EVALUATING CORROSIVE SITE PERFORMANCE OF COASTAL BRIDGES

by

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ABSTRACT

ROSS ALAN NEWSOME. Evaluating Corrosive Site Performance of Coastal Bridges.
(Under the direction of DR. TARA L. CAVALLINE)

Corrosion of reinforcing steel in concrete structures, such as bridges, has historically been a concern in coastal environments. Such environments are chloride-rich due to proximity to the salt water found in oceans and brackish water further inland. When chlorides penetrate the concrete and become present in sufficient concentrations in the vicinity of the reinforcing steel, the chances of corrosion are increased. To combat this issue, the North Carolina Department of Transportation (NCDOT) implemented a construction policy in 1999 to mandate corrosion prevention measures in all new concrete structures built within environments believed to be conducive to corrosion. The policy provides guidelines to determine which of several approved corrosion prevention measures are required for structures in specific areas. Some such prevention measures include low water-to-cement ratios, pozzolans, epoxy coated reinforcement, and corrosion inhibiting admixtures. The goal of the policy is to increase the service life of these new structures and to decrease maintenance and repair costs.

This research study aimed to assess the effectiveness of this construction policy. In order to do this, a series of field and laboratory tests were conducted on selected concrete bridge elements within corrosive coastal environments. These tests provided insight into the existing condition of these bridges as well as their future performance by using service life modeling. By utilizing the results of this study, deficiencies in the policy were identified and a revised policy was suggested.

Eight bridges located in corrosive sites were selected based upon a specially developed set of criteria. Field evaluation of these bridges involved taking drilled powder samples, measuring the concretes’ surface resistivity, and determining the corrosion rate of embedded reinforcing steel. Laboratory testing included testing powder samples to determine chloride content (and subsequently diffusion coefficients) and quantifying the presence of corrosion inhibiting admixture. Modeling efforts utilized field measured concrete performance characteristics to predict future performance and produce an estimated maintenance free service life for each bridge.

The findings of the study indicated that most of the concrete bridge elements observed showed little, if any, corrosion related distress. Field tests showed that the concrete had high surface resistivity, indicative of good resistance to chloride ingress and ultimately desirable long-term durability. Corrosion readings indicated that most concrete elements do not have high levels of, if any at all, corrosion. Laboratory tests showed that the chloride content of the concrete quickly diminished through the cover and typically only background chloride levels were detected at the depth of steel. This resulted in the calculated diffusion coefficients being low, which is indicative of concrete with a high resistance to chloride ingress. Corrosion inhibitor dosage rates were detected within range of the minimum specified by the policy and at consistent concentrations throughout the concrete elements. Synthesizing results of laboratory and field testing show little to no risk of corrosion-related distress for elements that are located outside of the tidal zone. Within the tidal zone, where tidal fluctuations regularly allow the concrete to become wet and then dry, some concrete elements observed showed high levels of chloride loading and increased corrosion rates while other similar elements were performing well.

The conclusion was made that the policy is providing sufficient protection in its current form for the concrete bridges that have been constructed in accordance with it. The only recommendation made for the NCDOT is to potentially enhance acceptance standards for the construction process, particularly for concrete structural elements located within the tidal zone. With this exception of encouraging the addition of acceptance standards, no further recommendations to enhance the NCDOT corrosion policy were deemed necessary based upon the results of this research project.

ACKNOWLEDGEMENTS

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I would like to thank those who were involved in the field-testing and data analysis portion of this project including Keith Violette and Alex Dilworth. These individuals travelled countless miles over several weekends to gather data necessary to the study. Without their invested time and effort to this project it would have not been possible.

Lastly, I would like to thank my family and loved ones for their continued support of everything that I do. They have all played an integral role in allowing me to become who I am today.

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LIST OF ABREVIATIONS

ACI	American Concrete Institute
ASR	Alkali-Silica Reaction
AU	Absorbance Unit
CEPRA	Connectionless Electronical Pulse Response Analysis
CI	Corrosion Inhibitor
CTV	Chloride Threshold Value
DOT	Department of Transportation
FDOT	Florida Department of Transportation
FHWA	Federal Highway Administration
gal	Gallon
gal/CY	Gallons per Cubic Yard
in	Inch
in^2/yr	Square Inches per Year
$k\Omega \cdot cm$	Kilohms per Centimeter
km	Kilometer
lb/yd^3	Pounds per Cubic Yard
LCA	Life Cycle Analysis
LRFD	Load & Resistance Factor Design
m	Diffusion Decay Index
m	Meter
mg/mL	Milligrams per Milliliter
NCDOT	North Carolina Department of Transportation
NDT	Nondestructive Testing
NED	N-(1-Napthyl) Ethylenediamine Dihydrochloride
nm	Nanometer
PCA	Portland Cement Association
pcf	Pounds per Cubic Foot
pcy	Pounds per Cubic Yard

