Carolina coast that are at waterway crossings and have elements located directly in the water.

6.1 Creating the Dataset

The dataset for this analysis was generated by querying a database of NCDOT bridge maintenance records from 2016 that had been prepared for a previous research project for the NCDOT (Lukavsky 2019, Cavalline et al. 2020) as well as additional files provided by the current project's steering committee. The bridges considered in this analysis were divided into three groups based on their proximity to the coast. These groups included bridges within the highly corrosive and corrosive zones as defined by the NCDOT. Additionally, a group of structures located west of the corrosive zone boundary in non-corrosive areas was included for comparison. Structures were also segregated by whether they were constructed before or after the corrosion policy was enacted. The maintenance records were filtered to exclude structures that were not reinforced concrete, or if the reported defects were not likely caused by corrosion. The final data set of bridges included 229 structures with delamination/spalling, cracking (reinforced concrete and other), or efflorescence/rust staining, as shown in Table 6.1. The number of structures in each zone for this analysis was low because only structures having defects reported in the year 2016 were included in the study.

Table 6.1: Breakdown of structures included for defect mapping.

Location	Number of Structures Included	
	Pre-Policy	Post-Policy
Highly Corrosive Zone	19	15
Corrosive Zone	6	17
Outside Corrosive Zone	31	141
Total	56	173

6.2 Defect Mapping Results

The summary results presented in this section (Tables 6.2 through 6.4) are conveyed using two different measures. The first measure is the number of structures affected in each zone. The second measure is the average condition state that is included with each report of a specific defect. When a defect is reported, it is accompanied by a condition state which ranges from 1 to 4. Condition states are included to aid in defining the severity of a defect. The four states are generally described as good (1), fair (2), poor (3), or severe (4) condition (Ryan et al. 2012). The reports do not speculate as to the cause of damages or deterioration that is discovered during inspections. Therefore, while these defects are very commonly initiated by corrosion, there was no additional verification of their cause.

Table 6.2: Summary of delamination/spall reports

Corrosive Zone	Percentage of Structures Affected		Average Condition State	
	Pre-Policy	Post-Policy	Pre-Policy	Post-Policy
Highly Corrosive Zone	63%	20%	2.50	2.14
Corrosive Zone	67%	53%	2.59	2.11
Outside Corrosive Zone	61%	25%	2.37	2.44

Table 6.3: Summary of efflorescence/rust staining reports

Corrosive Zone	Percentage of Structures Affected		Average Condition State	
	Pre-Policy	Post-Policy	Pre-Policy	Post-Policy
Highly Corrosive Zone	42%	53%	2.28	2.53
Corrosive Zone	33%	59%	2.40	2.59
Outside Corrosive Zone	29%	55%	2.29	2.47

Table 6.4: Summary of cracking (RC and Other) reports.

Corrosive Zone	Percentage of Structures Affected		Average Condition State	
	Pre-Policy	Post-Policy	Pre-Policy	Post-Policy
Highly Corrosive Zone	58%	80%	2.22	2.15
Corrosive Zone	67%	71%	2.17	2.05
Outside Corrosive Zone	71%	78%	2.25	2.12

6.3 Defect Mapping Findings

The percentage of structures with reported delamination or spalls in 2016 was significantly lower (reduced by over a third) for structures in the highly corrosive zone that were built after the enactment of the current NCDOT corrosion policy. There was also a decrease of structures affected by delamination and spalls in the corrosive zone. However, it is not as significant of a decrease as in the highly corrosive zone. This could be due to the differing and/or lack of corrosion-related construction specifications in the corrosive zone.

The highly corrosive zone had a slightly greater percentage of structures affected by delamination or spalls compared with the bridges outside the corrosive zone prior to implementation of the corrosion policy. Following the implementation of the corrosion policy, structures in the highly corrosive zone had 5% fewer structures affected than outside the corrosive zone. The average condition state of bridges within the highly corrosive zone and corrosive zone is generally lower for bridges constructed under the current policy. This indicates that the defects reported are typically less severe on structures constructed under the corrosion policy.

The percentage of structures affected by efflorescence or rust staining increased in all three zones from pre-policy to post-policy. Although this is the opposite of the trend seen in Table 6.2 with delamination and spalling reports, the increase in reports between pre-policy and post-policy in the highly corrosive zone (11% increase) is significantly lower than the increase in the corrosive zone (26% increase) and outside corrosive zone (26% increase). Looking at only the post-policy percentages, the highly corrosive zone shows a lower percentage of structures affected by efflorescence and staining than bridges in the corrosive zone and bridges outside the corrosive zone. The average condition state of the reported efflorescence or rust staining increased in all three zones from pre-policy to post-policy. Although the average severity (condition state) of this defect increased for post-policy structures, the distribution between the highly corrosive, corrosive, and outside corrosive zone remained relatively the same when compared to pre-policy structures.

The percentage of structures exhibiting concrete cracking was higher in post-policy bridges than pre-policy bridges. The prevalence of cracking within the highly corrosive zone and the corrosive zone was lower than outside the corrosive zone before the corrosion policy was implemented. Following the implementation of the policy, the corrosive zone continued to contain a lower proportion of structures affected by cracking than outside the corrosive zone. The highly corrosive zone also featured more cracking in concrete structures than outside of the corrosive zone. The average condition state of the locations with cracking decreased in all three zones from pre-policy to post-policy, which could indicate that extent of this defect was typically less severe following the enactment of the policy. This was the only defect in the analysis that showed a decreased average condition state even though the percentage of structures affected increased following the implementation of the policy.

There were not large differences in the condition state or the prevalence of corrosion related defects reported within the corrosive zones compared to outside the corrosive zones. Because the structures in the pre-policy category were in service longer than those in the post-policy category and because the defects considered for both categories were from 2016, it is not possible to draw strong conclusions about whether structures constructed post-policy are more resistant to

corrosion damage than those constructed pre-policy. During this period of time, the standards for describing defect severity could also have changed with departmental inspection policies. While this study included the condition state as a portion of the analysis, the actual severity of defects could not be adequately captured with the information available (i.e. four level measure of severity). It is possible that, although post-policy structures appeared to be experiencing a larger percentage of defects, the defects are now typically less severe, or vice versa. With the records available for this analysis, it was not possible to determine this with a high level of certainty.

6.4 Deterioration Modeling

Condition rating records from the National Bridge Inventory (NBI) were used to determine the deterioration rate of a broader set of bridges than those that could be visited in person during this study. The deterioration models that were prepared allowed comparisons of bridge component type (substructure versus superstructure bridge component), geographic location in relation to the NCDOT corrosive zone map (highly corrosive zone, corrosive zone, or neither), and structure age (built before or after current NCDOT corrosion policy). The results of this analysis were used as evidence of whether the current NCDOT corrosion policy has extended the corrosion related maintenance-free service lives of structures located near the coast.

6.4.1 Methodology of Deterioration Modeling.

The design policy of the current corrosive sites was implemented in 2003. One of the anticipated outcomes of this analysis was to highlight any noticeable differences or trends in deterioration between bridges constructed before and after the policy was put into place. To ensure there was an equivalent representation of bridges constructed before the policy was enacted and bridges constructed after the policy was enacted, a range of 20 years between 1993 and 2012 was chosen to include 10 years before the policy and 10 years after. These ranges are referred to as pre-policy and post-policy, respectively. The dataset for this analysis was created using records sourced from the National Bridge Inventory (NBI) for North Carolina ranging from 1992 to 2018. After downloading the NBI characteristic and inspection data for all available years, the information necessary to complete the deterioration modeling and statistical analyses was compiled into a single Excel spreadsheet. This spreadsheet included the structure number, latitude and longitude, year built, substructure condition rating, superstructure condition rating, and crossing type. The condition ratings for both the substructure and superstructure follow the general guidelines below (Ryan et al. 2012):

<u>Code</u>	<u>Description</u>
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION – no problems noted.
7	GOOD CONDITION – some minor problems.
6	SATISFACTORY CONDITION – structural elements show some minor deterioration.
5	FAIR CONDITION – all primary structural elements are sound but may have minor section loss, cracking, spalling, or scour.
4	POOR CONDITION – advanced section loss, deterioration, spalling, or scour.
3	SERIOUS CONDITION – loss of section, deterioration, spalling, or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	CRITICAL CONDITION – advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	“IMMINENT” FAILURE CONDITION – major deterioration or section loss present in critical structural components, or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put bridge back in light service.
0	FAILED CONDITION – out of service; beyond corrective action.

The full dataset compiled from the NBI files consisted of 18,377 bridges and culverts across North Carolina. To limit the records contained in the spreadsheet to those only associated with structures related to this study of coastal bridges, several filters were applied including proximity to coast, structure type, structure crossing type, year built, and accuracy and length of available records. With the previously described filters applied, the final breakdown of structures included in the analysis is shown below in Table 6.5.

Table 6.5: Breakdown of structures included for deterioration modeling.

Location	Number of Structures Included	
	Pre-Policy 1993 - 2002	Post-Policy 2003 - 2012
Highly Corrosive Zone (Zone 1)	26	28
Corrosive Zone (Zone 2)	21	18
Outside Corrosive Zone (Zone 3)	68	67
Total	115	113

A map showing the geographic distribution of the structures described in Table 6.5 is shown in Figure 6.1 along with the boundary lines of the corrosive sites. The structures marked by a red pinpoint are in the highly corrosive zone, the structures denoted by a yellow pinpoint are in the corrosive zone, and the structures denoted by a green pinpoint are not in either zone.



Figure 6.1: Map of bridges included in analysis

Structures in NCDOT Divisions 1, 2 and 3 that are near the coast but outside the corrosive zones were also included in this analysis. These bridges outside the corrosive zones were used as a comparative or control group. Also, if any of them exhibited greater prevalence of corrosive damage, then that would indicate that the zone boundaries may need to be updated. In the figures and tables below, these zones will be abbreviated as 1- highly corrosive, 2- corrosive, and 3-outside the corrosive zone.

6.4.2 Deterioration Modeling.

Once data from the NBI was compiled, a script in the application MATLAB was used to count the number of years the substructure and superstructure of each bridge spent in each condition rating. Following the count, the

performance of each group (as the duration spent in each condition rating between 9 and 5), and the confidence interval of each group was determined using the following steps:

1. Calculate the average time spent in a condition rating
2. Calculate the standard deviation (std.dev) of the time spent in a condition rating
3. Determine the number of records (n)
4. Calculate the standard error (std.err) using the formula below

$$\text{Standard Error} = \frac{\text{std. dev}}{\sqrt{n}}$$

5. Use the built-in Excel function T.INV.2T (probability, deg_freedom) where probability is set to 0.25 (for a 75% confidence interval) and deg_freedom is equal to n minus one
6. Calculate the 75% confidence interval in Excel by using the formula below

$$75\% \text{ Confidence Interval} = \text{std. err} \times T.INV.2T(0.25, n - 1)$$

Most of the bridges considered in this analysis had not reached condition ratings below 7. Either they have been rehabilitated and/or repaired during their lifecycle, or they have not experienced sufficient exposure to reach high levels of deterioration. Therefore, the confidence in the time spent in lower condition ratings is less than higher ratings. Table 6.6 presents the n-value for each element (substructure versus superstructure), age (pre-policy versus post-policy), and condition rating (9 through 5) of the bridges in the dataset. The width of confidence intervals shown in the subsequent summary output are influenced by the magnitude of the n-value; a larger n-value results in a more confident statement whereas a smaller n-value results in a less confident statement (i.e. a small range of potential values versus a large range of potential values, respectively).

Table 6.6: N-value for observations of structures in each condition rating.

Element	Policy / Zone	n-Value				
		CR 9	CR 8	CR 7	CR 6	CR 5
Substructure	Pre / 1	4	19	20	8	0
	Post / 1	3	19	26	1	2
	Pre / 2	4	16	13	1	0
	Post / 2	3	11	18	2	0
	Pre / 3	11	59	33	9	0
	Post / 3	14	40	67	6	2
Superstructure	Pre / 1	6	20	13	5	0
	Post / 1	3	19	25	5	1
	Pre / 2	5	16	8	1	0
	Post / 2	3	11	18	0	0
	Pre / 3	14	57	20	4	0
	Post / 3	13	40	64	2	1

Figures 6.2 and 6.3 show the deterioration rate for substructures of bridges built before (6.2) and after (6.3) the current corrosion policy. Prior to the current corrosion policy, substructures appear to be performing similarly regardless of their proximity to the coast, with substructures in the highly corrosive zone deteriorating at a slightly faster rate than within the corrosive zone and outside the corrosive zone (see Figure 6.2). This implies that prior to implementation of the corrosion policy, the substructures performed roughly equally whether they were in a corrosive environment or not (i.e. with no defined corrosive boundary or difference in specifications).

After implementation of the corrosion policy, substructures located within the highly corrosive zone and the corrosive zone appear to be performing equally or slightly better than those located west of both corrosive boundary lines until reaching condition rating 6. After condition rating 6, bridge substructures in the highly corrosive zone appear to outperform substructures located outside of the corrosive zone (see Figure 6.3). This is an indication that the current corrosion policy is having the intended effect of extending the maintenance-free service lives of the substructure elements. This is shown in Figure 6.3 where the deterioration models follow nearly the same trajectory from condition rating 9 to condition rating 6. After condition rating 6, the structures located in the highly corrosive zone and the corrosive zone appear to be performing better (i.e. taking a longer time to deteriorate from condition rating 7 to condition rating 5) than structures located outside the corrosive zones.

The deterioration curves in Figure 6.4 show that substructures constructed pre-policy are performing better than substructures constructed post-policy. However, this may be due to the shorter period of time structures built post-policy spend in condition rating 9. Also, due to the small number of bridges available to establish the deterioration trends in the lower condition ratings, the 95% confidence interval is very wide, as shown with horizontal bars in Figure 6.4. The post-policy substructures appear to spend more time in condition rating 8 than pre-policy substructures by 2.1 years on average.

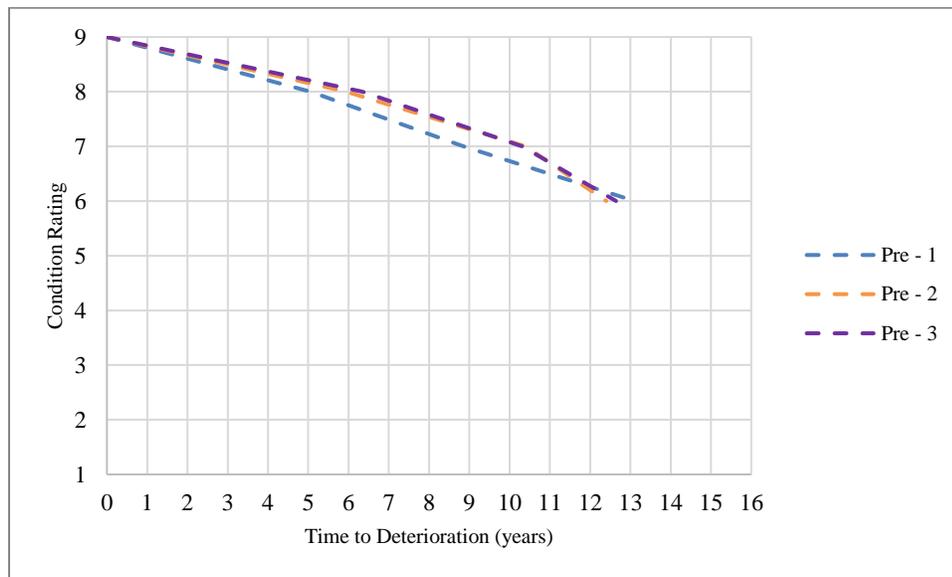


Figure 6.2: Deterioration model comparing pre-policy substructures.
(Zone 1: n = 28. Zone 2: n = 21. Zone 3: n = 68)

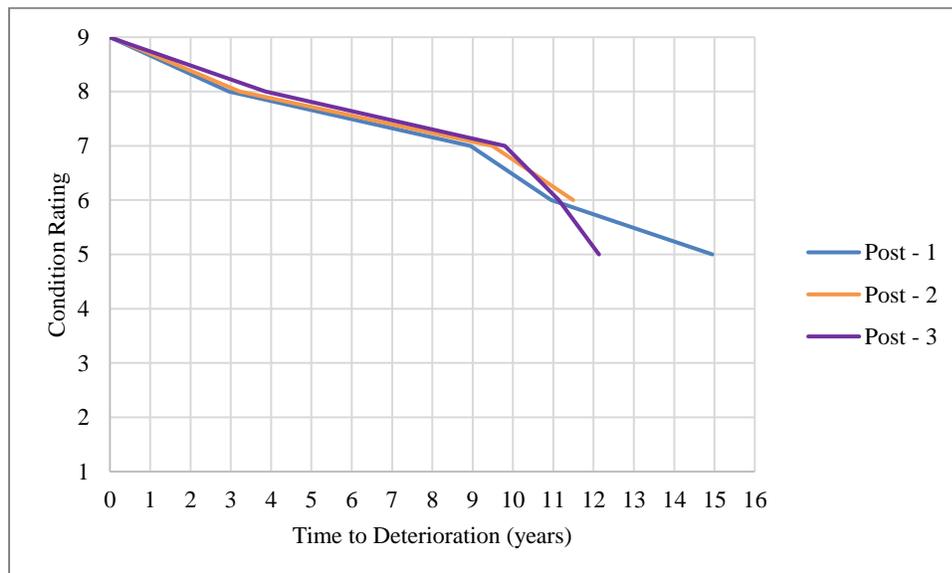


Figure 6.3: deterioration model comparing post-policy substructures.
 (Zone 1: n = 26. Zone 2: n = 18. Zone 3: n = 67)

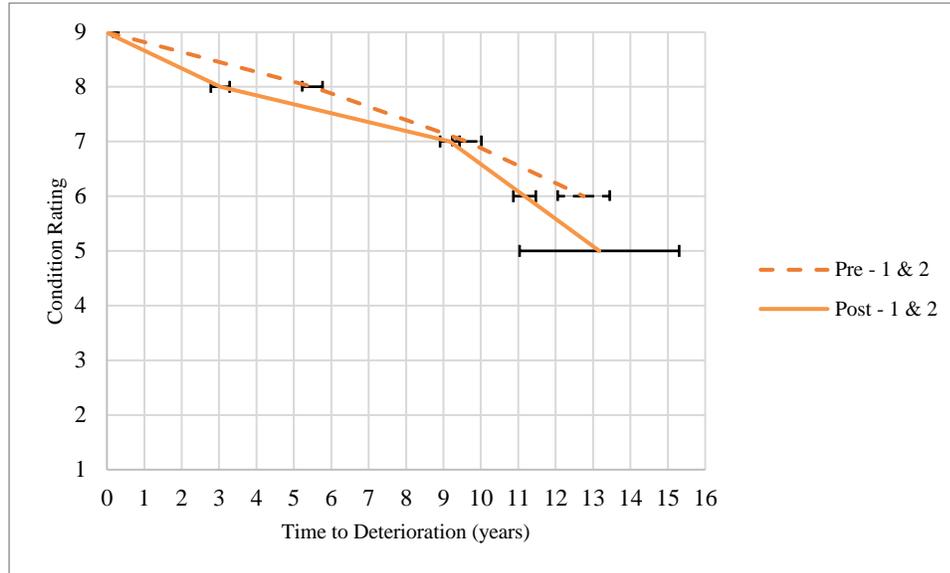


Figure 6.4: Deterioration model comparing pre- and post-policy substructures.
 (Pre-Policy: n = 49. Post-Policy: n = 44)

As is shown in Figure 6.5, superstructures built in the corrosive zone before the current corrosion policy appear to perform equally to those outside of the corrosive zones until reaching condition rating 7. Under the current policy, superstructures in the highly corrosive and corrosive zones appear to be performing equally to those outside the corrosive zone until condition rating 7. After this point, the condition rating of superstructures located within the highly corrosive zone begins to decline at a faster rate than structures located in the corrosive zone and outside corrosive zone (see Figure 6.6).

The superstructures constructed post-policy appear to be deteriorating at a faster rate than the superstructures constructed pre-policy (see Figure 6.7). Similar to the trend shown for the substructures, post-policy superstructures appear to be spending slightly more time in condition rating 8 than pre-policy superstructures by 0.5 years on average. The analysis showed a statistically significant difference of time spent in condition rating 8 (post-policy outperforming pre-policy), with the differences of time spent in condition ratings 7 and 6 being insignificant. This implies that the differences seen on the deterioration models are mostly statistically insignificant (except for condition rating 8) and structures built post-policy are performing very similarly to structures constructed pre-policy.