pH	Potential Hydrogen
ppm	Parts per Million
RCPT	Rapid Chloride Permeability Test
RCT	Rapid Chloride Test
SCDOT	South Carolina Department of Transportation
SCM	Supplementary Cementitious Material
SHA	State Highway Agencies
SMU	Structures Management Unit
SN	Structure Number
STADIUM	Software for Transport and Degradation in Unsaturated Materials
UNCC	University of North Carolina at Charlotte
VDOT	Virginia Department of Transportation
w/cm	Water Cement Ratio
µg/mL	Micrograms per Milliliter
µm/yr	Micrometers per Year
γ	Unit Weight
% wt. conc.	Percentage by Weight of Concrete
ºF	Degrees Fahrenheit

CHAPTER 1: INTRODUCTION

1.1 Background and Significance

Corrosion of reinforcing steel in concrete structures, such as bridges, has historically been a concern in coastal environments. Such environments are chloride-rich due to proximity to the salt water found in oceans and brackish water further inland. When chlorides penetrate the concrete and contact the reinforcing steel, the chances of corrosion are increased. Other mechanisms can be responsible for corrosion, but chloride ingress is by far the most prevalent cause for its development and related deterioration (ACI 2010). Since coastal environments have larger deposits of chlorides than most inland areas (with the exception of areas receiving heavy deicer use), state highway agencies often develop unique specifications to protect coastal reinforced concrete structures from corrosion.

The deterioration effect on concrete from corrosion is due to the increase in volume the reinforcing steel undergoes while corroding. The products of corrosion, commonly called “rust,” take up a greater volume inside the concrete structure than the initially placed reinforcing steel. This increase in volume produces expansive tensile stresses on the surrounding concrete that lead to cracking and spalling (ACI 2010). Once these signs of deterioration manifest themselves the rate of corrosion typically increases exponentially as the protective concrete cover is removed and chloride ions, introduced by the environment, become more readily able to access the steel through the cracks and spalls.

Many methods are utilized in an attempt to reduce the impact of corrosion on concrete. Some methods seek to prevent corrosion by making concrete that is impermeable to chloride ingress. Concrete that is particularly good at resisting chloride ingress can be obtained by utilizing a lowered water-to-cement ratio (*w/cm*) and/or replacing a percentage of the cement with pozzolans (ACI 2010). Other methods of corrosion prevention seek to prevent corrosion of steel even if chlorides are able to penetrate the concrete. The use of epoxy coated reinforcement and corrosion inhibiting admixtures containing calcium nitrite both prevent corrosion in the presence of chlorides (ACI 2010). A combination of these two types of methods is commonly used when attempting to mitigate or reduce the chance for corrosion.

1.2 Objectives and Scope

The state of North Carolina has approximately 12,009 miles of estuarine coastline along the Atlantic Ocean, most of which are developed and populated areas (UNC TV 2020). The state-owned infrastructure in this region is vital to serve the residents of the many cities and to facilitate industrial operations along the coast. However, developing infrastructure in these areas poses a unique set of challenges brought about by the negative impact the chloride-rich coastal environment has on reinforced concrete structures. These negative impacts manifest when chloride ions from the environment migrate through the protective concrete cover and interact with reinforcing steel causing early onset of corrosion. The corrosion of reinforcing steel in concrete bridges is a prevalent issue that directly costs approximately $8.3 billion per year according to a Federal Highway Administration (FHWA) report (Poursaee 2016). Cognizant of the large cost of corrosion-related distress and understanding the role of chloride ion ingress in corrosion reactions, the North Carolina Department of Transportation (NCDOT) implemented a construction policy in 1999 to mandate corrosion prevention measures in all new concrete structures built within environments believed to be conducive to corrosion (NCDOT 2019). This policy is included in the Structures Management Unit Design Manual (Chapter 12-12) where guidelines are provided to determine what corrosion prevention measures are needed in specific situations. The goal of the policy is to increase the service life of these new structures and to decrease maintenance and repair costs. If the policy is successful, these structures are expected to have a service life of at least 50-years.