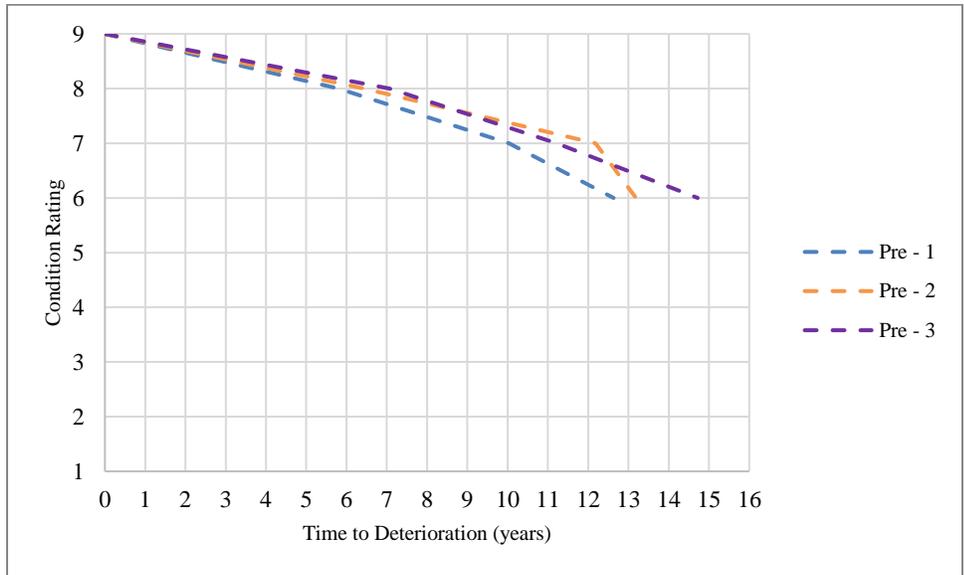


Figure 6.5. Deterioration model comparing pre-policy superstructures.
(Zone 1: n = 28. Zone 2: n = 21. Zone 3: n = 68)

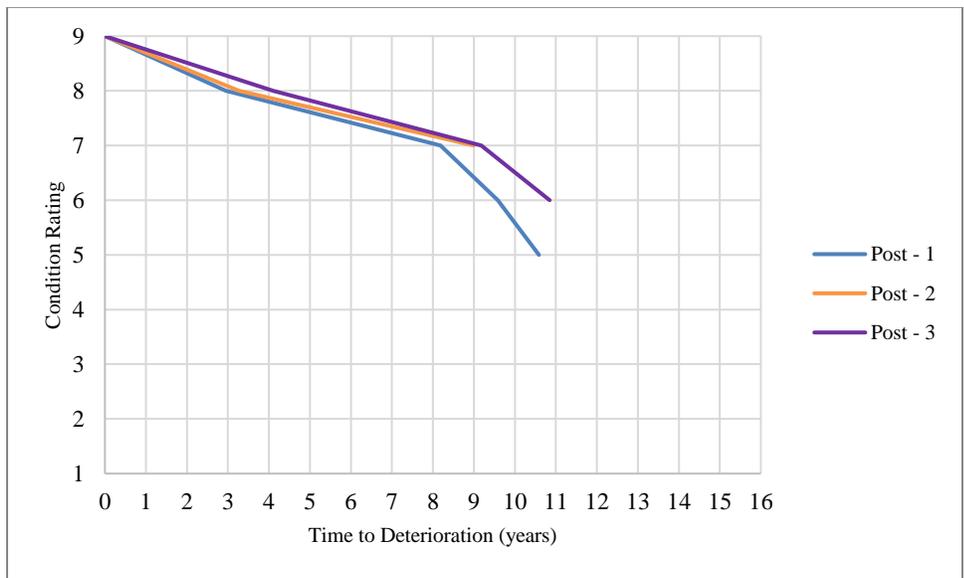


Figure 6.6. Deterioration model comparing post-policy superstructures.
(Zone 1: n = 26. Zone 2: n = 18. Zone 3: n = 67)

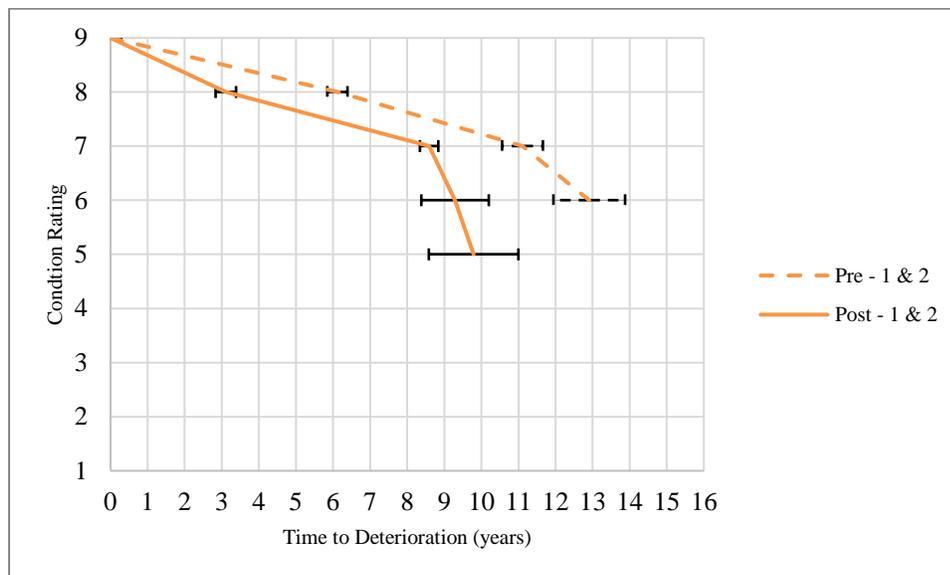


Figure 6.7. Deterioration model comparing pre- and post-policy superstructures. (Pre-Policy: n = 49. Post-Policy: n = 44)

6.5 Statistical Correlation Results

The results presented above were evaluated with a two-sample t-test analysis to indicate the significance of the findings based on the number of structures in the sample. This analysis was completed in the statistical application, Minitab, for substructures in the highly corrosive zone, substructures in the corrosive zone, superstructures in the highly corrosive zone, and superstructures in the corrosive zone. The Minitab analysis showed that pre-policy structures typically spent more time in condition rating 8 than post-policy structures. This was true for substructures and superstructures in both corrosive zones (highly corrosive and corrosive). It was also clear that post-policy substructures in the highly corrosive zone spent more time in condition rating 7 than pre-policy substructures. These observations were the only differences that the two-sample t-test determined to be significant.

The analysis determined that the difference in average time spent in condition rating 9 for both substructures and superstructures in the corrosive zone were not significant. This was also the case for substructures and superstructures in the corrosive zone and superstructures in the highly corrosive zone; the differences seen in condition rating 7 were not significant. Additionally, superstructures in the highly corrosive zone in condition rating 6 were determined to have no significant differences whether they were constructed pre-policy or post-policy. While there were four cases (condition rating 8 for all four scenarios) that showed a statistically significant difference in performance with the pre-policy structures performing better, there were six cases that were determined to have no statistically significant differences between pre-policy and post-policy and one case where the post-policy structures were performing better.

6.6 Summary of Findings

The completion of defect mapping and deterioration modeling have both suggested that the performance of bridges constructed with the provisions of the current corrosion policy has not significantly improved over the performance of bridges constructed prior to the policy. However, both of these analyses were completed using subjective data that was collected by different inspectors over a long period of time. The standards and methods for assessment during bridge inspections changed during this period, as have construction methods, construction conditions (schedule, materials quality, labor experience), and typical bridge designs. Although this analysis has not yielded strong evidence that the corrosion policy has lessened the rate of deterioration for bridges within corrosive areas, it is significant that bridges within the corrosive zones perform similarly to inland bridges. This can be taken as evidence that the policy has “leveled the playing field” for structures exposed to corrosive agents. In some cases, the added concrete quality requirements have caused slightly improved performance of bridges in corrosive zones over bridges outside of corrosive zones. This observation also supports the suitability of the current zone delineations.

7. SUMMARY AND CONCLUSIONS

During this study, field measurements and concrete samples were taken from the piers and pier caps of nine bridges that crossed waterways in the corrosive and highly corrosive zones of the North Carolina coast. Ongoing corrosion was detected in certain elements of all structures that were sampled, particularly areas exposed to tidal waters. The team used the data collected in the field to prepare service life models of the bridges in order to arrive at the conclusions highlighted and summarized below:

7.1 Conclusions from Field Testing

Visual Inspection- Although most bridges did not feature significant visual signs of corrosion, the team identified exposed prestressing strand and large, unfilled honeycombs (Structure number 660091), concrete spalling and exposed steel bar of the bent cap of (Structure number 090056) and frequent examples of consolidation problems, and efflorescence on most structures.

Corrosion Detection (Giatec Device) - Active corrosion at a range of rates was detected in the tidal zone of bridge piers, although the rate of corrosion dropped off quickly at locations outside of the tidal zone. Limited evidence of corrosion was detected or observed on the portions of the bridges that receive only atmospheric contact with chlorides through spray, splashing, and mist.

7.2 Conclusions from Laboratory Testing

Chloride Concentration Measurement - Chloride concentration tests indicated a rapid decrease of chloride contamination between the surface and two inches below the surface on almost all elements. Although the chloride levels were high at the level of the steel (above typical corrosion inducing thresholds), they were low towards the interior of the elements. This is most likely due to the limited exposure most structures experienced because of their relatively young age (less than 25 years in service) and the effect of the policy requirements to add SCMs, which impede ingress of chlorides. The significant contamination of chloride was limited mostly to the portions of the structure that are frequently exposed to coastal waters. Most concrete was found to have diffusion coefficients lower than the typical/target values proposed for 100-year service life by NCDOT (Rochelle 2000). However, most structures located very close to the ocean were found to have more severe exposure than would be anticipated by those models.

Corrosion Inhibitor Detection - In most cases the calcium nitrite corrosion inhibiting admixture concentrations measured from powder samples collected in the field met or exceeded the specified NCDOT minimum addition rate of 3 gal/yd³. This finding confirmed that calcium nitrite is being utilized in the mixtures in approximately the correct dosage rates. However, the dosage rate did not have the expected effect of raising the corrosion initiation threshold of chloride concentration to above 9 lb/yd³. Active corrosion was detected at moderate to high rates in components that contained corrosion inhibiting admixture and had less than 9 lb/yd³ of chloride.

7.3 Conclusions from Service Life Modeling and Deterioration Rate Modeling

Service Life Modeling - Three of the bridges studied had expected service lives greater than 100 years, one just over 50 years, and three less than 50 years. The three bridges with expected service lives of less than 50 years were all characterized by high chloride exposure and loading (i.e., less than 3 miles from the ocean) and problems with concrete quality that increased the diffusion coefficient. These problems included honeycombing, cracking, or excessive fouling by oysters.

Exposure to Chlorides - Service life modeling results indicated that the key factor impacting the service life is the tendency for the concrete structure to be exposed to (or intermittently exposed to) chloride rich waters, typically piers and/or pier caps. The severity of exposure was strongly related to proximity of the structure to the coast. Factors such as fouling and construction defects increased the vulnerability to high rates of chloride ingress. In all

cases where locations experienced infrequent wetting, corrosion was not predicted to be the predominant or significant deterioration mechanism.

Deterioration Modeling - Evaluation of bridge deterioration rates (tracked as condition ratings in the NBI database) and the frequency of various flaws listed in NCDOT bridge inspection reports did not indicate that bridges constructed under the current policy are less susceptible to typical corrosion damage than bridges constructed under previous policies.

Comparison with other States' Policies and Specifications - The NC coastal bridge corrosion policy has similar features to many other states' policies, but is more detailed than most in its specified requirements. Some states utilize site-based chloride sampling of coastal waters to determine the severity of the corrosive environment (rather than ocean proximity zones) and to set design guidelines based on severity. Several states allow the use of more corrosion resistant materials such as galvanized and stainless steel reinforcing.

Expected Service Life of Bridges in Corrosive Areas - The findings of this research project suggest that the corrosion policy is effective delaying the onset of corrosion in concrete components in most cases. It is effective at providing protection to concrete elements that are receiving chloride loading from splashing, spray, or atmospheric deposition. It is also effective at providing protection to bridges in locations where the concentration of chlorides in the brackish water is low. However, many bridges constructed under the current corrosion policy are not on track to have total maintenance-free service lives in excess of 75 or 100 years. Bridges that meet the current specified requirements of the corrosion policy to include fly ash and silica fume, proper concrete cover, and corrosion inhibiting admixture, may have maintenance-free service lives of less than 50 years. Bridges with piers in contact with brackish water within three miles of the coast are especially likely to be vulnerable to corrosion.

8. IMPLEMENTATION AND TECHNOLOGY TRANSFER PLAN

The findings of this study have focused attention on the potential to update the corrosion policy for structures that are constructed in close proximity to the coast. Therefore, there is an opportunity to revise the corrosion policy to address these particular conditions. The durability of concrete components exposed to seawater is a function of concrete quality, depth of concrete cover over the reinforcing steel, and both physical and chemical protection of reinforcing steel. Many combinations of these strategies could effectively achieve corrosion-free service life greater than 50 years and up to 100 years. The following provisions are recommended for inclusion in the corrosive sites specification:

Additional Site Characterization

- The tidal zone is not strictly defined in the SMU Design Manual, however it is referenced in the provision. It is recommended that NCDOT consider adopting a formal definition that identifies the areas of structures that will be exposed to wetting and drying cycles. This will enable the application of enhanced provisions for elements exposed to wetting and drying. These provisions could include increased concrete cover, protective jacketing, corrosion resistant steel, or other approaches.
- Water samples from the site of the structure could be used to establish the level potential chloride exposure and to determine the aggressiveness of individual bridge sites. Revisit the boundaries of the highly corrosive and corrosive zones based upon the results of chloride concentration testing of the coastal waters. Since the evaluation of water samples would add tasks and costs to the work of bridge designers, a map of coastal salinity could also be prepared and provided within the corrosive sites policy in the SMU Design Manual.

Concrete Quality

- Continue to require and encourage use of pozzolans through binary and ternary cementitious material blends, but develop performance standards for these mixtures when used in tidal zones.
- Add surface resistivity performance requirements to concrete mixture qualification processes. Add surface resistivity testing to the quality control protocol for concrete acceptance similar to those described in (Cavalline et al. 2020). Based on the findings of the previous study, a minimum surface resistivity of 16.0 kΩ-cm is recommended for coastal bridges as an initial performance target.
- Based on the findings of NCDOT RP2018-14, “Durable and Sustainable Concrete Through Performance Engineered Concrete Mixtures,” Cavalline et al (2020) recommended the inclusion of a surface resistivity testing specification into the NCDOT Standard Specifications for Roads and Structures. The method in which this specification was suggested for implementation was the same manner in which LADOTD initially implemented surface resistivity testing (LADOTD 2018). The recommendation for revising Section 1000-4C was presented as follows, with recommended changes shown in *italics*:

(C) Strength and Surface Resistivity of Concrete

The compressive strength *and surface resistivity* of the concrete will be considered the average test results of two 6 inch x 12 inch cylinders, or two 4 inch x 8 inch cylinders if the aggregate size is not larger than size 57 or 57M. Make cylinders in accordance with AASHTO T 23 from the concrete delivered to the work. Make cylinders at such frequencies as the Engineer may determine and cure them in accordance with AASHTO T 23 as modified by the Department. Copies of these modified test procedures are available upon request from the Materials and Tests Unit. Testing for compressive strength should be performed in accordance with AASHTO T 22. *Testing for surface resistivity should be performed in accordance with AASHTO T 358.* When the average compressive strength or surface resistivity of the concrete test cylinders is less than the minimum targets specified in Table 1000-1 and the Engineer determines it is within reasonably close conformity with design requirements, these properties will be considered acceptable. *When the Engineer determines average cylinder strength or surface resistivity is below the specification, the in-place concrete will be tested.* Based on these test results, the concrete will either be accepted with no reduction in payment or accepted at a reduced unit price or rejected as set forth in Article 105-3.

The following table (Table X.X) would be added or incorporated into Table 1000-1 with the associated footnote.

Table X.X: Suggested addition to NCDOT specification for roads and structures

Class of Concrete	Minimum surface resistivity at 56 days (kΩ-cm)
AA	15.0*
Pavement	11.0

*A 56 day minimum of 16.0 kΩ-cm can be required at the engineer's discretion for applications where risk of chloride ion penetration is high.

Protection of Reinforcing Steel

- Enhanced provisions for elements exposed to wetting and drying are recommended to ensure the service life goals are met. These provisions could include:
 - o increasing concrete cover to 3" for prestressed components in the tidal zone
 - o utilizing protective jacketing
 - o specifying corrosion resistant steel
 - o other performance-verified approaches

- The findings indicated that the currently required CNI dosage is likely too low to achieve the desired level of protection. The proper dosage is a function of the ratio of chloride concentration to nitrite concentration, and therefore is based on the severity of chloride loading. Other protective features, such as cover over the steel and low concrete permeability, are important to maintain because the CNI alone cannot reliably ensure extended service life. Also, high CNI dosage has negative effects such as accelerated set time and lowered compressive strength. It is recommended that the CNI dosages be increased for coastal bridges with tidal exposure by either: 1) following manufacturers guidelines for establishing appropriate dosage rates based on site severity, or 2) develop mixture designs that achieve suitable rheological and mechanical properties while containing the maximum dose of 6 gal/yd³.

- Reevaluate the effectiveness of CNI corrosion inhibitors at higher dosages after exposure in the field. Other corrosion-defensive measures, such as high-quality concrete and sufficient cover, should augment the protection offered by the CNI.

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APPENDICES FOR FINAL REPORT

**North Carolina Department of Transportation
Research Project No. 2019-22**

**Evaluating Corrosive Sites Policy
for Concrete Bridges
At the North Carolina Coast**

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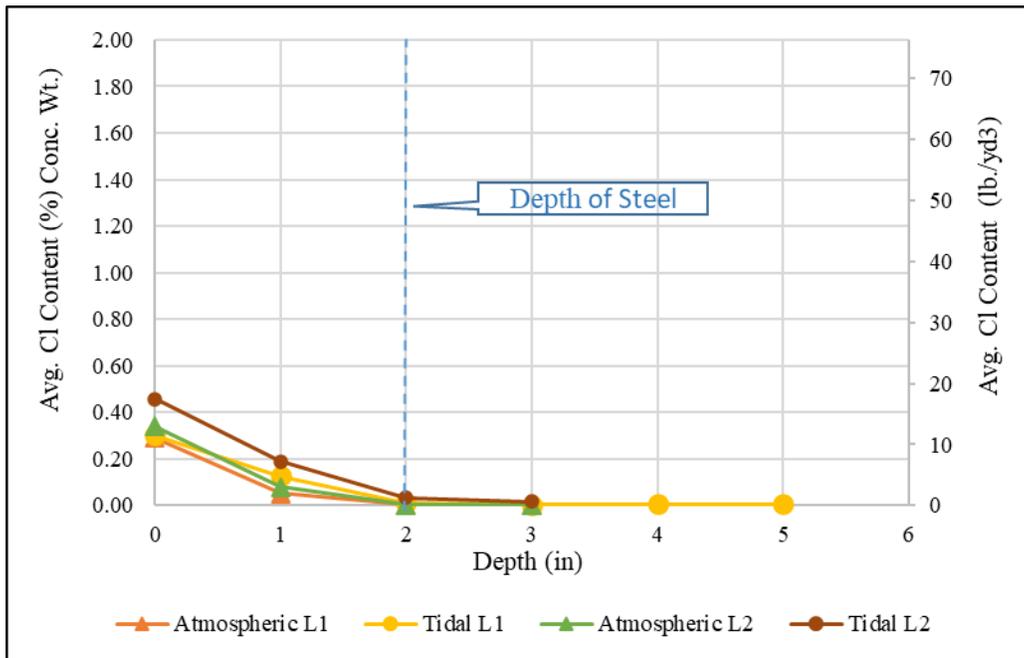
Charlotte, NC 28223

APPENDIX A: Data for Each Bridge

A.1 Structure 660019 Results Summary

A.1.1 RCT Test Results (Chloride Content Profile)

Structure #	Location	Zone	Chloride Content		
			Depth (in)	% Conc. Wt.	Chlorides (lb./CY)
660019	L1	Tidal	0	0.302	11.532
			1	0.125	4.768
			2	0.011	0.436
			3	0.004	0.154
			4	0.005	0.193
			5	0.005	0.183
		Atmospheric	0	0.291	11.096
			1	0.051	1.963
	L2	Tidal	2	0.003	0.109
			3	0.003	0.098
			0	0.457	17.429
			1	0.188	7.186
		Atmospheric	2	0.034	1.281
			3	0.018	0.692
			0	0.341	13.001
			1	0.081	3.096
		2	0.004	0.167	
		3	0.004	0.171	

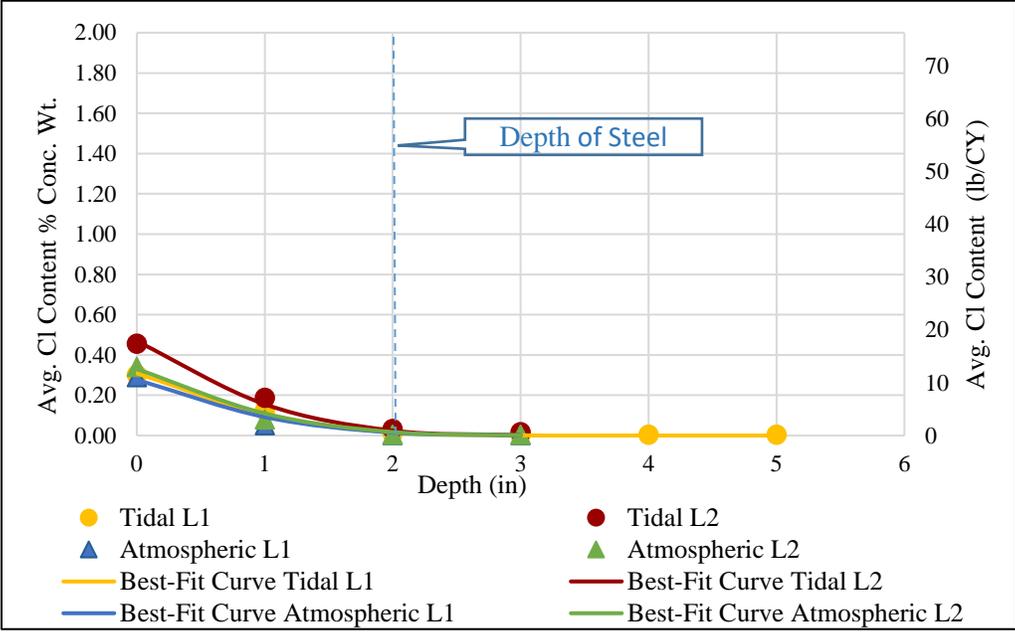


A.1.2 iCOR Test Results (Corrosion Rate & Surface Resistivity)

Structure #	Location	Atmospheric Zone					Tidal Zone				
		Corrosion Rate	Avg. Corrosion Rate	Concrete Resist.	Avg. Concrete Resist.	R ²	Corrosion Rate	Avg. Corrosion Rate	Concrete Resist.	Avg. Concrete Resist.	R ²
660019	L1	8.1	31.2	133.0	115.6	1	48.4	77.7	38.0	32.0	0.92
		8.4		99.0		1	67.6		28.0		0.95
		2.7		130.0		1	191.1		12.0		0.51
		5.0		187.0		1	35.3		37.0		0.98
		4.6		161.0		1	122.0		10.0		0.92
		10.0		190.0		0.95	1.9		67.0		0.8
		40.0		102.0		0.99					
		36.0		95.0		0.99					
		49.0		133.0		0.98					
		90.0		49.0		0.99					
		73.0		52.0		0.94					
		47.0		56.0		0.99					
	L2	2.9	22.9	104.0	99.6	1	12.1	64.3	37.0	57.2	0.98
		5.5		57.0		1	217.8		7.0		0.87
		0.8		129.0		1	2.5		84.0		0.99
		13.0		78.0		1	4.4		115.0		0.99
		3.6		147.0		0.99	84.6		43.0		0.98
		19.0		104.0		1					
		17.0		105.0		1					
		5.0		138.0		0.99					
		10.0		108.0		1					
		39.0		51.0		1					
136.0	75.0	0.95									

A.1.3 Model 2: Iterative Non-linear Least-Squares Regression

Corrosive/Silica Fume	Structure #	Location	Zone	Chloride Content			Age (t)	Model 2: Iterative Non-linear Least-Squares Regression			
				Depth (in) (x)	% Conc. Wt.	Chlorides (lb/CY)		Diffusion Coefficient (in*in/yr) (D)	Surface Concentration (lb/CY) (C _o)	Best-Fit Curve C _(x,t)	Residual Square
660019	L1	Tidal	0	0.302	11.5320	12	0.044	11.805	11.805	0.074	
			1	0.125	4.7684				3.920	0.720	
			2	0.011	0.4364				0.618	0.033	
			3	0.004	0.1541				0.043	0.012	
			4	0.005	0.1931				0.001	0.037	
			5	0.005	0.1831				0.000	0.034	
		Atmospheric	0	0.291	11.0959	12	0.044	10.560	10.560	0.287	
			1	0.051	1.9626				3.507	2.384	
			2	0.003	0.1092				0.553	0.197	
			3	0.003	0.0975				0.038	0.004	
										0.000	
	L2	Tidal	0	0.457	17.4292	12	0.044	17.866	17.866	0.191	
			1	0.188	7.1863				5.933	1.572	
			2	0.034	1.2813				0.936	0.119	
			3	0.018	0.6924				0.065	0.394	
										0.000	
		Atmospheric	0	0.341	13.0012	12	0.044	12.613	12.613	0.151	
			1	0.081	3.0959				4.188	1.193	
2			0.004	0.1674	0.661				0.243		
							0.046	0.016			
								0.000			



A.1.4 Corrosion Modeling Service Life Reports

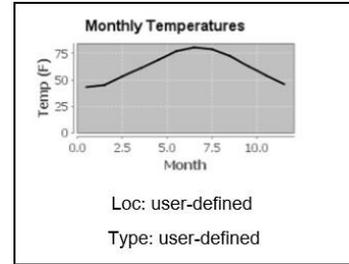
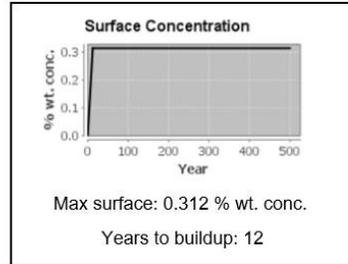
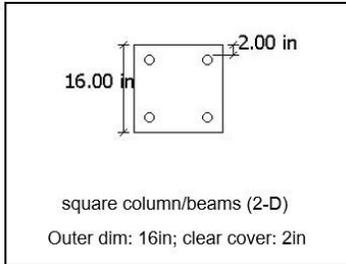
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 660019 L1