The policy designates two corrosive zones spanning the coastline of North Carolina. These zones are seen in Figure 1.1 as the corrosive zone (located on or east of the blue line) and the highly corrosive zone (located on or east of the red line) (NCDOT 2019). All concrete structures constructed west of the blue corrosive line are not considered to be located in a corrosive environment and do not require any specific corrosion protection measures. Corrosion protection measures in both corrosive zones are limited to stream crossings only. Bridges that do not serve as a stream crossing do not require special provisions for corrosion protection regardless of any corrosive zone they are located in.

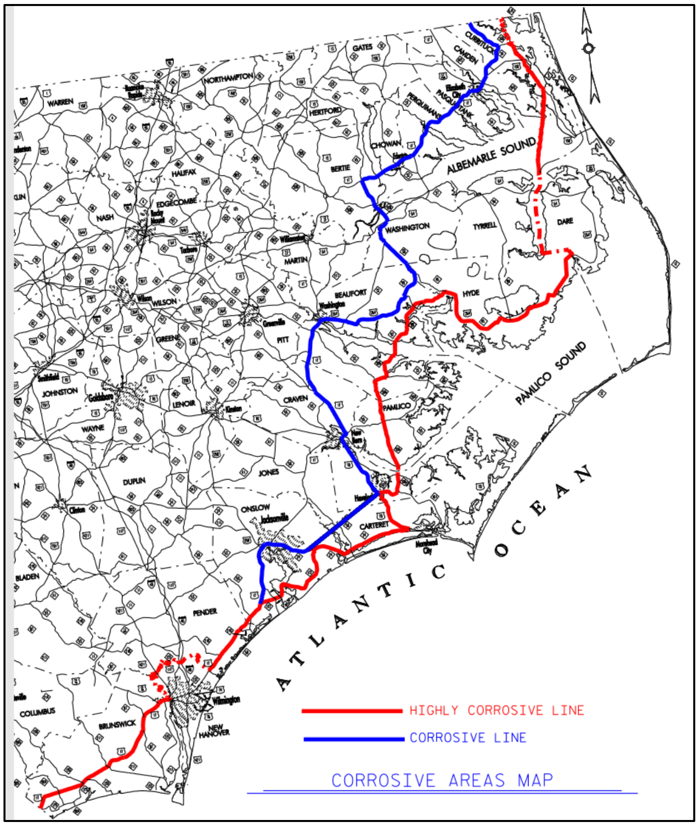


Figure 1.1: NCDOT corrosive areas map (NCDOT 2019)

The following corrosion protection measures are applicable to the corrosive zone for elements within 15 feet of the mean high tide and all concrete bridge elements at stream crossings in the highly corrosive zone:

- *Class AA concrete for bridge decks shall contain fly ash or granulated blast furnace slag at the approved substitution rates.*
- *Class AA concrete shall be used in all cast-in-place columns, bent caps, pile caps, and footings, and shall contain calcium nitrite corrosion inhibitor.*
- *The water/cement ratio for concrete piles shall not exceed 0.40.*

- *All bar supports used in the (barrier rail, parapet, sidewalk, deck, bent caps, columns, pile caps, footings) and all incidental reinforcing steel shall be epoxy coated.*
- *Prestressed concrete (girders, precast deck panels, cored slab units, piles) shall contain calcium nitrite corrosion inhibitor.*

For both corrosive zones, elements that undergo repeated wetting and drying due to tidal fluctuations require that 5.0% of the portland cement be substituted with silica fume (a proportional replacement of Class F fly ash may also be approved).

The NCDOT monitors the current health of its infrastructure assets with a combined infrastructure health score. The score comprises of the condition ratings of bridges, pavements, and roadside features of each county as well as the state as a whole. The NCDOT strives to maintain an infrastructure health score of at or above 80%. However, in 2019 the state as a whole was given a rating of 74% and many coastal counties report bridge health scores of as low as 61% (NCDOT 2019). The NCDOT's corrosion policy is a critical step to improving the low bridge health scores in coastal areas by addressing the most prevalent deterioration mechanism affecting them.

In the years since this policy has been in place there have been more than 200 bridges within the zone that have been newly constructed or replaced using provisions required in the current corrosion prevention policy. By selecting and analyzing structures that have been in place for approximately 10 to 15 years, a fair assessment of the current and projected performance of the policy can be made. Research tasks necessary to support this assessment is as follows:

1. Identify a set of bridges within both the corrosive and highly corrosive zones that are of the proper age, accessibility, and similar construction to each other for means of drawing comparisons.
2. Collect concrete powder samples from various structural components of each bridge to examine the degree of chloride loading. These chloride concentrations will allow for an assessment of the actual severity of each area for being a chloride-rich environment.
3. Using the chloride concentrations found, determine a diffusion coefficient and estimated surface chloride concentration for each of the structures.
4. Conduct service life prediction modeling utilizing Life365 software and concrete performance properties determined through field and laboratory testing of each bridge.
5. Measure the amount of calcium nitrite found in concrete samples to verify that the specified dosage rates of corrosion inhibitor are being utilized in each structural element.