Description: Bent Cap/Highly Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 07/30/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.35	Silica Fume (5%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concrete Properties	4.09E-9 in ² /sec	0.2	0.24 % wt. conc.	136.7 yrs	6 yrs	142.7 yrs
Measured Concrete Properties	-> 2.33E-9 in ² /sec	-> 0.26	-> 0.24 % wt. conc.	330.8 yrs	-> 6 yrs	336.8 yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

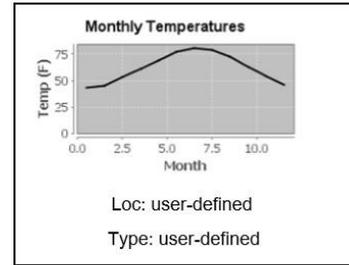
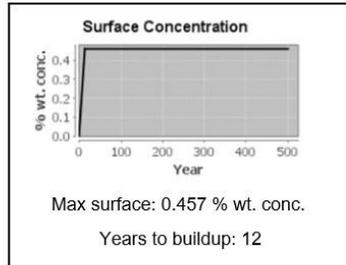
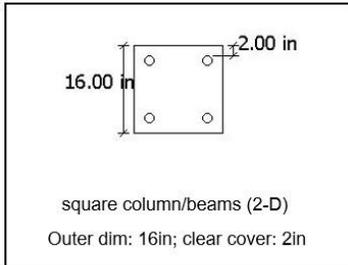
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 660019 L2

Description: Bent Cap/Highly Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 07/30/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concret Properties		0.35	Silica Fume (5%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concret Properties	4.09E-9 in ² /sec	0.2	0.24 % wt. conc.	57.2 yrs	6 yrs	63.2 yrs
Measured Concrete Properties	-> 2.33E-9 in ² /sec	-> 0.26	-> 0.24 % wt. conc.	134.7 yrs	-> 6 yrs	140.7 yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

A.1.5 Materials and Tests Unit Statement of Concrete Mix Design

Form 312U
3-96

North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials

Project	Date Expires 12/31/2075
Mix Design Status Active	Concrete Producer S & G PRESTRESS CONCRETE
County	Plant Location & DOT No. LELAND, NC - 2
Resident Engr.	Contractor
Class of Concrete PRESTRESS	Date Assigned
Mix Design No. 2PVU13CIMSE	Contractor's Signature
Note Mix Design Units (English or Metric) ENGLISH	

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	ARGOS USA	BARRANQUILLA PLANT - (FORMERLY CEMENT)	665 lbs.
Pozzolan	Degussa Rheomac SF 100 silica fume		35 lbs.
Fine Aggregate	MARTIN MARIETTA	ROCKY POINT QUARRY - ROCKY POINT	1244 lbs.
Coarse Aggregate	MARTIN MARIETTA	GARNER QUARRY - GARNER	1700 lbs.
Total Water		WELL	29.4 gals.
Air. Entr. Agent	BASF CONSTRUCTION CHEMICALS, LLC	MASTERAIR AE 90 (AKA MB AE 90)	As recommended
Retarder	BASF CONSTRUCTION CHEMICALS, LLC	POZZOLITH 122R	As recommended
Water Reducer			
Superplasticizer	BASF CONSTRUCTION CHEMICALS, LLC	MASTERGLENIUM 3030 (AKA GLENIUM 3030NS)	As recommended
Corrosion Inhibitor	BASF CONSTRUCTION CHEMICALS, LLC	MASTERLIFE CI 30 (AKA RHEOCRETE CNI)	As recommended

Mix Properties and Specifications

Slump	7.00 in.	Mortar Content	16.60 cu. ft.
Max Water	31.0 gals.	Air Content	5.0 %

Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.65	1.0	NA	2.40
Coarse Aggregate, #67	2.62	0.5	93.5	NA

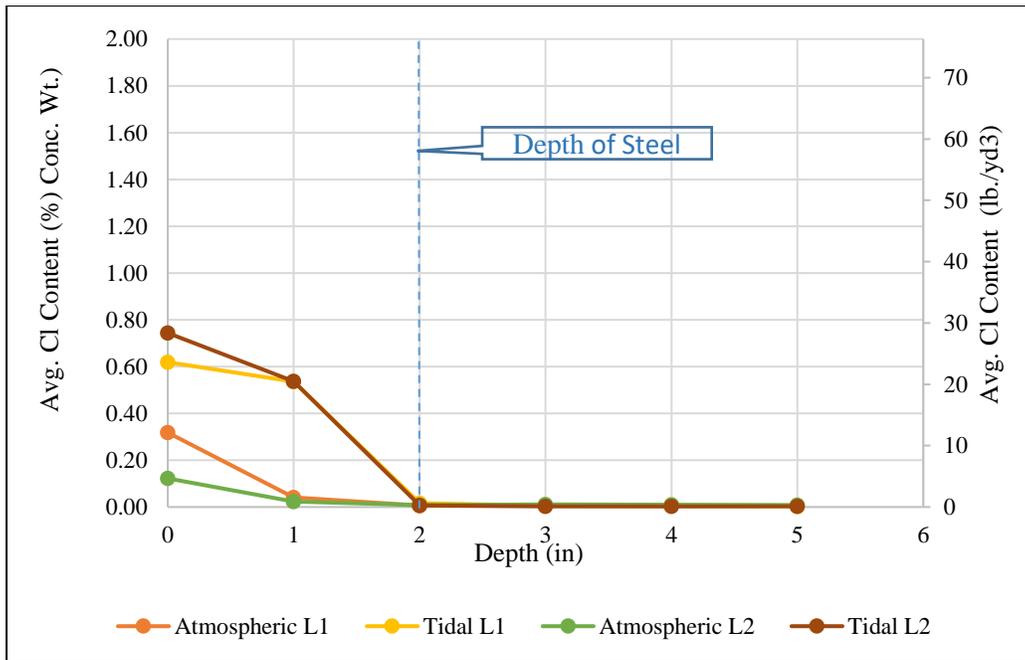
Comment Mix is designed for 6000 psi. Mix contains 3.0 gals. CNI and 35 lbs. Rheomac SF 100 silica fume. (Silica fume quantity is listed on Form 312 under "pozzolan.") Max. water-cement ratio is 0.4.

Cast-in-place concrete shall conform to Section 1000, precast concrete to Section 1077, and prestressed concrete to Section 1078 of the applicable edition of the Standard Specifications for Roads and Structures plus all applicable Special Provisions.

A.2Structure 090056 Results Summary

A.2.1 RCT Test Results (Chloride Content Profile)

Structure #	Location	Zone	Chloride Content		
			Depth (in)	% Conc. Wt.	Chlorides (lb/CY)
090056	L1	Tidal	0	0.619	23.610
			1	0.536	20.450
			2	0.017	0.659
			3	0.005	0.200
			4	0.007	0.269
		5	0.004	0.135	
		Atmospheric	0	0.318	12.133
			1	0.041	1.569
			2	0.006	0.240
			3	0.007	0.261
	4		0.007	0.283	
	L2	Tidal	0	0.744	28.370
			1	0.538	20.510
			2	0.007	0.257
			3	0.003	0.113
			4	0.003	0.100
		5	0.004	0.135	
		Atmospheric	0	0.122	4.668
			1	0.023	0.890
			2	0.008	0.321
3			0.011	0.406	
4	0.010		0.363		
		5	0.009	0.339	

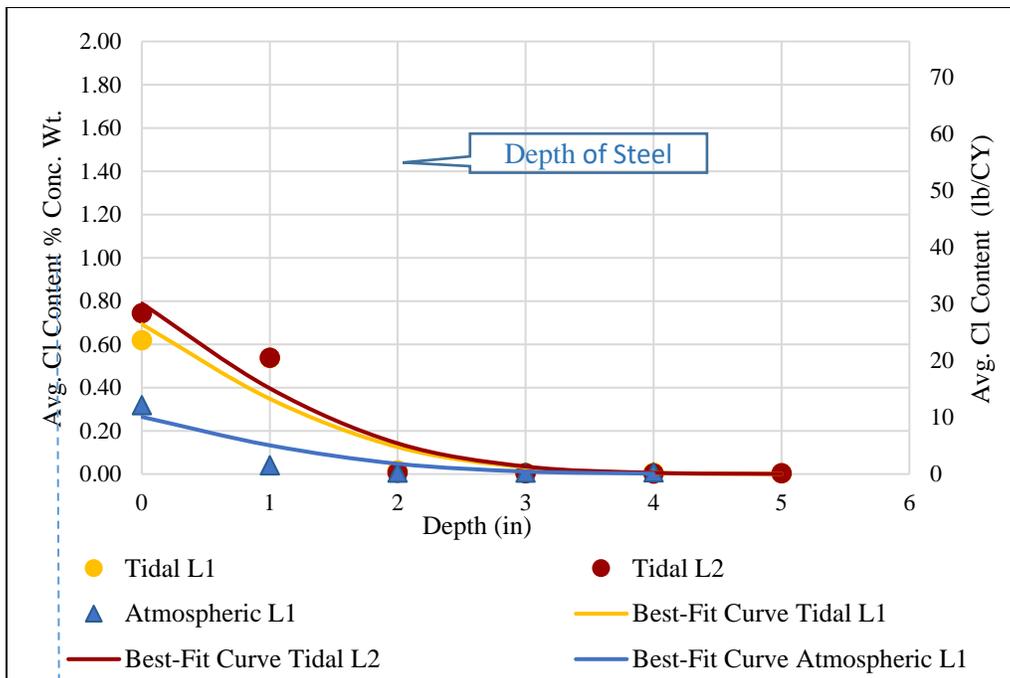


A.2.2 iCOR Test Results (Corrosion Rate & Surface Resistivity)

Structure #	Location	Atmospheric Zone					Tidal Zone						
		Corrosion Rate	Avg. Corrosion Rate	Concrete Resist.	Avg. Concrete Resist.	R2	Corrosion Rate	Avg. Corrosion Rate	Concrete Resist.	Avg. Concrete Resist.	R2		
090056	L1	7.86	27.98	151	171.35	0.98	71	81	21	45	0.98		
		3.22		188		1	119		17		0.95		
		98.98		46.4		0.95	155		18		0.94		
		1.86		300		1	148		17		0.95		
							83			16		0.98	
							147			18		0.93	
							65			33		0.96	
							27			101		0.96	
							32			96		0.96	
							15			78		0.98	
							29			79		0.97	
		L2							48		38		0.93
									87		31		0.94
									84		31		0.98
							63		24		0.97		
							60		39		0.96		
							4		55		0.94		
							24		72		0.96		
							11		64		0.99		
							11		111		0.98		
							7		154		0.97		
							4		159		0.98		
							2		209		0.96		
							1		210		0.99		
							10		134		0.95		
						40		64		0.96			
						37		67		0.96			
						20		86		0.99			
					22		69		0.99				

A.2.3 Model 2: Iterative Non-linear Least-Squares Regression

Highly Corrosive/Silica Fume-Fly Ash	Structure #	Location	Zone	Chloride Content			Age (t)	Model 2: Iterative Non-linear Least-Squares Regression			
				Depth (in) (x)	% Conc. Wt.	Chlorides (lb/CY)		Diffusion Coefficient (in*in/yr) (D)	Surface Concentration (lb/CY) (C ₀)	Best-Fit Curve C _(x,t)	Residual Square
090056	L1	Tidal	0	0.619	23.610	16	0.070	26.433	26.433	7.966	
			1	0.536	20.450				13.283	51.367	
			2	0.017	0.659				4.755	16.777	
			3	0.005	0.200				1.170	0.940	
			4	0.007	0.269				0.193	0.006	
		5	0.004	0.135	0.021	0.013					
		Atmospheric	0	0.318	12.133	16	0.070	10.084	10.084	4.195	
			1	0.041	1.569				5.068	12.242	
			2	0.006	0.240				1.814	2.479	
			3	0.007	0.261				0.446	0.034	
	4		0.007	0.283	0.074				0.044		
	L2	Tidal	0	0.744	28.370	16	0.070	30.095	30.095	2.974	
			1	0.538	20.510				15.123	29.019	
			2	0.007	0.257				5.414	26.595	
			3	0.003	0.113				1.332	1.486	
4			0.003	0.100	0.220				0.014		
5			0.004	0.135	0.024				0.012		



A.2.4 Corrosion Modeling Service Life Reports

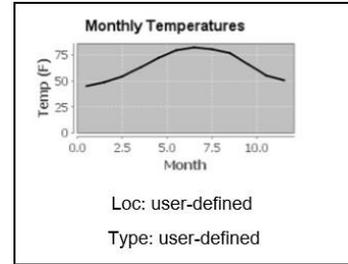
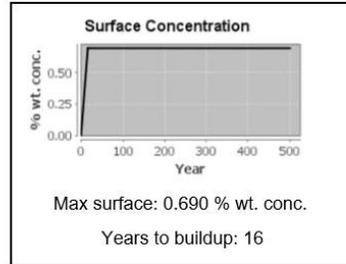
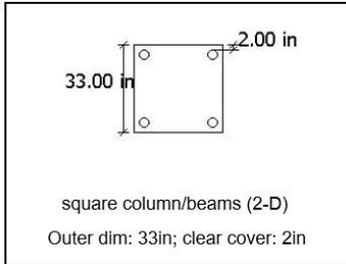
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 090056 L1

Description: Bent Cap/Highly Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 07/30/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concret Properties		0.37	Class F Fly Ash (25%); Silica Fume (5%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concret Properties	4.57E-9 in*in/sec	0.4	0.24 % wt. conc.	88.4 yrs	6 yrs	94.4 yrs
Measured Concrete Properties	-> 8.17E-9 in*in/sec	-> 0.4	-> 0.24 % wt. conc.	51.4 yrs	-> 6 yrs	57.4 yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

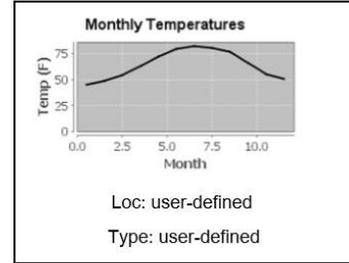
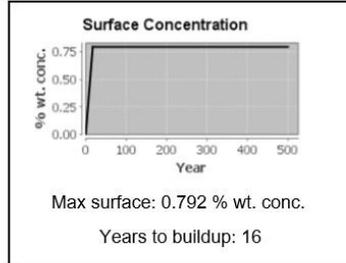
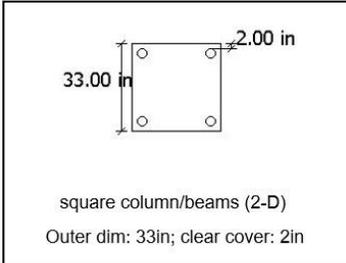
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 090056 L2

Description: Bent Cap\Highly Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 07/30/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concret Properties		0.37	Class F Fly Ash (25%); Silica Fume (5%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concret Properties	4.57E-9 in ² /sec	0.4	0.24 % wt. conc.	78.4 yrs	6 yrs	84.4 yrs
Measured Concrete Properties	-> 8.17E-9 in ² /sec	-> 0.4	-> 0.24 % wt. conc.	45.7 yrs	-> 6 yrs	51.7 yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

A.2.5 Materials and Tests Unit Statement of Concrete Mix Design

Form 312U
3-96

North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials

Project	Date Expires 02/28/2011
Mix Design Status Expired	Concrete Producer ARGOS USA LLC
County	Plant Location & DOT No. LELAND, NC - 15
Resident Engr.	Contractor
Class of Concrete CLASS AA	Date Assigned
Mix Design No. 152VF6034E	Contractor's Signature
Note Mix Design Units (English or Metric)	ENGLISH

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	CEMEX	KNOXVILLE, TN (TYPE I/II)	540 lbs.
Pozzolan	THE SEFA GROUP	WINYAH GENERATING STATION/GEORGETOWN	135 lbs.
Fine Aggregate	CAPE FEAR PAVING, LLC	WILMINGTON MATERIALS - OAK RIDGE PIT #1	1160 lbs.
Coarse Aggregate	MARTIN MARIETTA	CASTLE HAYNE QUARRY - CASTLE HAYNE	1540 lbs.
Total Water		WELL	30.0 gals.
Air. Entr. Agent	BASF CONSTRUCTION CHEMICALS, LLC	MASTERAIR AE 90 (AKA MB AE 90)	As recommended
Retarder	BASF CONSTRUCTION CHEMICALS, LLC	POZZOLITH 122R	As recommended
Water Reducer			
Superplasticizer			
Corrosion Inhibitor	BASF CONSTRUCTION CHEMICALS, LLC	MASTERLIFE CI 30 (AKA RHEOCRETE CNI)	As recommended

Mix Properties and Specifications

Slump 3.50 in. Mortar Content 16.84 cu. ft.
Max Water 32.3 gals. Air Content 6.0 %

Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.65	1.0	NA	2.40
Coarse Aggregate, #57	2.43	3.1	76.5	NA

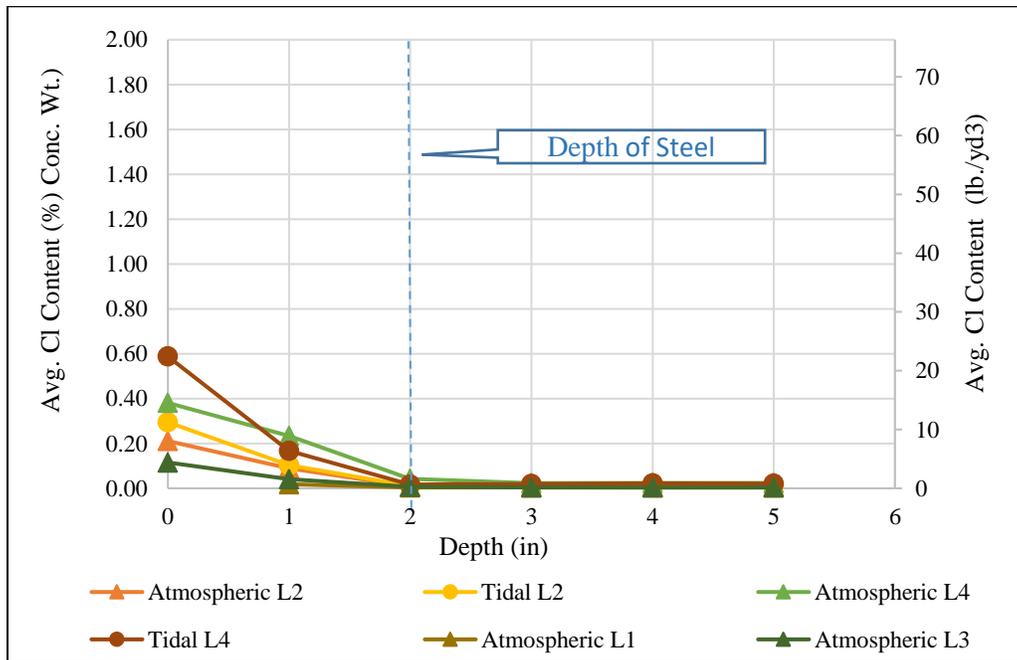
Comment Mix also contains 35 lbs. Elkim silica fume and 3 gals. CNI corrosion inhibitor. Use of 5 % silica fume and 1:1 cement:fly ash replacement rate are in accordance with a note on the plans for 8.1231503. Mix is for caps on bent #s 1 - 4.

Cast-in-place concrete shall conform to Section 1000, precast concrete to Section 1077, and prestressed concrete to Section 1078 of the applicable edition of the Standard Specifications for Roads and Structures plus all applicable Special Provisions.

A.3 Structure 660021 Results Summary

A.3.1 RCT Test Results (Chloride Content Profile)

Structure #	Location	Zone	Chloride Content		
			Depth (in)	% Conc. Wt.	Chlorides (lb/CY)
660021	L1	Tidal	0	0.220	8.397
			1	0.243	9.252
			2	0.150	5.738
			3	0.094	3.592
			4	0.018	0.705
		5	0.012	0.443	
		Atmospheric	0	0.115	4.404
			1	0.023	0.887
			2	0.004	0.156
			3	0.003	0.109
	4		0.003	0.105	
	L2	Tidal	0	0.412	15.718
			1	0.297	11.327
			2	0.161	6.157
			3	0.030	1.159
			4	0.006	0.247
		5	0.005	0.185	
		Atmospheric	0	0.183	6.987
			1	0.043	1.628
			2	0.004	0.145
3			0.004	0.142	
4	0.003		0.112		
		5	0.003	0.116	

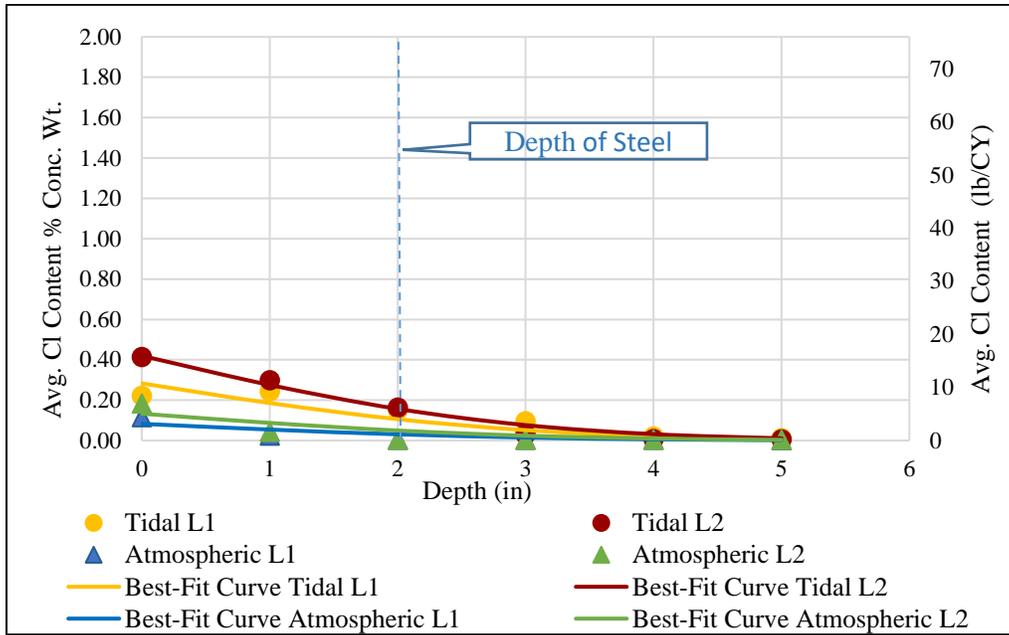


A.3.2 iCOR Test Results (Corrosion Rate & Surface Resistivity)

Structure #	Location	Atmospheric Zone					Tidal Zone									
		Corrosion Rate	Avg. Corrosion Rate	Concrete Resist.	Avg. Concrete Resist.	R2	Corrosion Rate	Avg. Corrosion Rate	Concrete Resist.	Avg. Concrete Resist.	R2					
660021	L1	4	4	119	180	1	403	206	15	16	0.96					
		5		112		0.97	13		28		0.99					
		5		208		0.99	231		11		0.99					
		1		204		0.99	336		9		0.98					
		4		234		0.96	182		17		0.99					
		7		200		0.89	153		22		0.98					
							162				12		0.92			
							172				11		0.99			
		L2		0		4	257		198		0.97	15	88	74	28	1
				3			226				0.97	136		23		1
	3		178	1	114		14	0.98								
	3		191	0.98	78		12	0.99								
	3		177	0.99	46		56	0.95								
	10		158	0.98	51		21	0.99								
							147			10		0.99				
							119			10		0.94				

A.3.3 Model 2: Iterative Non-linear Least-Squares Regression

Structure #	Location	Zone	Chloride Content			Age (t)	Model 2: Iterative Non-linear Least-Squares Regression			
			Depth (in) (x)	% Conc. Wt.	Chlorides (lb/CY)		Diffusion Coefficient (in*in/yr) (D)	Surface Concentration (lb/CY) (C _o)	Best-Fit Curve C _(x,t)	Residual Square
660021	L1	Tidal	0	0.220	8.397	15	0.169	10.765	10.765	5.606
			1	0.243	9.252				7.068	4.769
			2	0.150	5.738				4.025	2.936
			3	0.094	3.592				1.962	2.656
			4	0.018	0.705				0.811	0.011
			5	0.012	0.443				0.282	0.026
		Atmospheric	0	0.115	4.404	15	0.169	3.151	3.151	1.569
			1	0.023	0.887				2.069	1.397
			2	0.004	0.156				1.178	1.045
			3	0.003	0.109				0.574	0.217
			4	0.003	0.105				0.237	0.018
			5	0.003	0.102				0.083	0.000
	L2	Tidal	0	0.412	15.718	15	0.169	15.953	15.953	0.055
			1	0.297	11.327				10.475	0.726
			2	0.161	6.157				5.965	0.037
			3	0.030	1.159				2.908	3.057
			4	0.006	0.247				1.202	0.911
			5	0.005	0.185				0.418	0.055
		Atmospheric	0	0.183	6.987	15	0.169	5.059	5.059	3.717
			1	0.043	1.628				3.322	2.869
			2	0.004	0.145				1.892	3.050
3			0.004	0.142	0.922				0.609	
4			0.003	0.112	0.381				0.072	
5			0.003	0.116	0.133				0.000	



A.3.4 Corrosion Modeling Service Life Reports

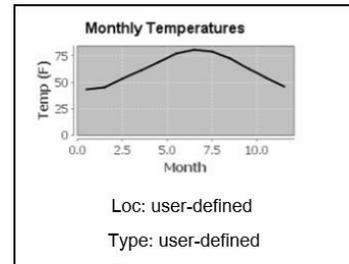
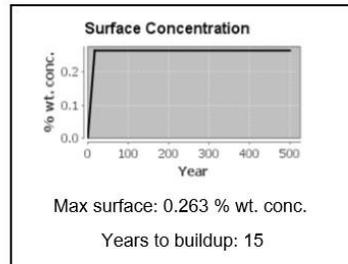
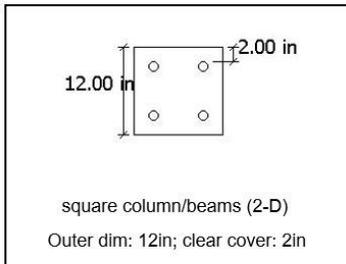
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 660021 L1

Description: Bent Cap\Highly Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 07/30/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concret Properties		0.32		Ca Nitrite - 3.5 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concret Properties	7.91E-9 in ² /sec	0.2	0.28 % wt. conc.	500+ yrs	6 yrs	506+ yrs
Measured Concrete Properties	-> 2.15E-8 in ² /sec	-> 0.26	-> 0.28 % wt. conc.	500+ yrs	-> 6 yrs	506+ yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

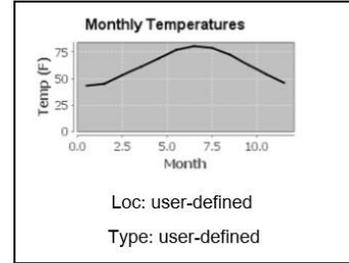
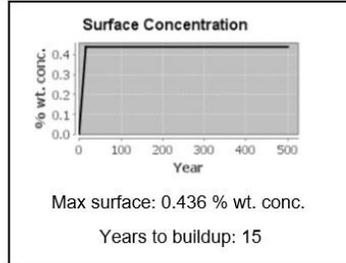
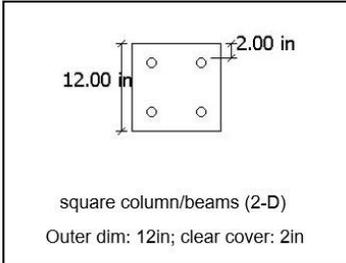
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 660021 L2

Description: Bent Cap/Highly Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 07/30/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concret Properties		0.32		Ca Nitrite - 3.5 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concret Properties	7.91E-9 in ² /sec	0.2	0.28 % wt. conc.	46.5 yrs	6 yrs	52.5 yrs
Measured Concrete Properties	-> 2.15E-8 in ² /sec	-> 0.26	-> 0.28 % wt. conc.	26.8 yrs	-> 6 yrs	32.8 yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

A.3.5 Materials and Tests Unit Statement of Concrete Mix Design

**North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials**

Project	Date Expires 12/31/2075
Mix Design Status Active	Concrete Producer FLORENCE CONCRETE PROD.
County	Plant Location & DOT No. SUMTER, SC - 10
Resident Engr.	Contractor
Class of Concrete PRESTRESS	Date Assigned
Mix Design No. 10PVO6167935E	Contractor's Signature
Note Mix Design Units (English or Metric) ENGLISH	

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	HOLCIM (US) INC.	HOLLY HILL, SC (TYPE I/II)	752 lbs.
Pozzolan			lbs.
Fine Aggregate	AMERICAN MATERIALS CO.	SUMTER COUNTY SAND PIT	1005 lbs.
Coarse Aggregate	MARTIN MARIETTA	CAYCE QUARRY - CAYCE, SC	1866 lbs.
Total Water		CITY	28.4 gals.
Air. Entr. Agent	SIKA CORPORATION	SIKA AEA-14	As recommended
Retarder	SIKA CORPORATION	PLASTIMENT	As recommended
Water Reducer			
Superplasticizer	SIKA CORPORATION	SIKA VISCOCRETE-6100	As recommended
Corrosion Inhibitor	SIKA CORPORATION	SIKA-CNI	As recommended

Mix Properties and Specifications

Slump 8.00 in. **Mortar Content** 15.59 cu. ft.
Max Water 33.0 gals. **Air Content** 5.0 %

Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.62	0.5	NA	
Coarse Aggregate, #67	2.62	0.4		NA

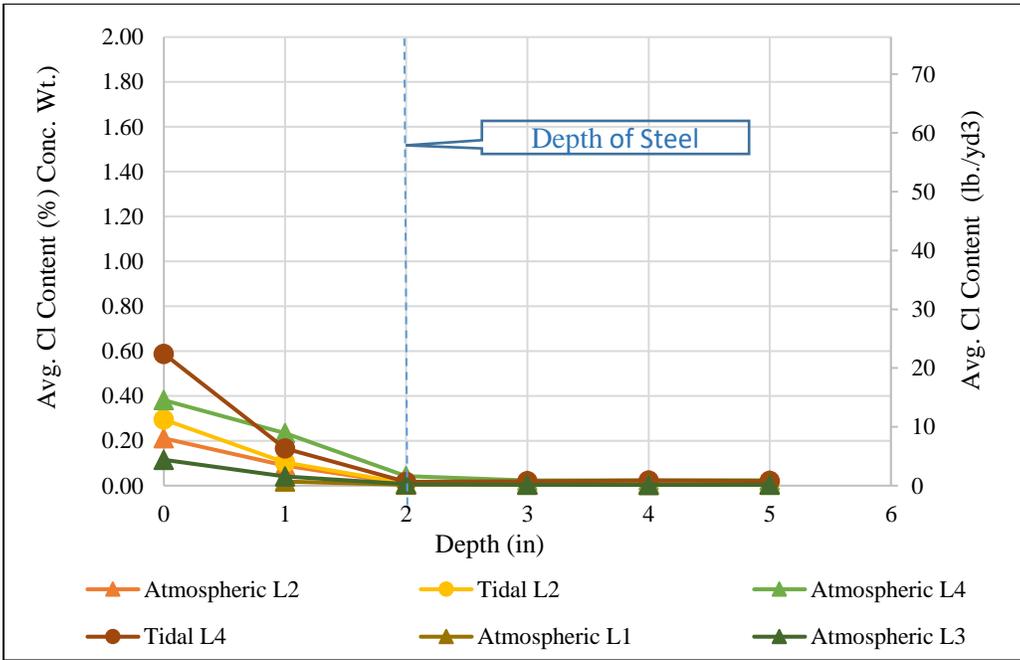
Comment Designed for 6000 psi at 28 days. Contains 3.5 gals. corrosion inhibitor. Max. w/c = 0.40.

Cast-in-place concrete shall conform to Section 1000, precast concrete to Section 1077, and prestressed concrete to Section 1078 of the applicable edition of the Standard Specifications for Roads and Structures plus all applicable Special Provisions.

A.4Structure 640010 Results Summary

A.4.1 RCT Test Results (Chloride Content Profile)

Structure #	Location	Zone	Chloride Content		
			Depth (in)	% Conc. Wt.	Chlorides (lb/CY)
640010	L1	Atmospheric	0		
			1	0.018	0.69240
			2	0.004	0.13930
			3	0.004	0.15010
			4	0.004	0.14460
			5	0.003	0.13160
	L2	Atmospheric	0	0.211	8.0443
			1	0.089	3.4037
			2	0.008	0.3142
			3	0.010	0.3771
			4	0.010	0.3861
			5	0.008	0.3052
		Tidal	0	0.295	11.237
			1	0.104	3.957
			2	0.010	0.391
			3	0.011	0.408
			4	0.013	0.504
			5	0.012	0.468
	L3	Atmospheric	0	0.115	4.3951
			1	0.041	1.5741
			2	0.007	0.2814
			3	0.004	0.1416
			4	0.003	0.1251
			5	0.004	0.1573
	L4	Tidal	0	0.381	14.5323
			1	0.234	8.9158
			2	0.043	1.6317
			3	0.023	0.8800
			4	0.024	0.9211
			5	0.024	0.9322
Atmospheric		0	0.588	22.424	
		1	0.166	6.347	
		2	0.017	0.648	
		3	0.019	0.738	
		4	0.023	0.860	
		5	0.021	0.793	

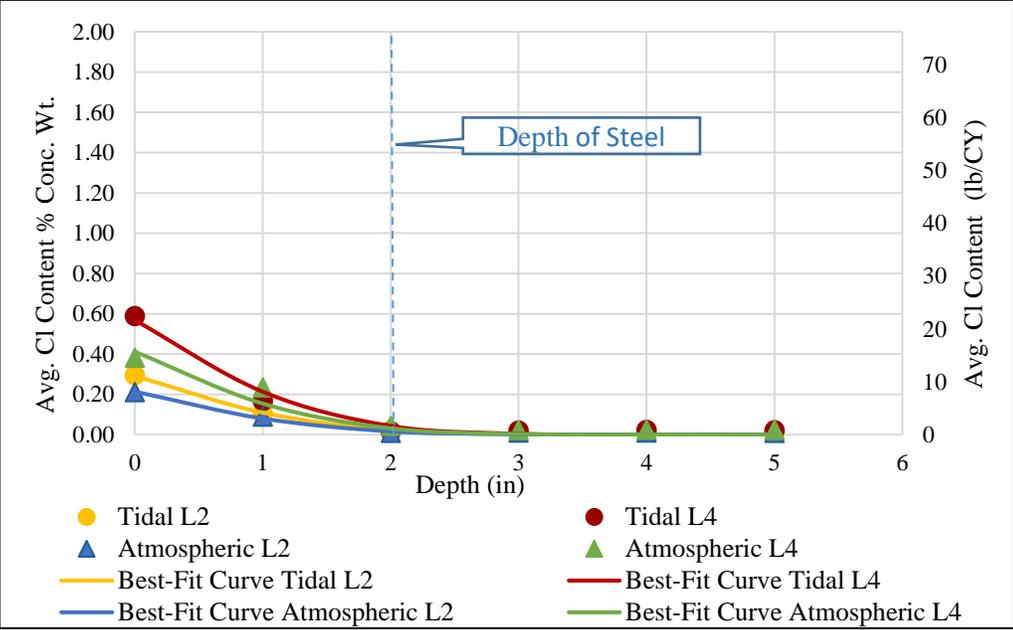


A.4.2 iCOR Test Results (Corrosion Rate & Surface Resistivity)

Structure #	Location	Atmospheric Zone				Tidal Zone					
		Corrosion Rate	Avg. Corrosion Rate	Concrete Resist.	Avg. Concrete Resist.	R2	Corrosion Rate	Avg. Corrosion Rate	Concrete Resist.	Avg. Concrete Resist.	R2
640010	L1	2.7	2.2725	159	242.38	1					
		2.5		161		1					
		3.8		162		0.99					
		1.6		182		1					
		3		62		0.96					
		1.8		364		0.96					
		0.88		352		1					
		1.9		497		0.99					
	L2	8.7	7.4333	72	90.889	1	17.7	26.493	71	60.5	0.95
		6		65		1	14.37		93		0.98
		4.3		13		1	21.18		61		0.97
		9.2		189		1	21.78		67		0.91
		7		50		1	39.22		32		0.97
		5.4		57		1	44.71		39		0.83
		1.7		160		0.99					
		17		88		1					
		7.6		124		1					
		L3		0.83		2.104	173		390.6		1
	3.3		127	1							
	2.9		186	1							
	7		172	1							
	3		350	0.99							
	0.46		442	1							
	0.37		801	1							
	0.76		624	1							
	0.92		461	1							
	1.5		570	0.99							
	L4	19	31.4	172	128.44	1	23.43	29.538	74	52.444	0.97
		4.1		144		0.98	17.59		85		0.98
		6.5		130		1	30.69		43		0.93
		77		91		0.97	31.03		40		0.97
		14		136		1	39.8		33		0.86
		67		134		0.96	34.44		32		0.97
		36		107		0.99	33.27		34		0.96
		44		111		0.96	17.16		87		0.97
		15		131		0.97	38.43		44		0.94

A.4.3 Model 2: Iterative Non-linear Least-Squares Regression

Structure #	Location	Zone	Chloride Content			Age (t)	Model 2: Iterative Non-linear Least-Squares Regression			
			Depth (in) (x)	% Conc. Wt.	Chlorides (lb/CY)		Diffusion Coefficient (in*in/yr) (D)	Surface Concentration (lb/CY) (C _o)	Best-Fit Curve	Residual Square
									C _(x,t)	
640010	L2	Atmospheric	0	0.211	8.0443	14	0.045	8.153	8.153	0.012
			1	0.089	3.4037				3.057	0.120
			2	0.008	0.3142				0.619	0.093
			3	0.010	0.3771				0.063	0.098
			4	0.010	0.3861				0.003	0.147
		5	0.008	0.3052	0.000	0.093				
		Tidal	0	0.295	11.237	14	0.045	11.125	11.125	0.013
			1	0.104	3.957				4.171	0.046
			2	0.010	0.391				0.845	0.206
			3	0.011	0.408				0.086	0.103
	4		0.013	0.504	0.004				0.250	
	L4	Atmospheric	0	0.381	14.5323	14	0.045	15.707	15.707	1.379
			1	0.234	8.9158				5.889	9.164
			2	0.043	1.6317				1.193	0.192
			3	0.023	0.8800				0.122	0.574
			4	0.024	0.9211				0.006	0.837
		5	0.024	0.9322	0.000	0.869				
		Tidal	0	0.588	22.424	14	0.045	21.684	21.684	0.547
			1	0.166	6.347				8.130	3.178
			2	0.017	0.648				1.647	0.998
3			0.019	0.738	0.168				0.324	
4	0.023		0.860	0.008	0.725					
5	0.021	0.793	0.000	0.629						



A.4.4 Corrosion Modeling Service Life Reports

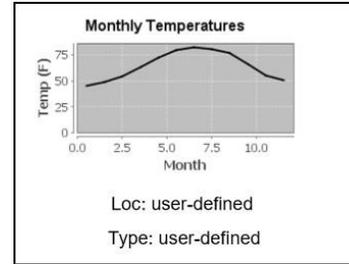
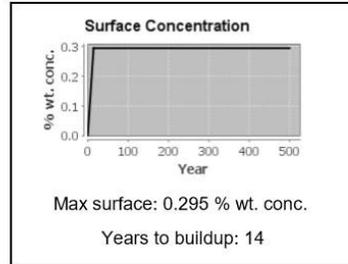
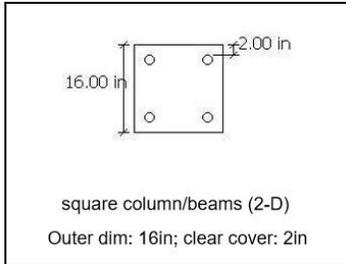
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 640010 L2

Description: Prestressed Pile\Highly Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 07/30/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.32	Silica Fume (5%);	Ca Nitrite - 3.5 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concrete Properties	3.47E-9 in ² /in/sec	0.2	0.28 % wt. conc.	500+ yrs	6 yrs	506+ yrs
Measured Concrete Properties	-> 2.49E-9 in ² /in/sec	-> 0.26	-> 0.28 % wt. conc.	500+ yrs	-> 6 yrs	506+ yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

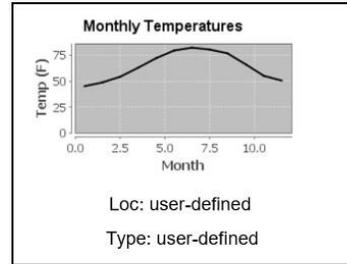
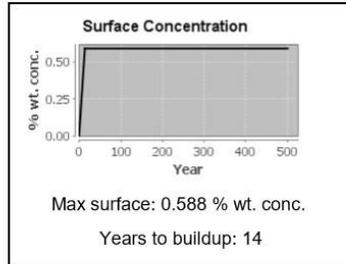
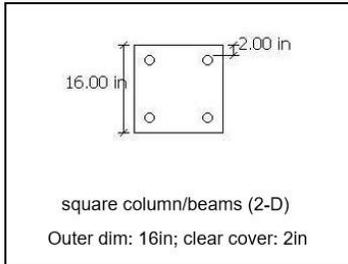
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 640010 L4

Description: Prestressed Pile\Highly Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 07/30/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concret Properties		0.32	Silica Fume (5%);	Ca Nitrite - 3.5 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concret Properties	3.47E-9 in ² /in/sec	0.2	0.28 % wt. conc.	56.5 yrs	6 yrs	62.5 yrs
Measured Concrete Properties	-> 2.49E-9 in ² /in/sec	-> 0.26	-> 0.28 % wt. conc.	104.7 yrs	-> 6 yrs	110.7 yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

A.4.5 Materials and Tests Unit Statement of Concrete Mix Design

Form 312U
3-96

**North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials**

Project	Date Expires 12/31/2075
Mix Design Status Active	Concrete Producer ARGOS USA LLC
County	Plant Location & DOT No. WILMINGTON, NC - 192
Resident Engr.	Contractor
Class of Concrete CLASS AA	Date Assigned
Mix Design No. 1922VF6034CE	Contractor's Signature
Note Mix Design Units (English or Metric) ENGLISH	

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	ARGOS USA	BARRANQUILLA PLANT - (FORMERLY CEMENT)	540 lbs.
Pozzolan	SOUTHEASTERN FLY ASH	SEFA - WINYAH GENERATING STATION/GEORG	135 lbs.
Fine Aggregate	CAPE FEAR PAVING, LLC	WILMINGTON MATERIALS - OAK RIDGE PIT #1	1160 lbs.
Coarse Aggregate	MARTIN MARIETTA	CASTLE HAYNE QUARRY - CASTLE HAYNE	1540 lbs.
Total Water		WELL	30.0 gals.
Air. Entr. Agent	BASF CONSTRUCTION CHEMICALS, LLC	MASTERAIR AE 90 (AKA MB AE 90)	As recommended
Retarder	BASF CONSTRUCTION CHEMICALS, LLC	POZZOLITH 122R	As recommended
Water Reducer			
Superplasticizer			
Corrosion Inhibitor	BASF CONSTRUCTION CHEMICALS, LLC	MASTERLIFE CI 30 (AKA RHEOCRETE CNI)	As recommended

Mix Properties and Specifications

Slump 3.50 in. Mortar Content 16.84 cu. ft.
Max Water 32.3 gals. Air Content 6.0 %

Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.65	1.0	NA	2.40
Coarse Aggregate, #57	2.43	3.1	76.5	NA

Comment Mix contains 35 lbs. Elkim silica fume and 3 gals. CNI corrosion inhibitor. Use of 5 % silica fume and 1:1 cement:fly ash replacement rate are in accordance with a plan note for 8.1231503. Mix is for caps on bent #s 1 - 4. Cement imported by Port Royal.

Cast-in-place concrete shall conform to Section 1000, precast concrete to Section 1077, and prestressed concrete to Section 1078 of the applicable edition of the Standard Specifications for Roads and Structures plus all applicable Special Provisions.

**North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials**

Project	Date Expires 12/31/2075
Mix Design Status Active	Concrete Producer S & G PRESTRESS CONCRETE
County	Plant Location & DOT No. LELAND, NC - 2
Resident Engr.	Contractor
Class of Concrete PRESTRESS	Date Assigned
Mix Design No. 2PVU13CIMSE	Contractor's Signature
Note Mix Design Units (English or Metric) ENGLISH	

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	ARGOS USA	BARRANQUILLA PLANT - (FORMERLY CEMENTO)	665 lbs.
Pozzolan	Degussa Rheomac SF 100 silica fume		35 lbs.
Fine Aggregate	MARTIN MARIETTA	ROCKY POINT QUARRY - ROCKY POINT	1244 lbs.
Coarse Aggregate	MARTIN MARIETTA	GARNER QUARRY - GARNER	1700 lbs.
Total Water		WELL	29.4 gals.
Air. Entr. Agent	BASF CONSTRUCTION CHEMICALS, LLC	MASTERAIR AE 90 (AKA MB AE 90)	As recommended
Retarder	BASF CONSTRUCTION CHEMICALS, LLC	POZZOLITH 122R	As recommended
Water Reducer			
Superplasticizer	BASF CONSTRUCTION CHEMICALS, LLC	MASTERGLENIUM 3030 (AKA GLENIUM 3030NS)	As recommended
Corrosion Inhibitor	BASF CONSTRUCTION CHEMICALS, LLC	MASTERLIFE CI 30 (AKA RHEOCRETE CNI)	As recommended

Mix Properties and Specifications

Slump 7.00 in. Mortar Content 16.60 cu. ft.
Max Water 31.0 gals. Air Content 5.0 %

Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.65	1.0	NA	2.40
Coarse Aggregate, #67	2.62	0.5	93.5	NA

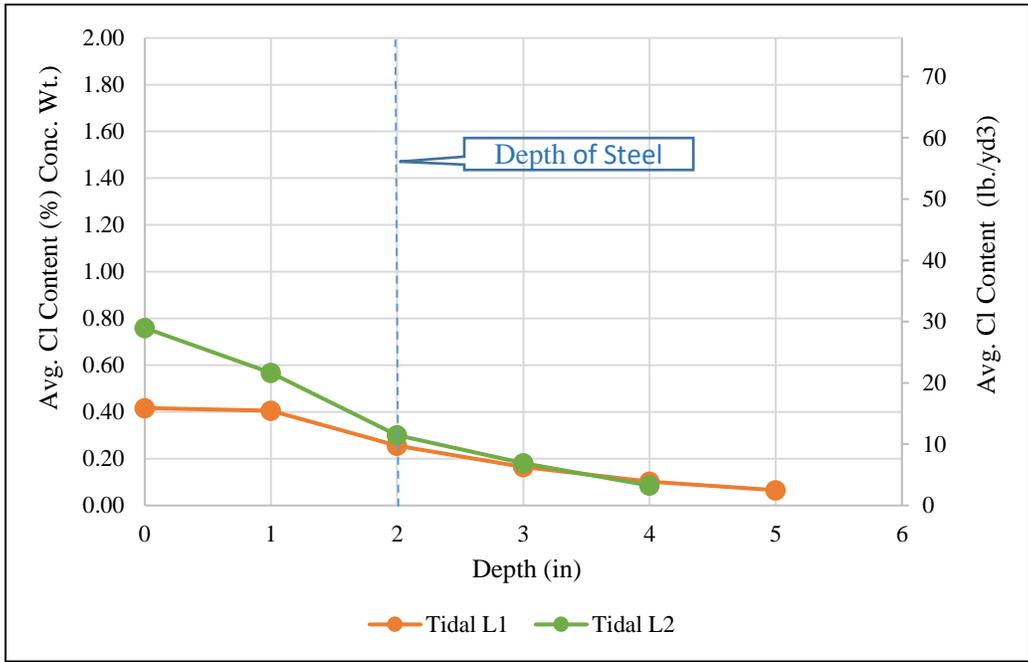
Comment Mix is designed for 6000 psi. Mix contains 3.0 gals. CNI and 35 lbs. Rheomac SF 100 silica fume. (Silica fume quantity is listed on Form 312 under "pozzolan.") Max. water-cement ratio is 0.4.