With the successful completion of the research tasks listed above, and associated analysis of the data collected, it is hypothesized that sufficient information will be gathered to assess if the current policy in place is achieving desired results. This assessment of the policy will include a decision on if the policy supports design provisions that sufficiently support the expected 50-year service life, and if the use of specific corrosion prevention measures on various structural concrete elements are warranted or unnecessary in specific conditions. Additionally, findings will show whether the specified dosage rates of calcium nitrite have been utilized on bridges included in the study.

Verification of the effectiveness of this policy and the appropriateness of each of its provisions is important to NCDOT for two key reasons. These reasons are:

- The use of corrosion protection measures is costly and the decision to utilize specific provisions for specific structural elements is not taken lightly.
- Failure to provide adequate corrosion protection to concrete structures in these highly corrosive zones will result in significantly decreased maintenance free service lives.

The first reason stems from the NCDOT's financial motivation to minimize the cost of their construction projects statewide. Corrosion prevention measures create an inflated initial cost to all structures constructed in the corrosive zone compared with structures constructed elsewhere in the state. In the event that this policy is either deficient or excessive, substantial cost savings could be generated associated with reducing the amount of corrosion related deterioration and maintenance needed through the service life (if the policy is currently deficient and can be improved), or by reducing initial costs on new construction (if the policy is currently excessive and provisions can be reduced). Ultimately, the policy's use is best justified financially if it is appropriately tuned to the anticipated needs of the structures in these corrosive areas as required to serve the design service life. The second reason this research is important is that it will verify that the structures constructed under this policy have adequate protection from corrosion. It is vital that if the policy is deficient in some way, a change is made to correct it so that new structures constructed are protected from corrosion as efficiently as possible with an acceptable level of risk. Both reasons are interrelated because the NCDOT desires to both reduce corrosion related maintenance and increase service life. This research also supports the NCDOT’s core priorities to keep roads and bridges safe, to be effective and efficient, and to reduce construction delays all while providing great customer service for the public. The verification of this policy stands to increase the NCDOT’s ability to achieve each one of these organizational performance goals, as well as its ultimate goal of achieving an infrastructure health composite index of eighty percent or greater (NCDOT).

CHAPTER 2: LITERATURE REVIEW

2.1 Overview of Corrosion

The use of embedded reinforcing steel has led to an expansion of applications in which concrete can be utilized. Reinforced concrete is now, and has historically been, used in a wide variety of construction projects. The low tensile capacity of concrete is mitigated by the addition of steel to construct structural members that can withstand large amounts of internal tensile stress. The mechanical properties of the steel, as well as the surrounding concrete, are jeopardized if the steel is subjected to conditions that facilitate corrosion. The corrosion of steel in concrete is one of the most prevalent and costly problems affecting the integrity, strength, and aesthetics of concrete structures. Although other mechanisms not involving chloride ions can lead to the corrosion of steel in concrete, such as in carbonation of concrete, chloride ingress is by far the most prevalent cause for the development of corrosion and corrosion related deterioration (ACI 2010). Chloride ions can be introduced to concrete in a variety of ways both before and after the concrete has been placed. Some such ways include introduction through admixtures, contaminants, exposure to coastal environments, industrial brine, or deicing salts (ACI 2010). Chloride rich coastal waters provide a plentiful supply of chloride ions making bridges and structures in close proximity particularly prone to corrosion. Even without direct contact, coastal structures are still subjected to high levels of chlorides due to airborne contaminants that can deposit them into concrete. The level of contamination through these means is typically dependent on proximity to sea water, wind orientation, and contact of contaminated rainfall on the concrete (Salta et al. 2012).

The negative impact that corrosion has on concrete is produced due to the change of volume the reinforcing steel undergoes while corroding. The products of corrosion, often called “rust”, take up a greater volume inside the concrete structure than the initially placed reinforcing steel. Certain instances of corrosion can increase the volume of the steel by as much as eight to ten times (ICRI 2013). This increase in volume produces expansive tensile stresses on the surrounding concrete that led to staining, cracking, and spalling (ACI 2010). Once these signs of deterioration manifest themselves the rate of corrosion typically increases exponentially as the protective concrete cover is removed and chloride ions introduced by the environment become readily able to access the steel through the cracks and spalls. This chain reaction of effects throughout the stages of corrosion onset is illustrated in Figure 2.1 below.