Cast-in-place concrete shall conform to Section 1000, precast concrete to Section 1077, and prestressed concrete to Section 1078 of the applicable edition of the Standard Specifications for Roads and Structures plus all applicable Special Provisions.

A.5.1 RCT Test Results (Chloride Content Profile)

Structure #	Location	Zone	Chloride Content		
			Depth (in)	% Conc. Wt.	Chlorides (lb/CY)
090061	L1	Tidal	0	0.417	15.894
			1	0.405	15.463
			2	0.256	9.762
			3	0.164	6.268
			4	0.102	3.908
	L2	Tidal	0	0.759	28.972
			1	0.567	21.638
			2	0.300	11.451
			3	0.181	6.905
			4	0.085	3.249



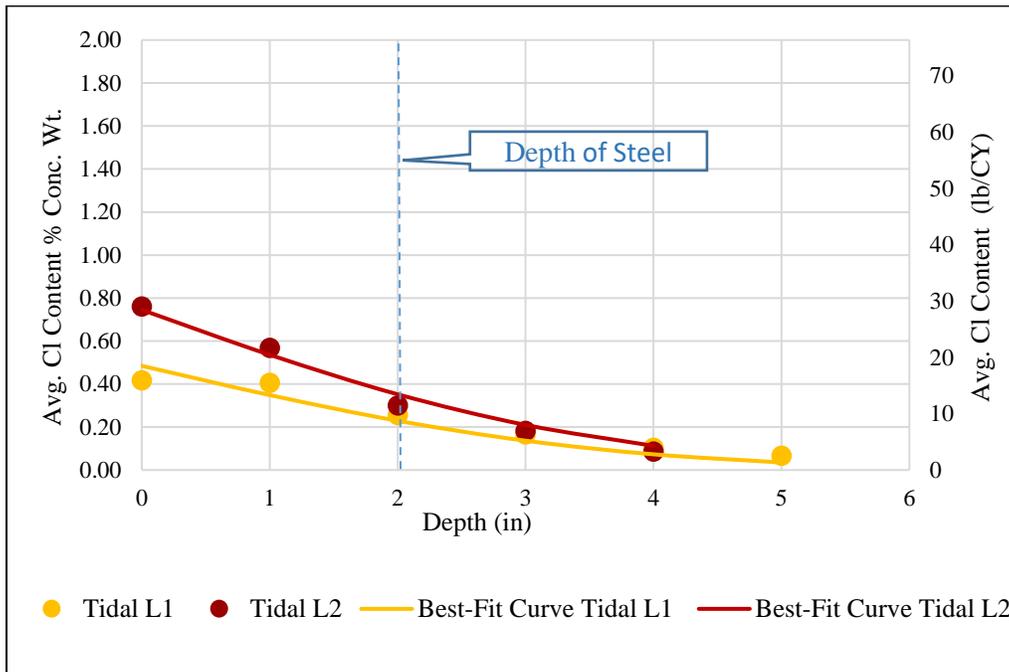


A.5.2 iCOR Test Results (Corrosion Rate & Surface Resistivity)

Structure #	Location	Tidal Zone				R2
		Corrosion Rate (Y)	Avg. Corrosion Rate	Concrete Resist. (Y)	Avg. Concrete Resist.	
090061	L2	9	76	73	84	0.93
		9		144		0.98
		4		283		1
		88		30		0
		104		40		97
		1		176		0.96
		47		44		1
		13		59		0.99
		1		403		0.99
		37		11		0.98
		65		29		0.99
		61		73		0.99
		251		6.7		0.99
		209		14		0.98
		4.2		49		0.98
		203		6.8		0.99
		238		14		0.99
	16	49	1			
	L3	5.1	63	70	58	0.96
		0.6		185		0.92
		3.6		128		1
		7.2		52		0.98
		16		51		1
		4.5		35		1
		61		58		0.93
		15		73		0.99
		7.4		103		1
		130		9.9		1
		139		40		1
		31		72		0.99
		274		7.2		0.99
		35		27		1
		112		31		1
233		7.8		1		
53	41	0.99				
15	55	0.99				

A.5.3 Model 2: Iterative Non-linear Least-Squares Regression

Silica Fume	Structure #	Location	Zone	Chloride Content			Age (t)	Model 2: Iterative Non-linear Least-Squares Regression			
				Depth (in) (x)	% Conc. Wt.	Chlorides (lb/CY)		Diffusion Coefficient (in*in/yr) (D)	Surface Concentration (lb/CY) (C ₀)	Best-Fit Curve C _(x,t)	Residual Square
090061	L1	Tidal	0	0.417	15.8939	15	0.259	18.484	18.484	6.709	
			1	0.405	15.4630				13.302	4.671	
			2	0.256	9.7618				8.740	1.044	
			3	0.164	6.2677				5.204	1.131	
			4	0.102	3.9075				2.792	1.243	
	5	0.065	2.4984	1.344	1.333						
	L2	Tidal	0	0.759	28.972	15	0.259	28.419	28.419	0.305	
			1	0.567	21.638				20.452	1.407	
			2	0.300	11.451				13.438	3.946	
			3	0.181	6.905				8.002	1.204	
4			0.085	3.249	4.293				1.091		



A.5.4 Corrosion Modeling Service Life Reports

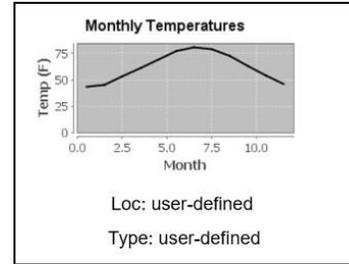
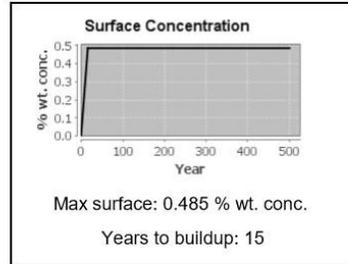
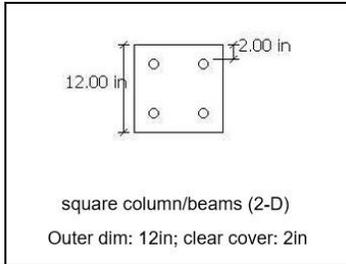
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 090061 L3

Description: Bent Cap\Highly Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 07/30/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.35	Silica Fume (5%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concrete Properties	4.09E-9 in ² /sec	0.2	0.24 % wt. conc.	54.6 yrs	6 yrs	60.6 yrs
Measured Concrete Properties	-> 1.45E-8 in ² /sec	-> 0.26	-> 0.24 % wt. conc.	25.3 yrs	-> 6 yrs	31.3 yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

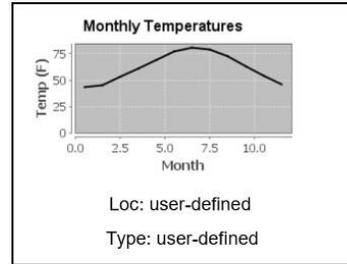
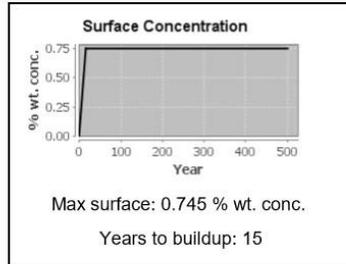
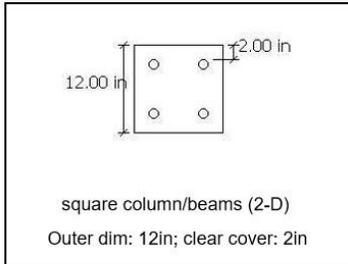
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 090061 L4

Description: Bent Cap\Highly Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 07/30/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concret Properties		0.35	Silica Fume (5%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concret Properties	4.09E-9 in ² /in/sec	0.2	0.24 % wt. conc.	35.7 yrs	6 yrs	41.7 yrs
Measured Concrete Properties	-> 1.45E-8 in ² /in/sec	-> 0.26	-> 0.24 % wt. conc.	17.9 yrs	-> 6 yrs	23.9 yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

A.5.5 Materials and Tests Unit Statement of Concrete Mix Design

Form 312U
3-96

North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials

Project	Date Expires 12/31/2075
Mix Design Status Active	Concrete Producer S & G PRESTRESS CONCRETE
County	Plant Location & DOT No. LELAND, NC - 2
Resident Engr.	Contractor
Class of Concrete PRESTRESS	Date Assigned
Mix Design No. 2PVU13CIMSE	Contractor's Signature
Note Mix Design Units (English or Metric) ENGLISH	

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	ARGOS USA	BARRANQUILLA PLANT - (FORMERLY CEMENT)	665 lbs.
Pozzolan	Degussa Rheomac SF 100 silica fume		35 lbs.
Fine Aggregate	MARTIN MARIETTA	ROCKY POINT QUARRY - ROCKY POINT	1244 lbs.
Coarse Aggregate	MARTIN MARIETTA	GARNER QUARRY - GARNER	1700 lbs.
Total Water		WELL	29.4 gals.
Air. Entr. Agent	BASF CONSTRUCTION CHEMICALS, LLC	MASTERAIR AE 90 (AKA MB AE 90)	As recommended
Retarder	BASF CONSTRUCTION CHEMICALS, LLC	POZZOLITH 122R	As recommended
Water Reducer			
Superplasticizer	BASF CONSTRUCTION CHEMICALS, LLC	MASTERGLENIUM 3030 (AKA GLENIUM 3030NS)	As recommended
Corrosion Inhibitor	BASF CONSTRUCTION CHEMICALS, LLC	MASTERLIFE CI 30 (AKA RHEOCRETE CNI)	As recommended

Mix Properties and Specifications

Slump 7.00 in. Mortar Content 16.60 cu. ft.
Max Water 31.0 gals. Air Content 5.0 %

Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.65	1.0	NA	2.40
Coarse Aggregate, #67	2.62	0.5	93.5	NA

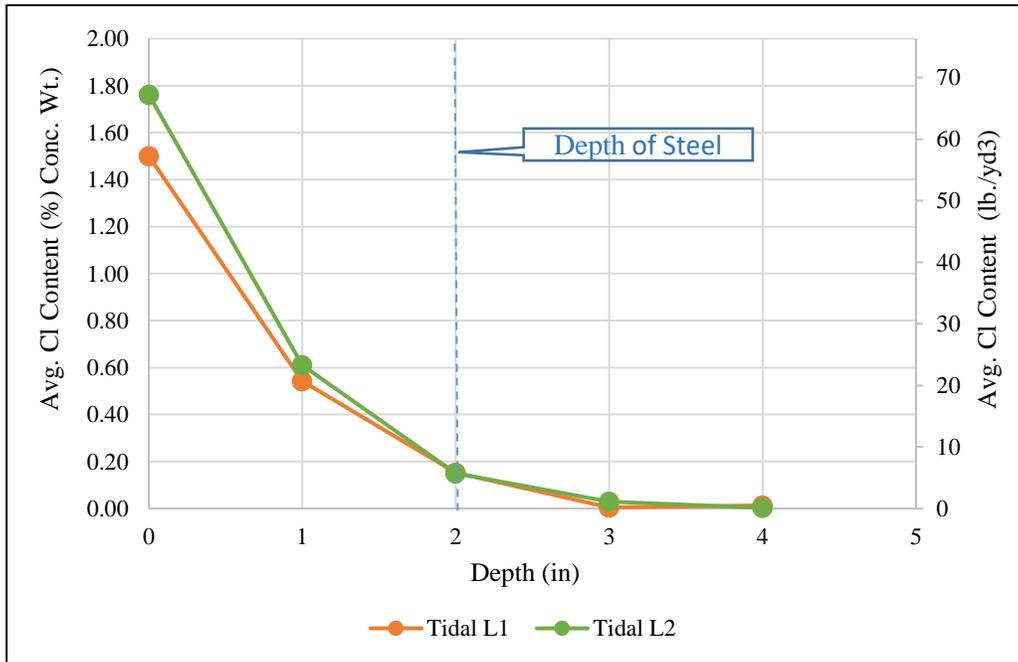
Comment Mix is designed for 6000 psi. Mix contains 3.0 gals. CNI and 35 lbs. Rheomac SF 100 silica fume. (Silica fume quantity is listed on Form 312 under "pozzolan.") Max. water-cement ratio is 0.4.

Cast-in-place concrete shall conform to Section 1000, precast concrete to Section 1077, and prestressed concrete to Section 1078 of the applicable edition of the Standard Specifications for Roads and Structures plus all applicable Special Provisions.

A.6 Structure 090206 Results Summary

A.6.1 RCT Test Results (Chloride Content Profile)

Structure #	Location	Zone	Chloride Content		
			Depth (in)	% Conc. Wt.	Chlorides (lb/CY)
090206	L1	Tidal	0	1.500	57.241
			1	0.543	20.711
			2	0.153	5.844
			3	0.005	0.185
			4	0.013	0.508
	L2	Tidal	0	1.762	67.231
			1	0.611	23.305
			2	0.150	5.741
			3	0.030	1.143
			4	0.003	0.119



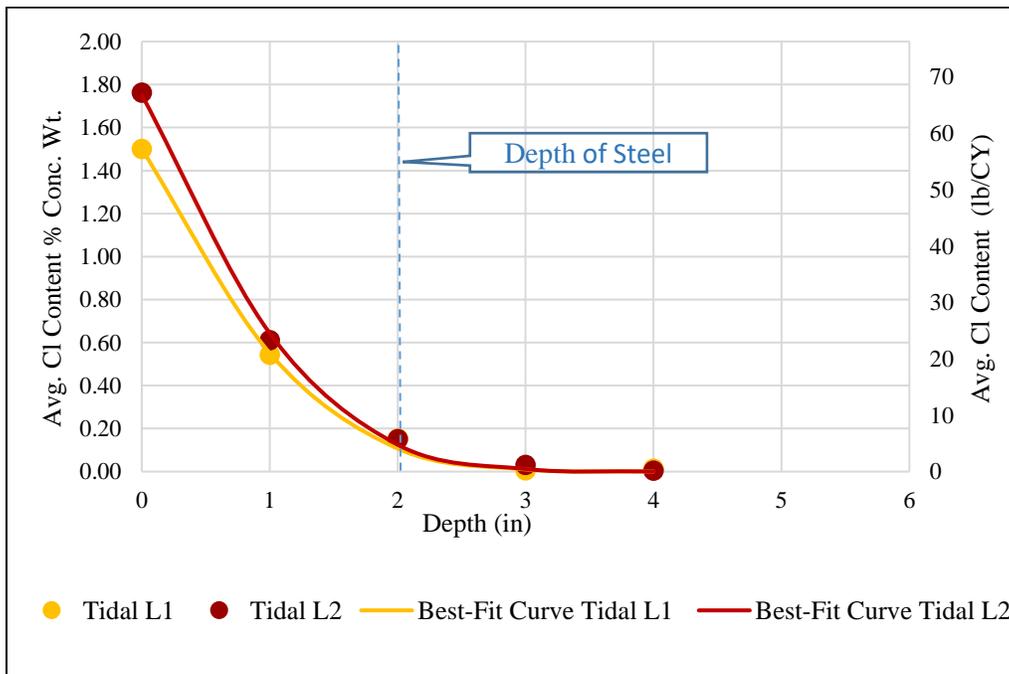


A.6.2 iCOR Test Results (Corrosion Rate & Surface Resistivity)

Structure #	Location	Tidal Zone				R2
		Corrosion Rate (Y)	Avg. Corrosion Rate	Concrete Resist. (Y)	Avg. Concrete Resist.	
090206	L1	48	83	42	35	0.98
		49		39		0.98
		90		38		0.92
		110		31		0.98
		120		25		0.98
	L2	79	103	43	30	0.98
		119		36		0.98
		93		33		0.92
		98		32		0.98
		89		24		0.98
		138		13		0.99

A.6.3 Model 2: Iterative Non-linear Least-Squares Regression

Silica Fume	Structure #	Location	Zone	Chloride Content			Age (t)	Model 2: Iterative Non-linear Least-Squares Regression			
				Depth (in) (x)	% Conc. Wt.	Chlorides (lb/CY)		Diffusion Coefficient (in*in/yr) (D)	Surface Concentration (lb/CY) (C ₀)	Best-Fit Curve C _(x,t)	Residual Square
090206	L1	Tidal	0	1.500	57.2408	12	0.051	57.248	57.248	0.000	
			1	0.543	20.7113				21.031	0.102	
			2	0.153	5.8441				4.088	3.084	
			3	0.005	0.1847				0.392	0.043	
			4	0.013	0.5079				0.018	0.240	
	L2	Tidal	0	1.762	67.231	12	0.051	66.845	66.845	0.149	
			1	0.611	23.305				24.556	1.565	
			2	0.150	5.741				4.773	0.935	
			3	0.030	1.143				0.458	0.470	
			4	0.003	0.119				0.021	0.010	



A.6.4 Corrosion Modeling Service Life Reports

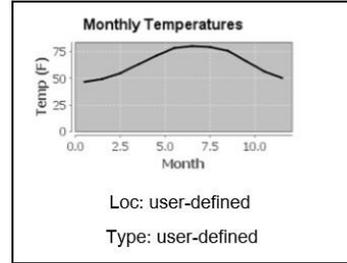
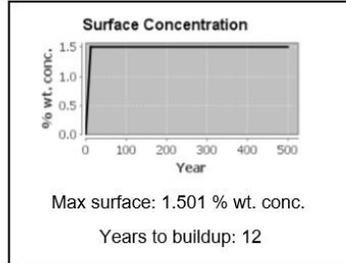
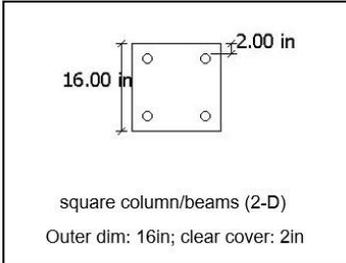
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 090206 L1

Description: Prestressed Pile\Highly Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 02/16/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.38	Class F Fly Ash (46%); Silica Fume (7%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concrete Properties	3.47E-9 in ² /sec	0.57	0.24 % wt. conc.	163.5 yrs	6 yrs	169.5 yrs
Measured Concrete Properties	-> 7.65E-9 in ² /sec	-> 0.57	-> 0.24 % wt. conc.	73.2 yrs	-> 6 yrs	79.2 yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

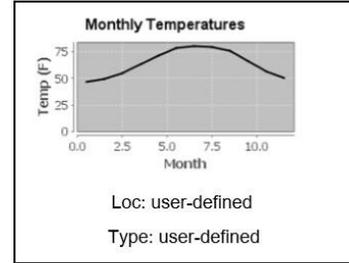
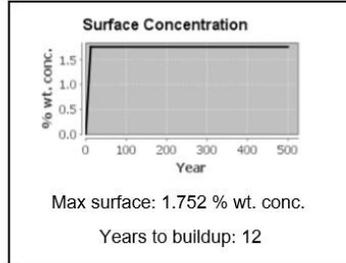
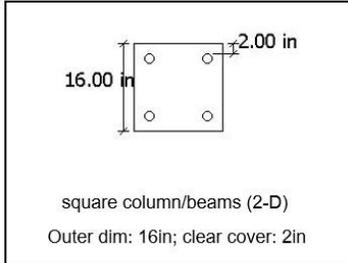
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 090206 L2

Description: Prestressed Pile\Highly Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 02/16/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concret Properties		0.38	Class F Fly Ash (46%); Silica Fume (7%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concret Properties	3.47E-9 in ² /sec	0.57	0.24 % wt. conc.	149.8 yrs	6 yrs	155.8 yrs
Measured Concrete Properties	-> 7.65E-9 in ² /sec	-> 0.57	-> 0.24 % wt. conc.	66.8 yrs	-> 6 yrs	72.8 yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

A.6.5 Materials and Tests Unit Statement of Concrete Mix Design

**North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials**

Project	Date Expires 12/31/2075
Mix Design Status Active	Concrete Producer S & G PRESTRESS CONCRETE
County	Plant Location & DOT No. LELAND, NC - 2
Resident Engr.	Contractor
Class of Concrete PRESTRESS	Date Assigned
Mix Design No. 2PVU13CIMSE	Contractor's Signature
Note Mix Design Units (English or Metric) ENGLISH	

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	ARGOS USA	BARRANQUILLA PLANT - (FORMERLY CEMENTC	665 lbs.
Pozzolan	Degussa Rheomac SF 100 silica fume		35 lbs.
Fine Aggregate	MARTIN MARIETTA	ROCKY POINT QUARRY - ROCKY POINT	1244 lbs.
Coarse Aggregate	MARTIN MARIETTA	GARNER QUARRY - GARNER	1700 lbs.
Total Water		WELL	29.4 gals.
Air. Entr. Agent	BASF CONSTRUCTION CHEMICALS, LLC	MASTERAIR AE 90 (AKA MB AE 90)	As recommended
Retarder	BASF CONSTRUCTION CHEMICALS, LLC	POZZOLITH 122R	As recommended
Water Reducer			
Superplasticizer	BASF CONSTRUCTION CHEMICALS, LLC	MASTERGLENIUM 3030 (AKA GLENIUM 3030NS)	As recommended
Corrosion Inhibitor	BASF CONSTRUCTION CHEMICALS, LLC	MASTERLIFE CI 30 (AKA RHEOCRETE CNI)	As recommended

Mix Properties and Specifications

Slump 7.00 in. **Mortar Content** 16.60 cu. ft.
Max Water 31.0 gals. **Air Content** 5.0 %

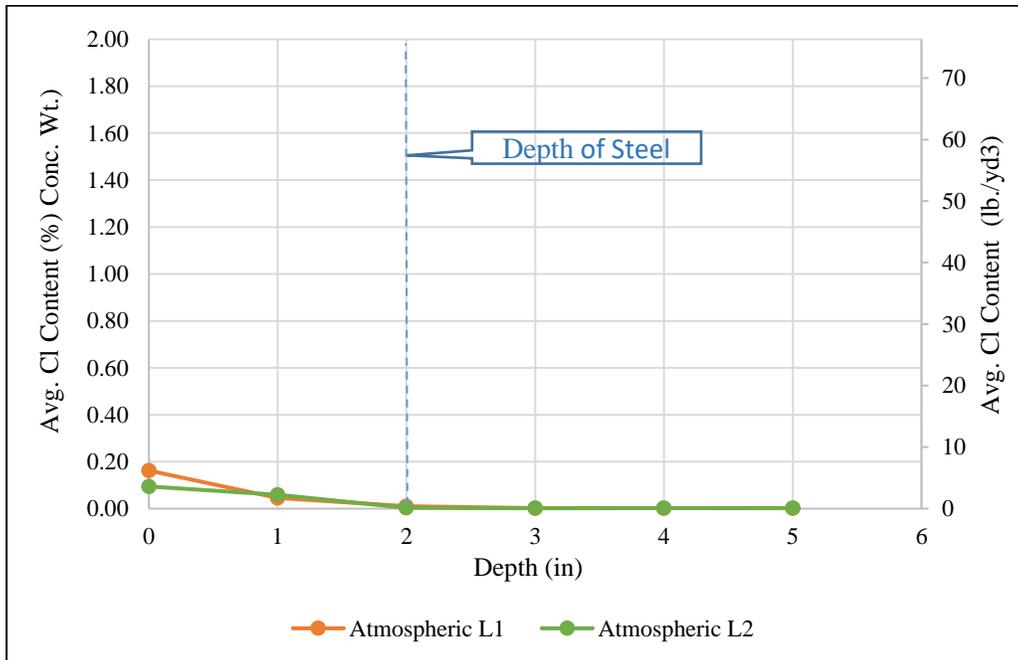
Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.65	1.0	NA	2.40
Coarse Aggregate, #67	2.62	0.5	93.5	NA

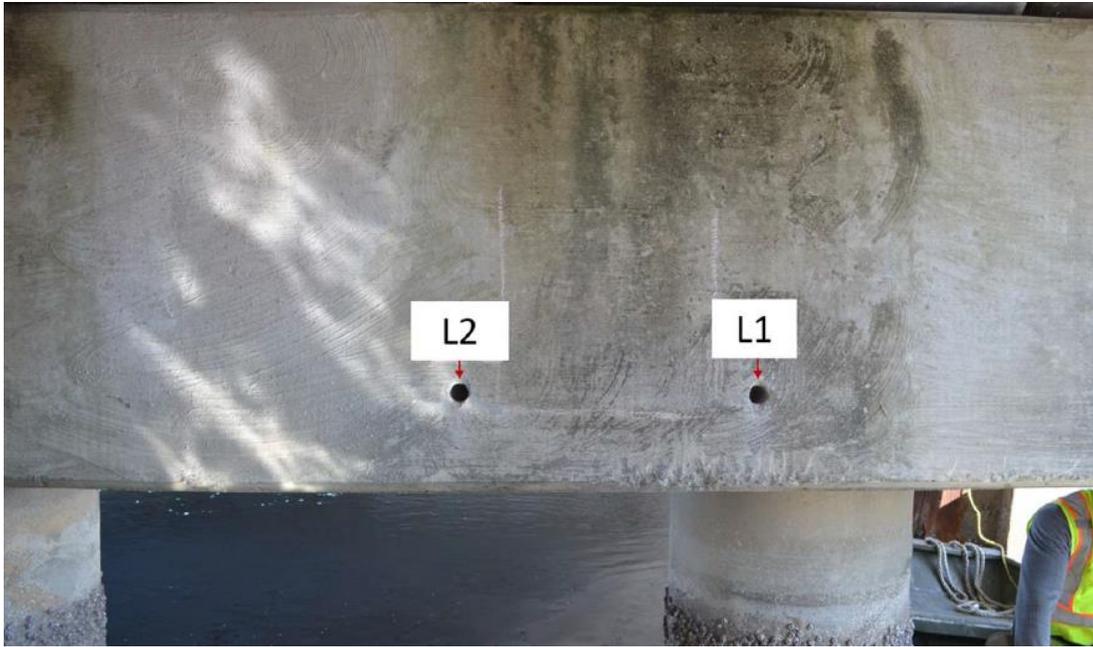
Comment Mix is designed for 6000 psi. Mix contains 3.0 gals. CNI and 35 lbs. Rheomac SF 100 silica fume. (Silica fume quantity is listed on Form 312 under "pozzolan.") Max. water-cement ratio is 0.4.

Cast-in-place concrete shall conform to Section 1000, precast concrete to Section 1077, and prestressed concrete to Section 1078 of the applicable edition of the Standard Specifications for Roads and Structures plus all applicable Special Provisions.

A.7.1 RCT Test Results (Chloride Content Profile)

Structure #	Location	Zone	Chloride Content		
			Depth (in)	% Conc. Wt.	Chlorides (lb/CY)
260007	L1	Atmospheric	0	0.162	6.189
			1	0.045	1.726
			2	0.010	0.400
			3	0.002	0.077
			4	0.002	0.072
	L2	Atmospheric	0	0.095	3.608
			1	0.059	2.250
			2	0.004	0.163
			3	0.002	0.064
			4	0.002	0.076
			5	0.002	0.065



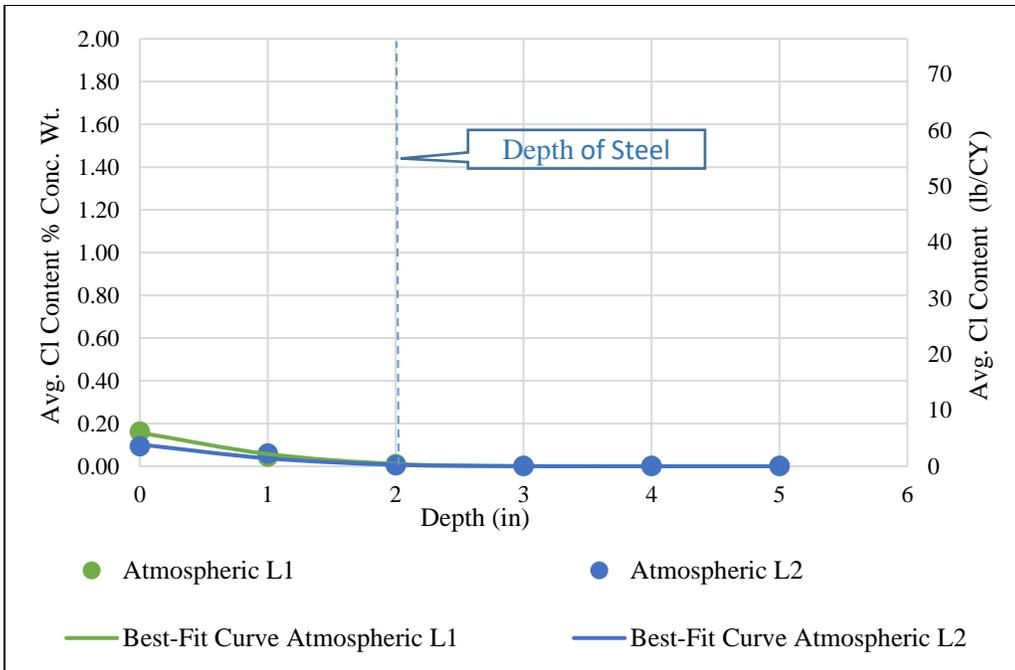


A.7.2 iCOR Test Results (Corrosion Rate & Surface Resistivity)

Structure #	Location	Atmospheric Zone				R2
		Corrosion Rate (Y)	Avg. Corrosion Rate	Concrete Resist. (Y)	Avg. Concrete Resist.	
260007	L1 & L2	19.0	22	125	149	0.99
		17.0		76		0.99
		9.2		88		1
		31.0		112		0.99
		40.0		95		0.98
		52.0		108		0.99
		46.0		67		0.99
		0.7		315		0.97
		18.0		170		0.95
		22.0		170		0.97
		4.6		210		0.98
		7.0		254		1

A.7.3 Model 2: Iterative Non-linear Least-Squares Regression

Slag / Highly Corrosive	Structure #	Location	Zone	Chloride Content			Age (t)	Model 2: Iterative Non-linear Least-Squares Regression			
				Depth (in) (x)	% Conc. Wt.	Chlorides (lb/CY)		Diffusion Coefficient (in*in/yr) (D)	Surface Concentration (lb/CY) (C _o)	Best-Fit Curve C _(x,t)	Residual Square
260007	L1	Atmospheric	0	0.162	6.1892	13	0.047	6.012	6.012	0.032	
			1	0.045	1.7263				2.206	0.230	
			2	0.010	0.4005				0.428	0.001	
			3	0.002	0.0767				0.041	0.001	
			4	0.002	0.0723				0.002	0.005	
			5	0.002	0.0686				0.000	0.005	
	L2	Atmospheric	0	0.095	3.6077	13	0.047	3.900	3.900	0.086	
			1	0.059	2.2495				1.431	0.670	
			2	0.004	0.1631				0.277	0.013	
			3	0.002	0.0637				0.026	0.001	
			4	0.002	0.0760				0.001	0.006	
			5	0.002	0.0654				0.000	0.004	



A.7.4 Corrosion Modeling Service Life Reports

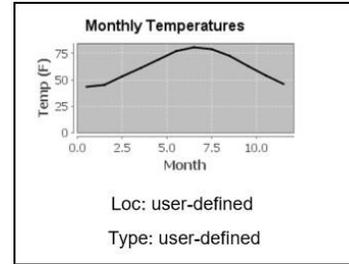
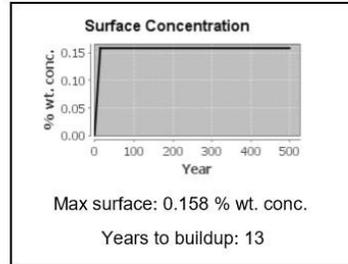
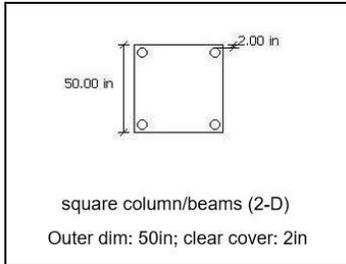
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 260007 L1

Description: Bent Cap\Highly Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 07/30/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concret Properties		0.35	Slag (0.4%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concret Properties	9.34E-9 in ² /in/sec	0.2	0.24 % wt. conc.	500+ yrs	6 yrs	506+ yrs
Measured Concrete Properties	-> 1.47E-8 in ² /in/sec	-> 0.45	-> 0.24 % wt. conc.	500+ yrs	-> 6 yrs	506+ yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

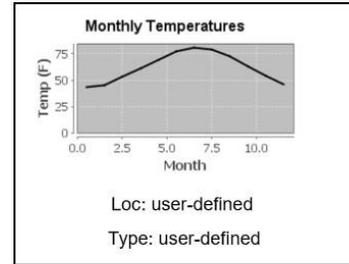
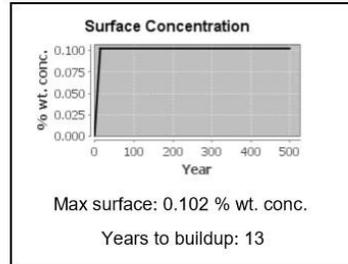
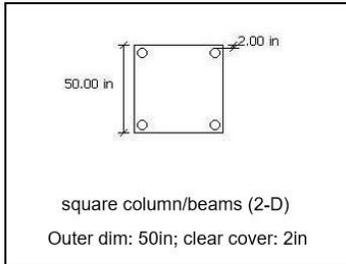
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 260007 L2

Description: Bent Cap\Highly Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 07/30/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concret Properties		0.35	Slag (0.4%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concret Properties	9.34E-9 in ² /in/sec	0.2	0.24 % wt. conc.	500+ yrs	6 yrs	506+ yrs
Measured Concrete Properties	-> 1.47E-8 in ² /in/sec	-> 0.45	-> 0.24 % wt. conc.	500+ yrs	-> 6 yrs	506+ yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

A.7.5 Materials and Tests Unit Statement of Concrete Mix Design

Form 312U
3-96

**North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials**

Project	Date Expires 12/31/2075
Mix Design Status Active	Concrete Producer COMMERCIAL READY MIX PRODUCTS
County	Plant Location & DOT No. MOYOCK, NC - 299
Resident Engr.	Contractor
Class of Concrete CLASS AA	Date Assigned
Mix Design No. 2992VGAACN1E	Contractor's Signature
Note Mix Design Units (English or Metric) ENGLISH	

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	ROANOKE CEMENT COMPANY	TROUTVILLE, VA (TYPE I/II)	480 lbs.
Pozzolan	LAFARGE	LAFARGE - SLAG	205 lbs.
Fine Aggregate	VULCAN MATERIALS CO.	CURLES NECK Q -- HENRICO CO. VA.	1161 lbs.
Coarse Aggregate	VULCAN MATERIALS CO.	LAWRENCEVILLE Q. - FREEMAN, VA	1756 lbs.
Total Water		WELL	30.0 gals.
Air. Entr. Agent	SIKA CORPORATION	SIKAAEA-15	As recommended
Retarder	SIKA CORPORATION	PLASTIMENT	As recommended
Water Reducer	SIKA CORPORATION	*SIKEMENT NL	As recommended
Superplasticizer			
Corrosion Inhibitor	SIKA CORPORATION	SIKA-CNI	As recommended

Mix Properties and Specifications

Slump 3.50 in. Mortar Content 16.54 cu. ft.
Max Water 35.0 gals. Air Content 6.0 %

Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.65	0.5	NA	2.79
Coarse Aggregate, #67	2.69	0.9	97.2	NA

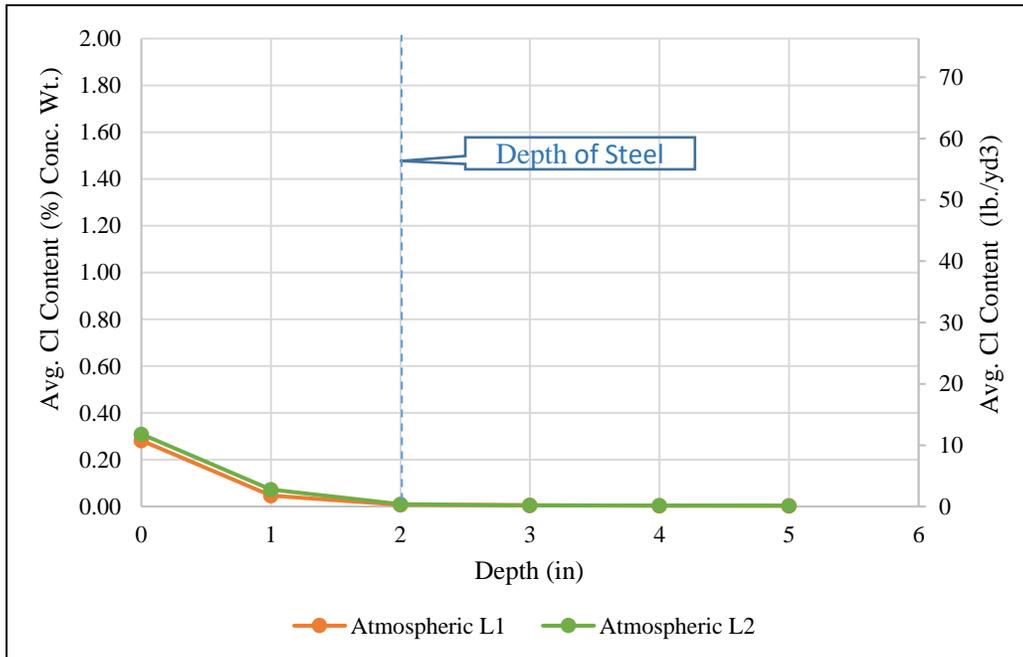
Comment Mix contains 3.0 gals. Sika CNI corrosion inhibitor.

Cast-in-place concrete shall conform to Section 1000, precast concrete to Section 1077, and prestressed concrete to Section 1078 of the applicable edition of the Standard Specifications for Roads and Structures plus all applicable Special Provisions.

A.8Structure 150020 Results Summary

A.8.1 RCT Test Results (Chloride Content Profile)

Structure #	Location	Zone	Chloride Content		
			Depth (in)	% Conc. Wt.	Chlorides (lb/CY)
150020	L1	Atmospheric	0	0.282	10.775
			1	0.046	1.767
			2	0.007	0.277
			3	0.006	0.237
			4	0.004	0.138
			5	0.003	0.119
	L2	Atmospheric	0	0.310	11.817
			1	0.073	2.781
			2	0.010	0.393
			3	0.004	0.165
			4	0.004	0.169
			5	0.004	0.150



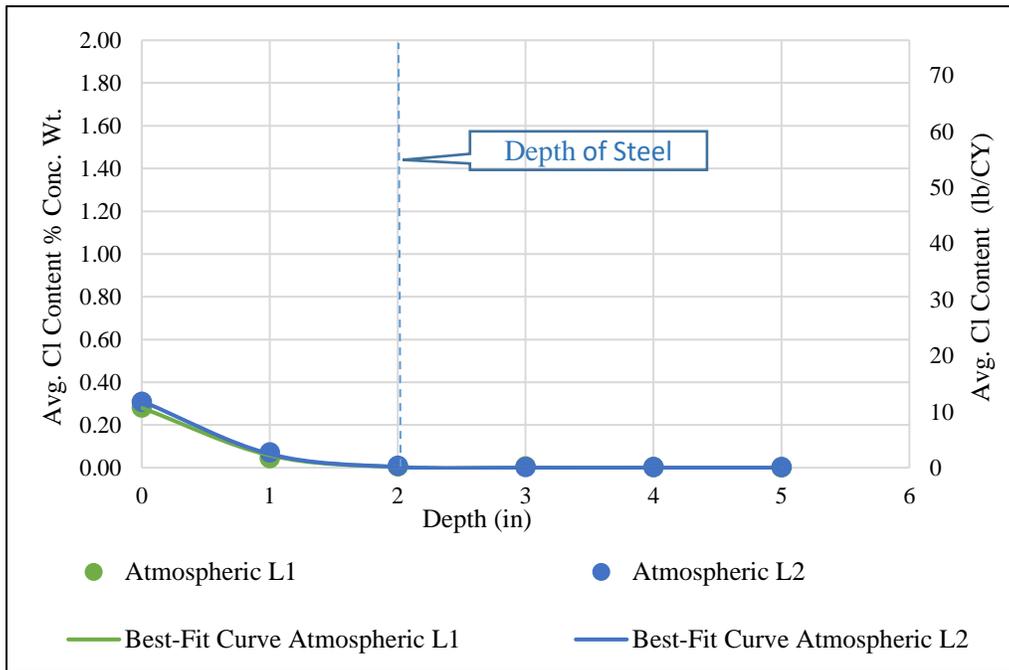


A.8.2 iCOR Test Results (Corrosion Rate & Surface Resistivity)

Structure #	Location	Atmospheric Zone				
		Corrosion Rate (Y)	Avg. Corrosion Rate	Concrete Resist. (Y)	Avg. Concrete Resist.	R2
150020	L1	0.45	4	477	267	0.94
		1.40		367		0.98
		1.40		332		0.87
		11.00		183		0.79
		6.40		108		0.89
		2.10		136		0.7
	L2	0.62	5	851	736	1
		4.00		681		0.94
		0.46		786		0.97
		11.00		498		0.97
		9.20		866		0.97
		9.40		739		0.93
		6.90		470		0.96
		3.50		549		1
		2.30		666		0.98
		1.40		407		0.9
		4.70		453		0.99
		2.10		565		0.91

A.8.3 Model 2: Iterative Non-linear Least-Squares Regression

Fly Ash/ Corrosive	Structure #	Location	Zone	Chloride Content			Age (t)	Model 2: Iterative Non-linear Least-Squares Regression			
				Depth (in) (x)	% Conc. Wt.	Chlorides (lb/CY)		Diffusion Coefficient (in*in/yr) (D)	Surface Concentration (lb/CY) (C _o)	Best-Fit Curve C _(x,t)	Residual Square
150020	L1	Atmospheric	0	0.282	10.775	12	0.026	10.686	10.686	0.008	
			1	0.046	1.767				2.209	0.195	
			2	0.007	0.277				0.124	0.024	
			3	0.006	0.237				0.002	0.056	
			4	0.004	0.138				0.000	0.019	
			5	0.003	0.119				0.000	0.014	
	L2	Atmospheric	0	0.310	11.817	12	0.026	11.887	11.887	0.005	
			1	0.073	2.781				2.457	0.105	
			2	0.010	0.393				0.137	0.065	
			3	0.004	0.165				0.002	0.027	
			4	0.004	0.169				0.000	0.028	
			5	0.004	0.150				0.000	0.023	



A.8.4 Corrosion Modeling Service Life Reports

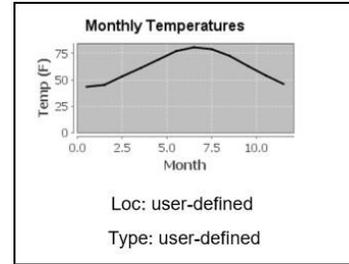
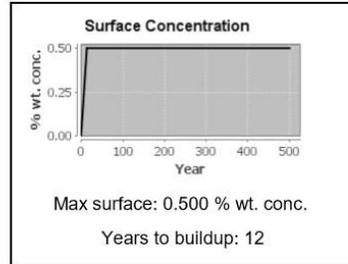
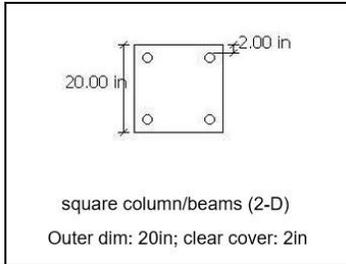
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 150020 L1

Description: Pressressed Pile\ Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 07/30/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.36	Class F Fly Ash (30%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concrete Properties	9.87E-9 in ² /in/sec	0.44	0.24 % wt. conc.	75.9 yrs	6 yrs	81.9 yrs
Measured Concrete Properties	-> 7.65E-9 in ² /in/sec	-> 0.44	-> 0.24 % wt. conc.	97.8 yrs	-> 6 yrs	103.8 yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

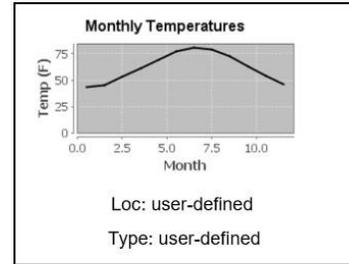
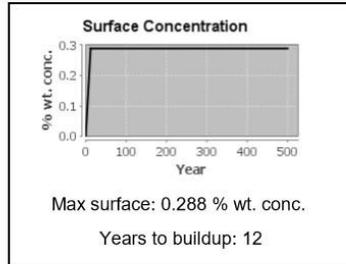
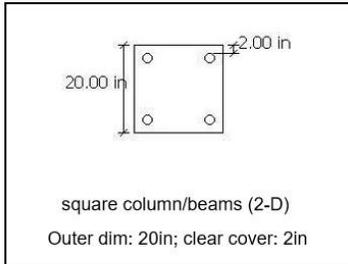
Life-365 v2.2 - Concrete Mixes and Service Lives