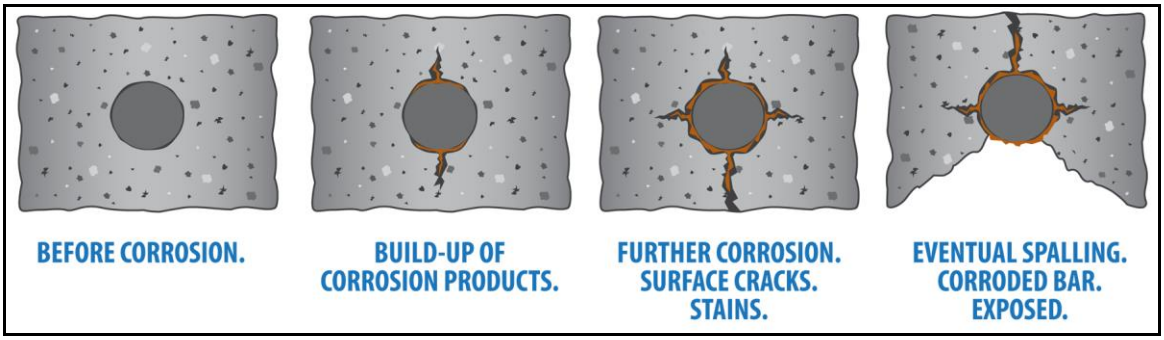


Figure 2.1: Stages of corrosion related deterioration (The Constructor 2020)

2.1.1 Principles

Concrete provides corrosion resistance to steel by being both physical and chemical resistant to chloride ingress and corrosion (Poursaee 2016). It provides a natural corrosion resistant environment for embedded reinforcing steel due to its high alkaline content and high pH of between 12.5 and 13.5 (ACI 2010, Poursaee 2016). These alkalis passivate the steel giving it a protective layer from corrosion known as the passive film

(ACI 2010). This film is created when iron oxides form as solid phases around the surface of the steel. The oxide molecules presence on the steel blocks the sites where reactions would dissolve the metal atoms (ACI 2010). This passive film is the first line of defense before extensive corrosion can begin. The layer does not actually prevent corrosion, but instead slows the corrosion rate down by upwards to an order of three magnitudes what it would be without the passive layer (ACI 2010). This means that the typical passive corrosion rate is as low as 0.1 µm/yr (ACI 2010). When the passive layer has been lost, through a process called depassivation, the active corrosion of reinforcing steel begins (Poursaee 2016).

The corrosion of reinforcing steel in concrete due to chloride ingress is a process involving an electrochemical reaction (ACI 2010). This type of reaction requires the presence of both oxygen and moisture and results in the transfer of electrons from one molecule to another. In the case of corrosion of reinforcing steel in concrete there must be two half-cell reactions. The necessity of these half-cell reactions is due to the absence of an external electrical source. These half-cell reactions require one molecule that is capable of producing electrons (anodic reaction) and one that is capable of receiving electrons (cathodic reaction). In corrosion the anodic reaction is the oxidation of iron. This reaction produces iron atoms in a +2 oxidation state [Fe^{+2}] which are known as ferrous ions (Smith and Virmani 2000, ACI 2010). There are four possible anodic half-cell reactions that can take place in the corrosion of steel. These possibilities are as listed as follows (ACI 2010):

$$\mathrm{Fe} \rightarrow \mathrm{Fe}^{++} + 2\mathrm{e}^{-}$$

$$2\mathrm{Fe}^{++} + 4\mathrm{OH}^{-} \rightarrow 2\mathrm{Fe(OH)}_2$$

$$2\mathrm{Fe(OH)}_2 + 1/2\mathrm{O}_2 \rightarrow 2\mathrm{FeOOH} + \mathrm{H}_2\mathrm{O}$$

$$\mathrm{Fe} + \mathrm{OH}^{-} + \mathrm{H}_2\mathrm{O} \rightarrow \mathrm{HFeO_2}^{-} + \mathrm{H}_2$$

The cathodic reaction is the reduction of oxygen. This reaction produces hydroxyl ions which are negatively charged hydroxide atoms [OH^-] (Smith and Virmani 2000, ACI 2010). Two possible cathodic half-cell reactions can result from the corrosion of steel. Both of these possibilities are listed below (ACI 2010):

$$2\mathrm{H}_2\mathrm{O} + \mathrm{O}_2 + 4\mathrm{e}^{-} \rightarrow 4(\mathrm{OH})^{-}$$

$$2\mathrm{H}^{+} + 2\mathrm{e}^{-} \rightarrow \mathrm{H}_2$$

The specific anodic or cathodic half-cell reaction that occurs is dependent upon the environment at the time and location of the reaction. The availability of oxygen and the potential hydrogen (pH) level of the cement paste pore solution are key factors dictating which reactions can or will occur (ACI 2010). Typically, reactions with an abundance of oxygen will tend to result in a positive anodic electrochemical potential. An illustration of a typical half-cell reaction attributed to corrosion of reinforcing steel in concrete can be seen in Figure 2.2.

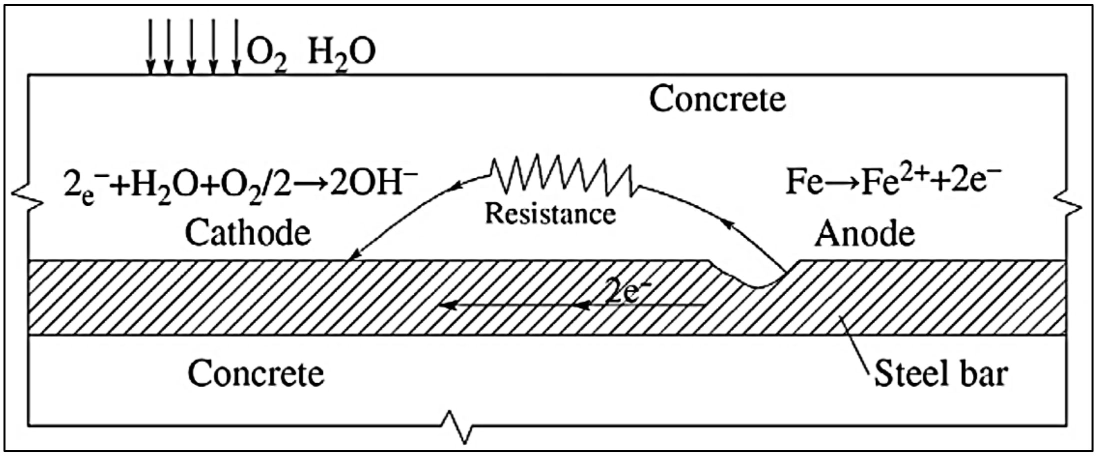


Figure 2.2: Half-cell reaction of steel corrosion in concrete (Yeganeh et al. 2019)

All metals are thermodynamically unstable. This means that even under normal atmospheric conditions, without exposure to chlorides, they will oxidize in an attempt to break down fully into the original constituent parts (PCA 2019). Because of this, there is no way to prevent corrosion of carbon steel in its entirety. Knowing that corrosion is inevitable, the important factor determining the impact to concrete is not if corrosion will occur, but the rate at which the corrosion will occur. The corrosion rate is the rate that the anodic half-cell reaction can remove electrons from the iron molecules (ACI 2010). Factors that are known to influence corrosion rate are concentration of water and oxygen, chloride presence, resistivity of concrete, pH, temperature, humidity, and the ratio of the steel reinforcement’s surface area that acts as an anode compared to the amount that is a cathode (Smith and Virmani 2000). A high corrosion rate is most detrimental and can result in noticeable distress of the concrete from an early age. A very low corrosion rate may have little or no impact on concrete even after a very long time due to the slow and limited development. Concrete that is exposed to repeated cycles of wetting and drying, such as in a tidal zone in a coastal environment, are susceptible to some of the highest rates of corrosion. This is because concrete has a lower resistivity when wet and the alternating cycles of wetting and drying provide moisture and oxygen respectively to the reaction (Smith and Virmani 2000).

Concrete is susceptible to chloride penetration even when it has matured. This is due to a process of chloride diffusion, which takes place within the capillary pore structure of the cement paste (ACI 2010). This ability to transport chloride ions is impacted by the physical and chemical binding of ions in concrete (Bhattacharjee and M. 1998). Pastes which contain a dense capillary pore structure have a natural resistivity to penetration of chlorides and contaminants and are therefore more desirable for use when corrosion mitigation is desired (Poursaee 2016). The ability for chlorides to pass through the pore structure can be quantified as the rate of diffusion (ACI 2010). The rate of diffusion for a given paste is dependent upon many factors such as *w/cm*, cement type, type of chloride, temperature, saturation, and concrete maturity (ACI 2010). Chloride diffusion relates to time until corrosion initiation, which is commonly modeled with the use of Fick's second law of diffusion (Cavalline et al. 2013). Time until corrosion initiation is given by the function:

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

Where C is the concentration of chlorides, C_0 is the surface chloride content, *erf* is the error function, *t* is the time in years, and D_c is the chloride diffusion coefficient in in^2/year (Cavalline et al. 2013). The chloride diffusion coefficient is an integral part of this formula and obtaining an accurate value of it is necessary in service life modeling (Salta et al. 2012).