Project: 150020 L2

Description: Pressressed Pile /Corrosive Zone

Analyst: Taiseer Al Salihi

Date: 07/30/2021



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concret Properties		0.36	Class F Fly Ash (30%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Assumed Concret Properties	9.87E-9 in ² /in/sec	0.44	0.24 % wt. conc.	312.6 yrs	6 yrs	318.6 yrs
Measured Concrete Properties	-> 7.65E-9 in ² /in/sec	-> 0.44	-> 0.24 % wt. conc.	403 yrs	-> 6 yrs	409 yrs

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

A.8.5 Materials and Tests Unit Statement of Concrete Mix Design

Form 312U
3-96

North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials

Project	Date Expires 12/31/2075
Mix Design Status Active	Concrete Producer ATLANTIC METROCAST, INC.
County	Plant Location & DOT No. PORTSMOUTH, VA - 7
Resident Engr.	Contractor
Class of Concrete PRESTRESS	Date Assigned
Mix Design No. 7PVF8106E	Contractor's Signature
Note Mix Design Units (English or Metric) ENGLISH	

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	LEHIGH	UNION BRIDGE, MD	638 lbs.
Pozzolan	BORAL MATERIAL TECHNOLOGIES	BORAL - CHESAPEAKE	212 lbs.
Fine Aggregate	Heard, Waverly, VA		1078 lbs.
Coarse Aggregate	VULCAN MATERIALS CO.	SKIPPERS Q. - SKIPPERS, VA	1598 lbs.
Total Water		WELL	36.0 gals.
Air. Entr. Agent	SIKA CORPORATION	SIKA AEA-14	As recommended
Retarder	SIKA CORPORATION	PLASTIMENT	As recommended
Water Reducer			
Superplasticizer	SIKA CORPORATION	SIKA VISCOCRETE-4100	As recommended
Corrosion Inhibitor			

Mix Properties and Specifications

Slump 9.00 in. Mortar Content 17.37 cu. ft.
Max Water 45.9 gals. Air Content 5.0 %

Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.61	0.5	NA	2.94
Coarse Aggregate, #68 (VA Spec)	2.66	0.5	97.0	NA

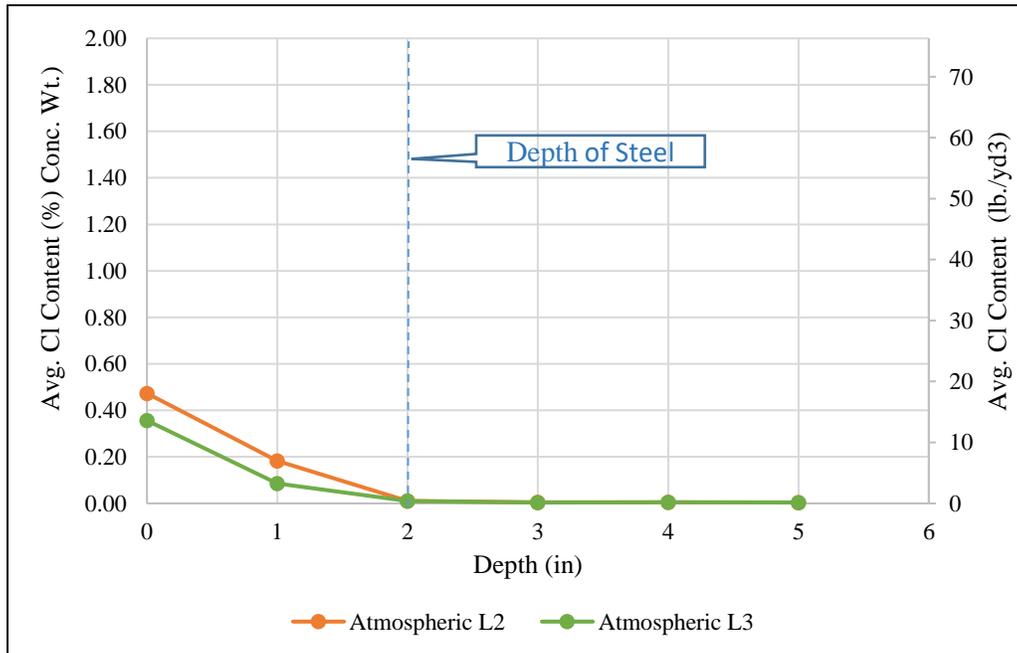
Comment Designed for 5500 psi. Cement is Type III.

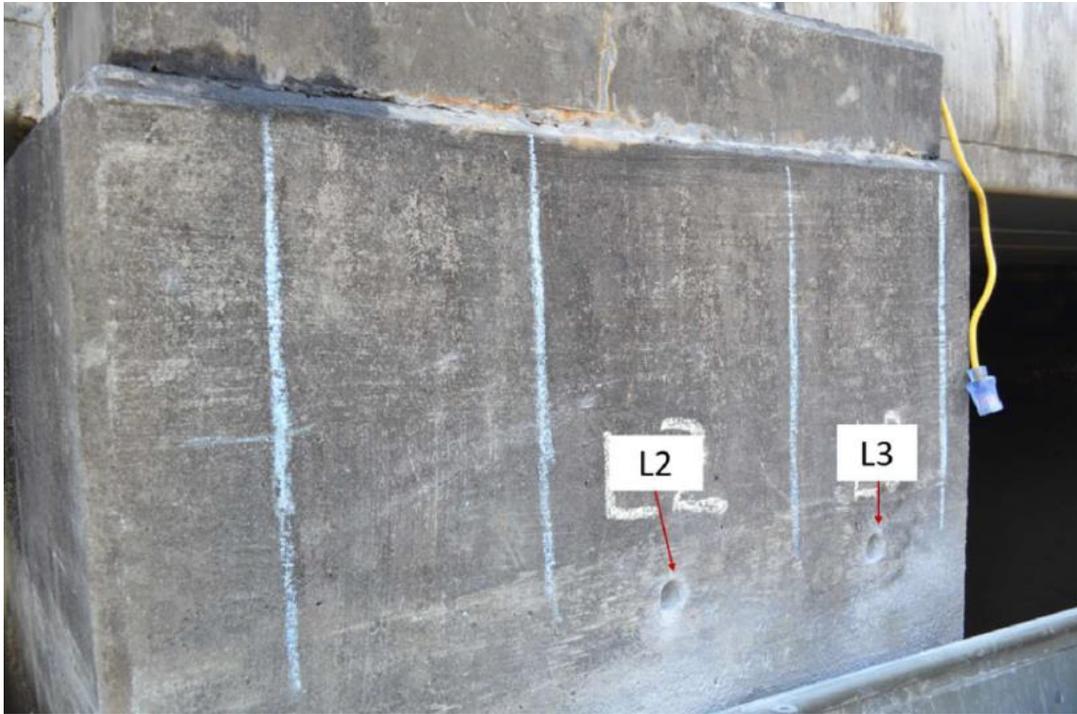
Cast-in-place concrete shall conform to Section 1000, precast concrete to Section 1077, and prestressed concrete to Section 1078 of the applicable edition of the Standard Specifications for Roads and Structures plus all applicable Special Provisions.

A.9 Structure 150026 Results Summary

A.9.1 RCT Test Results (Chloride Content Profile)

Structure #	Location	Zone	Chloride Content		
			Depth (in)	% Conc. Wt.	Chlorides (lb/CY)
150026	L2	Atmospheric	0	0.473	18.062
			1	0.183	6.976
			2	0.010	0.390
			3	0.005	0.181
			4	0.004	0.165
	L3	Atmospheric	0	0.357	13.603
			1	0.085	3.241
			2	0.011	0.401
			3	0.004	0.153
			4	0.004	0.168
			5	0.004	0.151



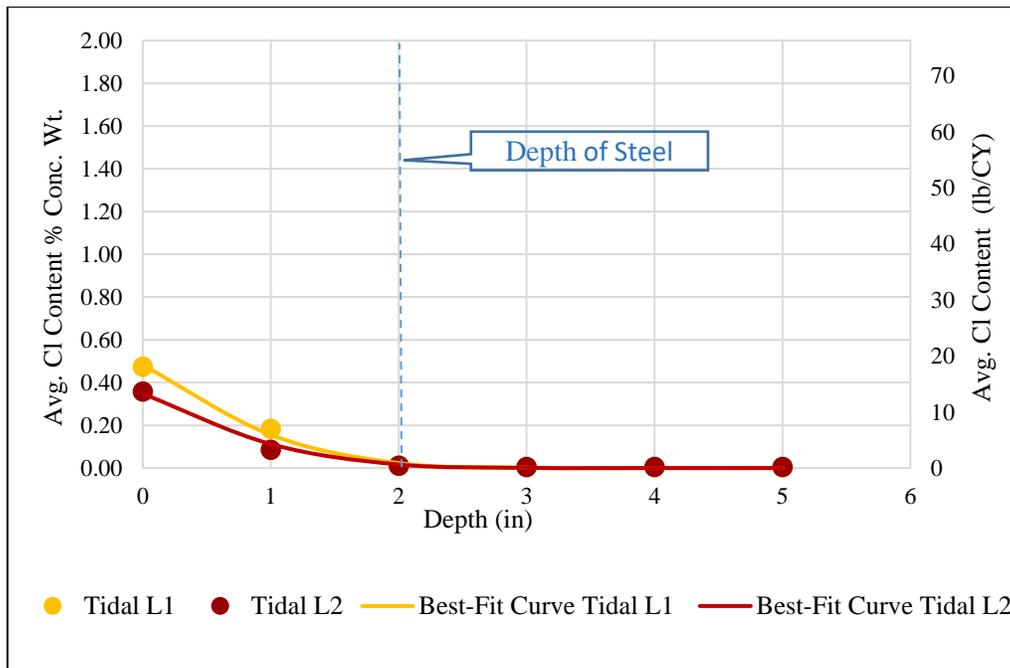


A.9.2 iCOR Test Results (Corrosion Rate & Surface Resistivity)

Structure #	Location	Atmospheric Zone				
		Corrosion Rate (Y)	Avg. Corrosion Rate	Concrete Resist. (Y)	Avg. Concrete Resist.	R2
150026	L2 & L3	1.2	6	745	433	0.99
		1.4		402		0.99
		1.0		400		0.99
		0.6		555		0.99
		0.6		775		1
		5.3		397		0.99
		2.6		379		0.99
		0.1		469		1
		2.5		309		1
		21.0		246		0.96
		28.0		199		0.94
		4.1		316		0.96

A.9.3 Model 2: Iterative Non-linear Least-Squares Regression

Fly Ash / Highly Corrosive	Structure #	Location	Zone	Chloride Content			Age (t)	Model 2: Iterative Non-linear Least-Squares Regression			
				Depth (in) (x)	% Conc. Wt.	Chlorides (lb/CY)		Diffusion Coefficient (in*in/yr) (D)	Surface Concentration (lb/CY) (C _o)	Best-Fit Curve C _(x,t)	Residual Square
150026	L2	Tidal	0	0.473	18.0619	14	0.037	18.375	18.375	0.098	
			1	0.183	6.9761				5.933	1.087	
			2	0.010	0.3903				0.883	0.243	
			3	0.005	0.1807				0.056	0.016	
			4	0.004	0.1648				0.001	0.027	
			5	0.004	0.1419				0.000	0.020	
	L3	Tidal	0	0.357	13.6030	14	0.037	13.257	13.257	0.120	
			1	0.085	3.2413				4.281	1.080	
			2	0.011	0.4011				0.637	0.056	
			3	0.004	0.1530				0.040	0.013	
			4	0.004	0.1683				0.001	0.028	
			5	0.004	0.1508				0.000	0.023	



A.9.4 Corrosion Modeling Service Life Reports

A.9.5 Materials and Tests Unit Statement of Concrete Mix Design

Form 312U
3-96

**North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials**

Project	Date Expires 12/31/2075
Mix Design Status Active	Concrete Producer S.T. WOOTEN CORP.
County	Plant Location & DOT No. NEWPORT, NC - 316
Resident Engr.	Contractor
Class of Concrete CLASS AA	Date Assigned
Mix Design No. 3162VFG5634CCIE	Contractor's Signature
Note Mix Design Units (English or Metric) ENGLISH	

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	CEMEX	KNOXVILLE, TN	560 lbs.
Pozzolan	SOUTHEASTERN FLY ASH	SEFA - WINYAH GENERATING STATION/GEORG	168 lbs.
Fine Aggregate	MARTIN MARIETTA	BELGRADE QUARRY - MAYSVILLE	1076 lbs.
Coarse Aggregate	MARTIN MARIETTA	CLARKS QUARRY - NEW BERN	1570 lbs.
Total Water		WELL	31.0 gals.
Air. Entr. Agent	W.R. GRACE & COMPANY	DAREX AEA	As recommended
Retarder	W.R. GRACE & COMPANY	DARATARD 17	As recommended
Water Reducer	W.R. GRACE & COMPANY	WRDA-35	As recommended
Superplasticizer			
Corrosion Inhibitor	W.R. GRACE & COMPANY	DCI S	As recommended

Mix Properties and Specifications

Slump 3.50 in. **Mortar Content** 16.34 cu. ft.
Max Water 33.2 gals. **Air Content** 6.0 %

Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.62	0.4	NA	2.87
Coarse Aggregate, #67	2.36	5.3	79.0	NA

Comment Contains 3 gals DCI-S corrosion inhibitor. Maximum allowable water has been reduced to account for water (2.6 gals) in DCI-S.

Cast-in-place concrete shall conform to Section 1000, precast concrete to Section 1077, and prestressed concrete to Section 1078 of the applicable edition of the Standard Specifications for Roads and Structures plus all applicable Special Provisions.

Appendix B – Corrosion Inhibitor Measurement

Corrosion inhibitor is dosed into the concrete mixture in concentrations that depend on the mixture design and anticipated chloride exposure from the environment. Once concrete has hardened, this concentration of corrosion inhibitor can be determined or verified from powder samples removed from the hardened concrete. This technique is often performed as a quality assurance tool to ensure the correct dosages were utilized and that the inhibitor was thoroughly mixed into the concrete. The method utilized for determining the corrosion inhibitor concentration in the bridge decks was adopted from the W. R. Grace chemical procedure #C-20.0 for Determination of Nitrite in Hardened Concrete (Jeknavorian 2005). The procedure can be divided into three parts: preparation of a standard calibration curve, sample extraction and preparation, and calculation.

The apparatus necessary to conduct this test includes equipment as well as reagents. The required equipment includes a hammer drill with $\frac{3}{4}$ inch bit, an analytical balance (accurate to ± 0.1 mg), a laboratory shaker, 500mL Erlenmeyer flasks, pipettes (Class A), volumetric flasks (100mL, 500mL, and 1000mL), graduated cylinders (50mL and 100mL), a funnel, No. 44 filter paper, a spectrophotometer with scanning capabilities, and glass cuvettes. The reagents required for sample extraction and preparation are sulfanilic acid, N-(1-Naphthyl) Ethylenediamine Dihydrochloride (NED), sodium nitrite, phenolphthalein indicator solution (1%), and hydrochloric acid (1N).

The first part of the procedure involves the preparation of a standard calibration curve. The curve consists of the measured absorbance readings from three sodium nitrite standard solutions. Creating the primary standard sodium nitrite solution was done by dissolving 2.8 grams of sodium nitrite in a volumetric flask containing one liter of distilled water. Using this primary standard solution, it was diluted to 50/500 by adding 50mL of the primary solution to 500mL of distilled water in a volumetric flask. With the secondary standard solution three separate solutions were prepared in 500mL volumetric flasks. One contained 5mL standard solution per 500mL of distilled water, another contained 10mL standard solution per 500mL of distilled water, and a final one contained 15mL standard solution per 500mL of distilled water. These standard solutions have nitrite concentrations of 0.187 microgram/mL, 0.373 microgram/mL, and 0.560 microgram/mL respectively. Using a pipette 10mL of each of these solutions was put into separate 100mL volumetric flasks containing 100mL of distilled water. A fourth volumetric flask was also prepared as a blank that only contains 100mL of distilled water and no secondary solution. For each 100mL flask 2mL of sulfanilic acid was added and mixed by swirling. After the sulfanilic acid was introduced the flasks were allowed to sit for five minutes. Then, 2mL of NED reagent were added and allowed to sit for ten minutes before diluting to the desired volume of 100mL. The preparation of the standard samples was then completed, and their absorbance was determined by utilizing a spectrophotometer. The spectrophotometer was zeroed before being set to a wavelength of 540nm. The samples were loaded into the spectrophotometer one at a time and their absorbance was measured. With the absorbance measurements and the known nitrite concentration a calibration curve was generated in $\mu\text{g/mL}$ nitrite vs. absorbance using a graphing software. A linear trend line was fitted to the points of the calibration curve and its equation was determined in the form of $y=mx+b$ where y was equal to absorbance and x was equal to concentration in micrograms/mL.

The second part of the procedure involves sample preparation, extraction, and nitrite determination. The powder samples acquired for analysis were taken from the powders that were also used for chloride concentration measurements. The powder was ground and pulverized until a uniform consistency was achieved. Using an analytical balance that is accurate to ± 0.1 gram, a 2.0-gram sample was weighed and added to a 500mL Erlenmeyer flask containing 200mL of distilled water. The sample and distilled water in the flask were agitated using a laboratory shaker for 30 minutes. After agitation, the liquid was decanted through a #44 filter paper into a 500mL volumetric flask. The residue from the concrete powder was left in the Erlenmeyer flask after filtering. The filtering process was repeated by adding another 200mL of distilled water to the Erlenmeyer flask containing the residue. However, this time it was only agitated for 10 minutes before being filtered into the same volumetric flask as before. A third and final filtration was completed by adding another 75mL of distilled water to the Erlenmeyer flask containing the residue. It was then agitated for 10 minutes before being filtered into the volumetric flask for the final time. After these three filtered extractions were completed the 500mL volumetric flask (containing 475mL of extracted fluid) was diluted with 25mL of distilled water (until the 500mL line) and mixed. From the volumetric flask containing the extracted liquid, 3mL of the liquid were transferred into a 100mL volumetric flask containing 50mL of distilled water. Two drops of phenolphthalein indicator were added before the solution was neutralized with two drops of 1N HCl. An additional 2.0mL of sulfanilic acid were added by pipette before swirling and allowing to sit for five minutes. A reagent blank was prepared by mixing 50mL of distilled water with the reagents in the same quantities as in the prior step. After the solutions were left to stand for five minutes, 2.0mL of NED was added, and the flask was diluted to volume before being allowed to stand for ten minutes. Using the reagent blank, the spectrophotometer was zeroed in the absorbance mode. The sample can then be tested for absorbance at a wavelength of 540nm using a glass cuvette. With the

absorbance measurement completed, the concentration was calculated with use of the calibration curve equation generated in part one of this procedure.

The third part of the procedure involved reportioning the concentration from micrograms/mL into the more commonly utilized unit of lbs/yd³ of concrete. To make this conversion the unit weight of concrete was needed. Since the powder samples were taken from an existing structure and the unit weight was not known, an estimated value of 141.6 lbs/ft³ for normal weight concrete was utilized (Cavalline et al. 2013). The fraction of nitrite in the sample was then calculated as:

$$\text{Fraction of } NO_2^- = \frac{C \times (100/3) \times 500mL}{w \times 10^6}$$

where C = Concentration of nitrite in the sample,
 w = Weight of sample in grams.

In the previous equation, 100/3 represents a 3:100 dilution that was made with the reagents to extract the sample. After utilizing this equation to calculate the fraction of nitrite, the unit weight of the concrete in cubic yards was calculated by multiplying the unit weight of the concrete (γ) in lbs/ft³ by the number of cubic feet in a cubic yard (27ft³):

$$\gamma = 141.6 \text{ lb/ft}^3 \times 27\text{ft}^3$$

Using the unit weight and fraction of nitrite calculated from the previous equations the concentration of nitrite in of lbs/yd³ of concrete was calculated by multiplying the unit weight by the fraction of nitrite:

$$\text{Concentration (lb/yd}^3) = \gamma \times \text{Fraction of } NO_2^-$$

The NCDOT specifies calcium nitrite corrosion inhibitor be added in a minimum dose of 3 gal/CY. To verify that the correct dosages of corrosion inhibitor were utilized the nitrite concentration in lb/CY was converted to gal/CY with the following equation:

$$\text{Concentration (gal/yd}^3) = \frac{\text{Nitrite Concentration in lb/yd}^3}{0.209 \times 1.3 \times 8.33\text{lbs/gal}}$$

where 0.209 = Decimal percentage of NO₂⁻ in a corrosion inhibitor on average,
 1.3 = Specific Gravity of CI,
 8.33 lbs/gal = Weight of water per gallon.

The NCDOT corrosion policy currently in place specifies a minimum dosage rate of calcium nitrite corrosion inhibitor for all concrete bridge elements within the corrosive and highly corrosive zones. This minimum dosage rate is 3.0 gal/CY. It is important to this study to verify that this minimum dosage rate is in fact being utilized in the concrete mixtures and that the admixture is being evenly dispersed throughout the concrete. It is of interest to verify that the minimum dosage rate is being met at the outermost edge of the reinforcing steel as this is where chloride induced corrosion is most likely to occur. To verify that the minimum dosage rates and uniform dispersion of the calcium nitrite admixture are achieved, powder samples from two different depths at one location from each bridge were selected for testing to determine the concentration of corrosion inhibitor. Each bridge location selected included a test of the powder taken from two inches (which is the depth of the outermost steel) and of the powder taken at the deepest depth sampled (which in most cases was five inches).

To compute the concentration of calcium nitrite in each powder sample a calibration curve was developed with four standard samples. These standard samples were prepared containing known concentrations of calcium nitrite ranging from 0.000 mg/mL to 0.560 mg/mL following the procedure outlined in Chapter 3.3.2 of this thesis. Testing the absorbance of these standard samples with a spectrophotometer allowed for a linear trendline to be plotted between calcium nitrite concentration and absorbance. The equation of the resulting trend line was $y=0.7582x+0.0945$ and had an R² value of 0.9988. This equation was utilized in the determination of nitrite concentration of all powder samples with use of absorbance measurements from the spectrophotometer. The measured absorbance values of the standard samples can be seen in Table B1 and the resulting calibration curve is shown in Figure B1.

Table B1: Concentration and Absorbance of Standard Samples

Standard Sample	Concentration (mg/mL)	Absorbance (AU)
0	0.000	0.0894
1	0.187	0.2408
2	0.373	0.3837
3	0.560	0.5134

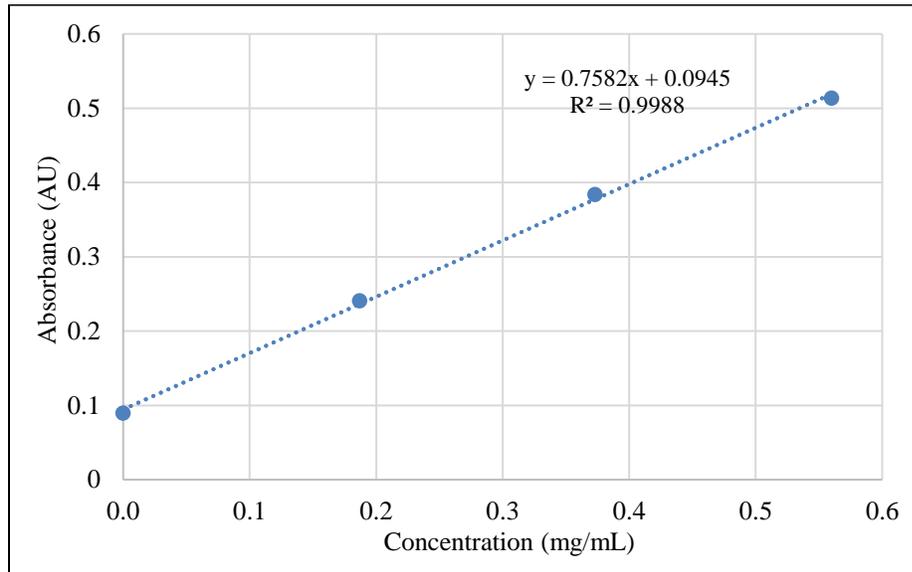


Figure B1: Calibration curve for calcium nitrite determination

Utilizing the equation of the calibration curve, two powder samples from each selected bridge location were evaluated with the spectrophotometer. Information about the depths of the powder samples utilized and the results of this testing can be seen in Table B2.