Diffusion rates for chlorides are time dependent and typically happen over an extended period of time and exposure (Bhattacharjee and M. 1998). The rate of diffusion can be accelerated with the presence of defects such as cracks or poor consolidation in the concrete. These imperfections act as a passageway for chlorides and moisture to bypass the capillary pore structure and penetrate the surface of the concrete quickly (ACI 2010).

The diffusion of chloride ions into concrete can be illustrated in a chloride profile. A chloride profile is a graph depiction of chloride concentrations at various depths in concrete. For typical concrete with a given surface chloride concentration chlorides the amount of chlorides that are able to be transferred through diffusion will decrease with depth into the concrete. By mapping the chloride contents at various depths, it can be determined what factors allow for concrete to become more or less permeable to chlorides. An example of such can be seen in Figure 2.3 which demonstrates a decrease in the *w/cm* will result in concrete that is more resistant to chloride ingress (ACI 2010).

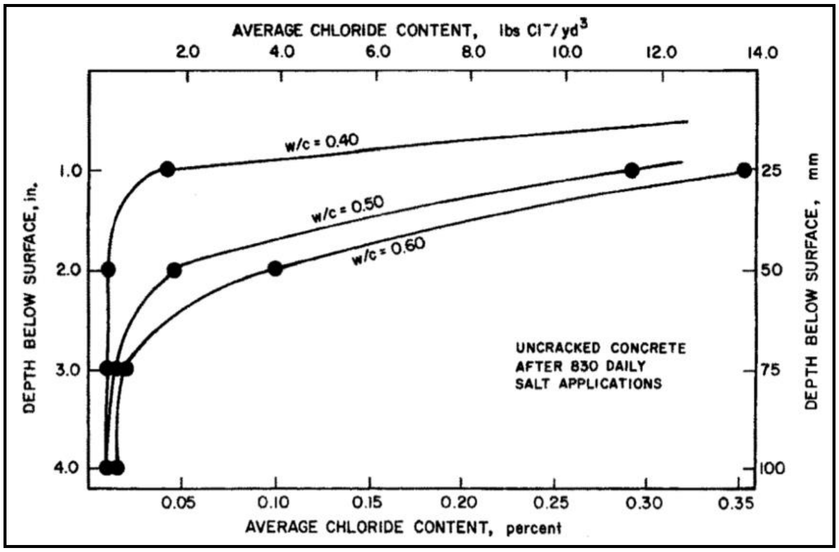


Figure 2.3: Effect of variable *w/cm* on chloride ingress (ACI 2010)

Corrosion is not guaranteed to take place even with chlorides present in the concrete (ACI 2010). In fact, most new concrete produced has chlorides present that have been introduced through contamination by aggregates or admixtures (GCP Applied Technologies 2016). A particular mass of chlorides, oxygen, and water is required to break down the passive film and initiate corrosion (ACI 2010). The value for the necessary chloride content to induce corrosion is referred to as the chloride-corrosion threshold or chloride threshold value (CTV). There is much debate around what the exact threshold levels are for steel, but field test results for CTV's commonly fall between 1.82% to 2.45% total chlorides by binder weight and laboratory test results fall between 0.88% and 1.58% by binder weight (Figueira et al. 2017). Environmental factors that influence the chloride-corrosion threshold are resistivity of the concrete, moisture, temperature, humidity, and oxygen concentration (Smith and Virmani 2000). Although environmental factors are typically considered to be the main reason to implement corrosion prevention measures, there are also physical factors of concrete that impact corrosion. These physical factors include tricalcium aluminate (C_3A) and tetracalcium aluminoferrite (C_4AF) concentrations, pH level, *w/cm*, and whether the chlorides were introduced to the fresh concrete or penetrated it while it was hardened (ACI 2010). Although these factors are known to influence corrosion, it is often hard to predict how they will react with one another. One example of such is that as pH increases so does the steel ability to resist corrosion without pitting. However, as pH increases so does the total number of chlorides present for a given chloride content (ACI 2010). It is complex phenomenon, such as this, that make understanding of CTV's so difficult.