Table B2: Laboratory measured absorbance and concentration of powder samples

Bridge-Location	Sample Depth (in)	Absorbance (AU)	Calculated Concentration (mg/mL)
150020-L2	2	0.2555	0.212
	5	0.2572	0.215
660019-L1	2	0.2897	0.257
	3	0.3252	0.304
260007-L1	2	0.2636	0.223
	5	0.1628	0.09
640010-L1	2	0.2229	0.169
	5	0.2475	0.202
660021-L1	2	0.3028	0.275
	5	0.2669	0.227
090061-L1	2	0.1998	0.139
	5	0.2587	0.217
150026-L2	2	0.2448	0.198
	5	0.2628	0.222

Appendix C: Corrosion Policies of Other Coastal States

Review of Coastal States Corrosion Policies

To summarize the state-of-the-art in terms of corrosion prevention policies, the bridge design manuals of transportation departments in coastal states were reviewed. The states included were Alabama, California, Connecticut, Delaware, Florida, Georgia, Louisiana, Maine, Massachusetts, Mississippi, New Jersey, New York, North Carolina, Oregon, Rhode Island, South Carolina, Texas, Virginia, and Washington.

This section is divided into two subsections. The first subsection presents Table C1 that summarizes common aspects of corrosion policies, like cover requirements or admixture specifications. The second subsection includes a more in-depth description of a select few states that employ more advanced corrosion related policies. The states that are discussed here include California, New Jersey, and Delaware.

Table C1: Comparison of Coastal States' Corrosion Policies.

State	Alabama	California	Connecticut	Delaware	Florida
Definition of Corrosive Zone	-	Based on ppm Cl (>500 ppm is corrosive)	-	Corrosive environments [2]	Based on distance from water and ppm Cl (Multiple zones)
Top of Deck Concrete Cover (in.)	2 [1]	2.5	AASHTO, unless noted	3 (coastal regions)	2 (bridges <100') 2.5 (bridges >100')
Required Steel Type	-	Engineers Discretion	Epoxy required on certain elements (deck)	Epoxy Coating	Explicitly stated NOT to use epoxy coating
Corrosion Inhibitor	-	-	-	-	Engineers Discretion
Corrosive Water	-	>500 ppm Cl	-	-	>2000 ppm Cl - Marine structure (varies by amount and location)
Expected Service Life (Years)	-	75	-	Existing 75 New 100	-
Common Admixtures	-	ASTM C618 Type F or N (Fly ash)	-	-	Fly ash, slag, silica fume, metakaolin
Chloride Testing	-	-	-	AASHTO C856, T-24, & T260, ASTM C876 & C1202, half-cell test	-

[-] No mention in appropriate DOT design manual.

[1] Not explicitly defined for corrosion prevention.

[2] Not explicitly defined in appropriate DOT design manual.

[3] No distinction between bridges in areas of high vulnerability to corrosion and those in areas of low vulnerability.

[4] Provision is defined more for deicing operations than seawater corrosion prevention.

Table C1: Comparison of Coastal States' Corrosion Policies (continued).

State	Georgia	Louisiana	Maine	Massachusetts	Mississippi
Definition of Corrosive Zone	Coastal Counties	Coastal splash zones, deicing areas, history of corrosion	All salt-water crossings	-	-
Top of Deck Concrete Cover (in.)	2 (Above Fall Line) 2.25 (Below F.L.) ^[4]	2 ^[1]	2 (unless noted) ^[1]	-	-
Required Steel Type	Epoxy coating for top of deck steel	Epoxy coating in Divisions using deicing agents ^[4]	Engineers Discretion (recommend not using epoxy)	Epoxy coating or galvanized ^[1,2]	-
Corrosion Inhibitor	-	Calcium Nitrite (0.0748 - 0.2245 gal/ft ³)	Calcium Nitrite in PSC units (5.5 gal/yd ³)	-	-
Corrosive Water	-	-	-	-	-
Expected Service Life (Years)	-	-	-	New, 75	-
Common Admixtures	-	Silica fume, fly ash	Silica fume in class LP (low permeability) concrete	-	-
Chloride Testing	-	-	Cl Content at 1in intervals starting at ½in depth	Cl Core Analysis	-

[-] No mention in appropriate DOT design manual.

[1] Not explicitly defined for corrosion prevention.

[2] Not explicitly defined in appropriate DOT design manual.

[3] No distinction between bridges in areas of high vulnerability to corrosion and those in areas of low vulnerability.

[4] Provision is defined more for deicing operations than seawater corrosion prevention.

Table C1: Comparison of Coastal States' Corrosion Policies (continued).

State	New Jersey	New York	North Carolina	Oregon	Rhode Island
Definition of Corrosive Zone	Zones 1, 2, 3A, 3B	-	Two corrosive boundary lines	Marine Environment (distance from ocean, nearby geography)	Designated coastal environments [2]
Top of Deck Concrete Cover (in.)	2.5	2.25	2.5 [3]	2.5 (minimum of 2 in Marine)	2 (wearing surface), 3 (exposed deck) [3]
Required Steel Type	Epoxy coating for all deck steel	Epoxy, stainless steel, or galvanized for all deck steel	Epoxy coating	Stainless Steel (Epoxy not permitted)	Epoxy Coating [3]
Corrosion Inhibitor	-	Calcium Nitrite in PSC	Calcium Nitrite	-	-
Corrosive Water	-	-	-	-	-
Expected Service Life (Years)	100	100 for stainless steel, 50-75 for chromium steel, 40 for galvanized or epoxy, 20 for plain	100	-	-
Common Admixtures	-	-	Fly ash, silica fume, granulated blast furnace slag	Microsilica	Engineers Discretion [2]
Chloride Testing	Cl Analysis, half-cell test, Cl permeability	-	-	AASHTO T260, Cl Core Analysis, ASTM C1152	-

[-] No mention in appropriate DOT design manual.

[1] Not explicitly defined for corrosion prevention.

[2] Not explicitly defined in appropriate DOT design manual.

[3] No distinction between bridges in areas of high vulnerability to corrosion and those in areas of low vulnerability.

[4] Provision is defined more for deicing operations than seawater corrosion prevention.

Table C1: Comparison of Coastal States' Corrosion Policies (continued).

State	South Carolina	Texas	Virginia	Washington
Definition of Corrosive Zone	Coastal Counties (NW part of state for deicing)	District-specific	Corrosive environment, Marine ^[2]	Coastal/corrosive environments ^[2]
Top of Deck Concrete Cover (in.)	2.5 ^[3]	2.5 ^[4]	2.5	AASHTO, unless noted
Required Steel Type	-	Epoxy coating or galvanized. Stainless steel for severe exposure	References to current IIM-S&B-81). Epoxy or galvanized not permitted	Epoxy coating
Corrosion Inhibitor	-	-	Apply to prestressed tendons if approved	-
Corrosive Water	-	-	-	-
Expected Service Life (Years)	-	-	-	-
Common Admixtures	-	-	Not specified for concrete	-
Chloride Testing	Cl Analysis	-	Not specified for concrete	-

[-] No mention in appropriate DOT design manual.

[1] Not explicitly defined for corrosion prevention.

[2] Not explicitly defined in appropriate DOT design manual.

[3] No distinction between bridges in areas of high vulnerability to corrosion and those in areas of low vulnerability.

[4] Provision is defined more for deicing operations than seawater corrosion prevention.

California is distinguished due to its strict and specific policies related to corrosion protection and prevention. The focus of this section is on the specifications related to minimum concrete cover.

California employs the use of a very detailed table to determine what the minimum concrete cover thickness is to be for a particular project in order to attain a service life of 75 years. The table, which is shown on the following page in Table C2, is organized by rows displaying the structural element of interest and columns displaying the minimum cover thickness based on the exposure condition. The following paragraph defines the terms used in the table to better understand what is being displayed.

The following definitions are all paraphrased from California's DOT bridge design specifications (Caltrans 2003). Marine atmosphere includes the atmosphere over land that is within 1,000 feet of ocean water or tidal water and the atmosphere directly above the splash zone. Tidal water is defined as being any body of water with a chloride content of at least 500 parts-per-million (ppm). 500 ppm is also the minimum threshold value defined for corrosive water. The splash zone is described as being the region between the Mean Lower Low Water (MLLW) elevation and up to 20 feet above the Mean Higher High Water (MHHW) elevation as well as a horizontal distance of 20 feet from the edge of the water (Caltrans 2003). Additionally, it is noted that for structural elements in direct contact with ocean spray the concrete cover shall be determined based on the requirements for a chloride concentration of greater than 10,000 ppm in the corrosive splash zone.

Table C2. Minimum Concrete Cover for 75-year Design Life.
(Originally Table 8.22.1 in Caltrans Bridge Design Specifications (Caltrans 2003))

	Exposure condition							Deicing salt, snow run-off, or snow blower spray (a),(c),(e)		
	Non-corrosive Atmosphere/soil/water	Marine Atmosphere	Corrosive soil above MLLW level		Corrosive soil below MLLW level (a)	Corrosive water permanently below MLLW level (a),(b)	Corrosive splash zone			
			Chloride Concentration (ppm)				Chloride concentration (ppm)		Greater than 10,000 (a),(b)	
			500 – 5,000 (a)	5,001 – 10,000 (a)						
Footings & pile caps	3	3	3	4	5	3	2	3	3.5	2.5
Walls, columns & cast-in-place piles	2	3	3	4	5	3	2	3	3.5	2.5
Precast piles and pile extensions	2	2 ^(d)	2 ^(d)	2 ^{(b),(d)}	3 ^{(b),(d)}	2 ^(d)	2	2	2.5 ^(d)	2 ^(d)
Top surface of deck slabs	2	2.5					2.5	2.5	2.5 ^(d)	2.5
Bottom surface of deck slabs	1.5	1.5					2	2.5	2.5 ^(d)	2.5
Bottom slab of box girders	1.5	1.5					2	2.5	2.5 ^(d)	1.5
Cast-in-place "I" and "T" girders; cast exposed faces of box-girder webs, bent caps, diaphragms, and hinged joints (f)	1.5	3					2	2.5	2.5 ^(d)	3
Curbs & railings	1	1 ^(b)					1	1	1 ^(d)	1
Concrete surface not exposed to weather, soil or water		Principal reinforcement: 1.5 inches Stirrups, ties and spirals: 1.0 inch								

General Notes: 1. Mineral admixtures conforming to ASTM Designation C618 Type F or N, are required for all exposure conditions, except for 'non-corrosive' exposure conditions.
2. For protection of bundled bars, ducts and /or prestressing steel, see Articles 8.22.2, 8.22.3 and 8.22.4.
3. The minimum cover at the corners, beveled edges, and curved surfaces shall be the same as that in the corresponding structural elements.

Footnotes: (a) The maximum water to cementitious material ratio shall not exceed 0.40.
(b) Use pre-fabricated epoxy coated reinforcing bars (ECR).
(c) Use post-fabricated ECR.
(d) Mineral admixtures conforming to ASTM Designation C1240 and/or ASTM Designation C618 Type F and/or N, may be required.
(e) The minimum concrete cover and other requirements in structural elements exposed to de-icing salt, snow/run-off, or snow blower spray shall be adopted only where the structural elements are directly exposed to these corrosive conditions, otherwise the requirements specified for non-corrosive conditions shall be adopted.
(f) For precast "I" and "T" girders, the minimum cover may be reduced (depending on site conditions).

Additionally, as shown in the general notes in Table C2, it is specified that mineral admixtures following ASTM C618 Type F or N are required for all exposure conditions, with the exception of non-corrosive conditions. Type F and N are both referring to a class of fly ash as designated within ASTM C618 (ASTM 2019). This means that all structures meeting any of the requirements to be considered within a corrosive area are required to include fly ash as a protective measure.

New Jersey DOT Bridge Design Manual

The New Jersey Department of Transportation (NJDOT) is distinguished from other states' corrosion related policies in that they provide a very detailed description of analyzing the extent of corrosion and defining what steps to take upon identification of corrosion related damage. This section will focus on the process utilized by the NJDOT to identify and remediate corrosion damage on bridge decks.

The field survey performed by the NJDOT includes visual observations, delamination detection, concrete sampling for chloride testing, half-cell potential testing, and a pachometer survey. This testing strategy is used to determine existing defects and areas of bridge decks that are actively deteriorating. The combined results of each section are then used to assist engineers in evaluating the current condition of a bridge deck. The following subsections provide a brief description of each step of the analysis.

The visual survey is used to identify the extent of damage such as spalling or cracking. The extent of spalling is generally reported as a percentage of the total deck area. The information gained from the visual survey is then used to determine specific areas of the bridge that may require additional investigation or testing. In addition to identifying locations that are spalling or cracking, a delamination survey is also performed to determine the subsurface condition of concrete bridge decks. This can be completed by either performing a chain drag or using a ground penetrating radar (GPR) (NJDOT 2016).

A chloride analysis is performed to provide a quantitative measure of the chloride levels within the concrete at varying depths. The threshold chloride content, or amount needed for corrosion to initiate, that the NJDOT defines is approximately 2 pounds of chloride per cubic yard of concrete (NJDOT 2016). The number of samples from each bridge should be randomly selected using statistical methods and the locations should be plotted on a plan view of the deck. The minimum requirement is that there be at least 10 locations tested for every 6,000 square yards of deck area (NJDOT 2016).

The half-cell potential test is performed to determine areas of the deck where there is active corrosion. The NJDOT Design Manual defines the following ranges for half-cell potential readings: a potential difference more negative than -0.35 volts (V) indicates a high probability of active corrosion; potential readings between -0.35 V and -0.20 V indicate the possibility of active corrosion; potential readings less negative than -0.20 V indicate the probability of inactive or no corrosion (NJDOT 2016). Additionally, it is specified that the ambient air temperature has been above 40°F for a minimum of 72 hours before performing the test.

A pachometer survey of the bridge deck is the final step in the deck survey process. The pachometer survey is used to identify the depth of the steel from the surface of the concrete deck, or the cover thickness. This can then be compared against the required minimum cover depth.

Following the investigation as described above, the bridge deck is then placed into one of the following categories (NJDOT 2016):

- Category 1 – Extensive Active Corrosion
 - 5% or more of the total deck area is spalled
 - OR 40% or more of the deck area has deteriorated or has been contaminated by any combination of the following: spalls, delamination, corrosion potentials more negative than -0.35 V
 - OR 40% or more of the area of the bridge deck indicated by random chloride sampling contains greater than 2.0 pounds of chloride per cubic yard of concrete at the level of the top reinforcing steel
- Category 2 – Moderate Active Corrosion
 - 0 to 5% of the total deck area is spalled
 - OR 5 to 40% of the deck area has deteriorated or has been contaminated by any combination of the following: spalls, delamination, corrosion potentials more negative than -0.35 V
 - OR 5 to 40% of the area of the bridge deck indicated by random chloride sampling contains greater than 2.0 pounds of chloride per cubic yard of concrete at the level of the top reinforcing steel
- Category 3 – Light to No Active Corrosion
 - No spalls

- OR 0 to 5% of the deck area has deteriorated or has been contaminated by any combination of the following: spalls, delamination, corrosion potentials more negative than -0.35 V
- OR 0 to 5% of the area of the bridge deck indicated by random chloride sampling contains greater than 2.0 pounds of chloride per cubic yard of concrete at the level of the top reinforcing steel

Once the bridge deck has been investigated and placed into the appropriate category, the recommended remediation procedures can be found in the following table, Table C3, which was created by the NJDOT.

Table C3: Restoration procedures for structures affected by corrosion.
(Originally Table 9.1 in NJDOT Design Manual (NJDOT 2016))

Category	Procedures	Restoration (Considered Permanent)	Restoration (Estimated extended life 10 to 15 yrs)
Structurally Inadequate		Complete Deck Replacement (Unless restorable)	
1. Extensive Active Corrosion	Required Restoration Work	Complete Deck Replacement	Removal of all deteriorated concrete. Follow the repair procedure approved for the protective system selected.
	Testing Steps (see below)	Steps 1 through 4 as necessary. (Probably only step 1)	Step 1 only, except all the testing steps on the first five (5) bridge decks (spans) plus 10% of the remaining bridge decks.
	Suggested Protective Systems	Membrane with HMA overlay*; Concrete Thin (less than 1") Overlay Protective System. *	Membrane with HMA overlay**; Concrete Thin (less than 1") Overlay Protective System. **
2. Moderate Active Corrosion		Same as Category 1 above or Same as Category 3 below, as determined by the State.	Same as Category 1
3. Light To No Active	Required Restoration Work	Removal and Replacement of all areas of deterioration and chloride contaminated concrete as determined by corrosion potentials and/or chloride sampling. (Less than 5% of the deck area is bad).	Same as Category 1 Note: For this category of condition, permanent restoration is recommended.
	Testing Steps	Steps 1 through 4.	Same as Category 1
	Suggested Protective System	Membrane with HMA overlay*; Concrete Thin (less than 1") Overlay Protective System. *	Membrane with HMA overlay**; Concrete Thin (less than 1") Overlay Protective System. **

* When approved prior to Preliminary Plan ** Submission on a project to project basis

Testing Steps:

1.	Visual	3.	Half-Cell Test		
2.	Chloride Analysis	4.	Pachometer Test		

Delaware is distinguished in their specifications related to the testing and inspections of concrete bridge decks that are showing potential signs of active corrosion. The description of testing and inspections falls under the chapter of Bridge Preservation Strategies in the Delaware Department of Transportation (DelDOT) bridge design manual which includes two types of projects, either bridge rehabilitation or preventative maintenance (DelDOT 2017). This section will focus on the many considerations when determining the current condition of a bridge deck and what type of remediation is appropriate.

The DelDOT defines the test specifications to be used when performing a bridge inspection, which include both ASTM and AASHTO standardized test methods. The following table, Table C4, describes which test is to be performed based on what is required of the inspection.

Table C4: Specifications for testing of field materials.
(Originally Table 109-1 in DelDOT Design Manual (DelDOT 2017))

Specification	Description
Tests performed by DelDOT Materials & Research (M&R) Section	
AASHTO T24 (ASTM C42)	Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
ASTM C856 (Annex A only)	Petrographic Examination of Hardened Concrete
AASHTO T260	Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials
ASTM C1583	Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off Method)
ASTM C457	Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete
Tests performed by Outside Agencies	
ASTM C856	Petrographic Examination of Hardened Concrete
ASTM C876	Corrosion Potentials of Uncoated Reinforcing Steel in Concrete
ASTM C1202	Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration

The DelDOT defines a procedure for determining the current condition of concrete bridge decks. This procedure involves a visual inspection, delamination survey, reinforcement corrosion survey, pachometer survey, and deck coring. Each section of the inspection is briefly described in the following paragraphs.

The visual inspection involves the assessment of the following five conditions: cracking, spalling, scaling, wear, and efflorescence (DelDOT 2017). The entire bridge deck, both the top surface and the underside, should be inspected for signs of the previously mentioned conditions.

The delamination survey can be completed using either the chain-drag or hammer-sound testing methods. The location, size, and amount of delamination should be documented on a plan drawing of the bridge being inspected. Generally, the DelDOT suggests surveying the entire deck surface, however, it is permitted to test select areas of larger decks that can be used to estimate the condition of the entire deck. These areas should be selected to include sections that experience the most heavy-truck traffic or deicing exposure and the pier and joint locations. In the case that the entire deck is not surveyed, the locations should be representative to provide a balanced report of the deck condition (DelDOT 2017).

The reinforcing corrosion survey consists of performing a half-cell potential survey of the bridge deck. The DelDOT suggests that the entire deck should be surveyed, however, if it is not practical then a sufficient number of typical areas should be tested to provide a complete picture of the current condition. In addition to the half-cell readings, the tester should note whether epoxy coated reinforcement is present in the top layer only or in both the top and bottom layer. The results of the half-cell potential testing should be plotted on a contour map to make it easier to identify regions where there is a high probability of active corrosion. Finally, it is suggested that cores in areas where active corrosion is predicted as well as areas where no active corrosion is predicted should be collected and visually inspected to confirm the predicted conditions (DelDOT 2017).

A pachometer survey is performed to determine the actual depth of steel cover where other tests have been performed. The DelDOT notes that this information is useful when determining the significance of the results from chloride content testing.

Finally, concrete core samples are collected from a bridge and are then either returned to a DelDOT research laboratory or sent to an external testing agency. One of the tests performed with concrete cores is a chloride content analysis (AASHTO T260, 2016). The range that is specified for a corrosion threshold is approximately 0.02 to 0.03% by weight of concrete, or 1.0 to 1.5 pounds of chloride per cubic yard, for uncoated steel in non-carbonated concrete (ACI 2008; DelDOT 2017). The following paragraph will describe the process used to perform chloride content testing.

The chloride content test is recommended to be determined and plotted against the depth of concrete where the sample was taken. The DelDOT suggests testing 0.25-inch (in.) slices from a 4-in. diameter concrete core at depths of 0.375 in., 1 in., 2 in., and 3 in. It is recommended that the test be performed at depths greater than 3 inches if there is significant chloride contamination at the 3-inch depth. Additionally, it is recommended that the background chloride concentration is determined by using at least two samples and testing at a depth that the chloride content would not be affected by chloride ingress from the concrete surface. The number of cores that should be taken for testing is approximately one core per 2,000 square feet of total deck area, however, there should be a minimum of three cores taken for chloride content testing.

Following the full inspection of a bridge deck as described above, the DelDOT performs a deck characterization. This is a process that is used to identify the current condition of a bridge deck and determines what type of remediation is required. The deck characterization process is based on the following four factors:

- Percent deck distress and visual condition ratings;
- Estimated time-to-corrosion;
- Deck surface condition;
- Concrete quality.

Based on how a bridge is scored in each of the above categories, the table on the following page (Table C5) is used to determine what type of remediation is required, if any. This is the final step in the full investigation process of a bridge deck.

Table C5: remediation choices based on deck characterization.
 (Originally Table 109-3 in DeIDOT Design Manual (DeIDOT 2017))

Table 109-3. Deck Repair Evaluation Matrix					
Factors Repair	Deck Distress		Time-to-Corrosion Initiation	Deck Surface Problems	Concrete Quality Problems
Do Nothing	% Distress	< 1%	> 10 years	none	none
	% Distress + half cell	< 5%			
	NBI deck rating	≥ 7			
	underside rating	≥ 7			
Maintenance	% Distress	1 - 10%	> 5 years or > 10 years	none	none
	% Distress + half cell	1 - 15%			
	NBI deck rating	≥ 5			
	underside rating	≥ 5			
Overlay	% Distress	2 - 35%	Ongoing to < 5 years	yes	yes
	% Distress + half cell	10 - 50%			
	NBI deck rating	≥ 4			
	underside rating	≥ 5			
Rehabilitation	% Distress	> 35%	Ongoing	yes	yes
	% Distress + half cell	> 50%			
	NBI deck rating	≤ 3			
	underside rating	≤ 4			

NBI = National Bridge Inventory