2.2 Impact of Corrosion

In countries globally, corrosion in reinforced concrete structures result in deterioration with an estimated value in the multi-billion-dollar range (ICRI 2013). The corrosion problem alone in the United States was estimated to be between one and three trillion dollars to repair all corrosion-related distress in 2002 by the FHWA in their *Cost of Corrosion Report* (ICRI 2013). For this reason, corrosion is the costliest and most prevalent deterioration mechanism of reinforced concrete and has been designated as a high priority area in the FHWA Structures Research Program (Moradllo et al. 2017, Virmani and Clemena 1998). The high levels of deterioration that are attributed to corrosion affect the safety, economy, and sustainability of the world's concrete infrastructure (Poursaee 2016). There have been many new advancements into technologies and materials to combat this this problem but it is still unclear what the best practices are to develop cost effective and durable structures (ICRI 2013).

Corrosion is a particularly prominent problem in coastal areas which typically have chloride-rich environments. Coastal corrosion is typically due to chloride ingress and is a major financial and public safety liability to state and federally owned bridges and roadways. This has forced state highway agencies to implement high standards for use of corrosion prevention methods. These corrosion prevention measures are important in extending the service life of bridges, but are expensive compared to standard construction practices. This increase in initial construction cost makes building in coastal areas particularly difficult as many state and highway agencies try to make due with already scarce funding.

Marine structures are most commonly impacted by corrosion in the areas referred to as splash, or tidal, zone where salt water most readily deposits chloride ions (Salta et al. 2012). This is due to wetting and drying cycles that decrease the concretes resistivity and provide moisture and oxygen to the reaction (Smith and Virmani 2000). Airborne contaminants can also deposit chlorides into concrete. The level of contamination through these means is typically dependent on proximity to the water, sea wind orientation, and contact of contaminated rainfall on the concrete (Salta et al. 2012).

2.3 Corrosion Mitigation

The effects of corrosion can be mitigated by applying a proactive approach to both structural design and mixture design to produce a well performing structure that is suitable for its service environment. The general principle behind corrosion mitigation is to stop chloride ions from reaching the steel and in the event that they do to control the reaction they have. Whenever reinforced concrete will be exposed to contaminants, coastal environments, industrial brine, or deicing salts there are several aspects of design in concrete and construction that should be considered to reduce the risk of corrosion onset.

2.3.1 Design Provisions

The most common corrosion mitigation technique used across the world is the increase of concrete cover (McDonald 2016). This design specific technique does not rely on concrete performance or chemical admixtures. ACI 318 (2014), Building Code Requirements for Structural Concrete, provides state of the practice construction specifications on how to incorporate corrosion resistance into the design of a reinforced structure. It specifies that to provide sufficient corrosion resistance a cover of not less than 1 ½ inches should be used for cast in place nonprestressed members exposed to weather or in contact with the ground. The same members not exposed to weather or the ground can utilize lesser amount of cover (ACI 2014). The cover requirements for different members, exposure conditions, and reinforcement sizes is specified as seen in Table 2.1 from ACI 318 (2014). Alternative methods to provide corrosion resistance such as membranes or coatings are permissible as long as they offer equivalent protection to the specified cover. Regardless of the alternative method utilized, concrete members exposed to weather or in contact with the ground can never have less cover than the equivalent member not exposed to weather conditions (ACI 2014).

Table 2.1: Cover requirements for cast in place nonprestressed concrete

Concrete Exposure	Element Type	Reinforcement	Specified Cover, in.
Case against and permanently in contact with ground	All	All	3
Exposed to weather or in contact with ground	All	No. 6 through No. 18 bars	2
		No. 5 bars, W31 or D31 wire, and smaller	1 1/2
Not exposed to weather or in contact with ground	Slabs, joists, and walls	No. 14 and No. 18 bars	1 1/2
		No. 11 bar and smaller	3/4
	Beams, columns, pedestals, and tension ties	Primary reinforcement, stirrups, ties, spirals, and hoops	1 1/2

Precast nonprestressed and prestressed members are generally allowed to have smaller cover requirements than equivalent nonprestressed members. This is because of the greater quality control while manufacturing proportioning, placing, and curing under plant conditions (ACI 2014). The specifications for cover of precast nonprestressed and prestressed members can be seen in Table 2.2. Cast in place members can also qualify under these cover requirements if the control of form dimensions, placing of reinforcement, quality of concrete, and curing procedures is equal to or higher than what would normally be expected in plant manufactured conditions (ACI 2014).

Table 2.2: Cover requirements for precast nonprestressed and prestressed concrete

Concrete Exposure	**Element Type**	**Reinforcement**	**Specified Cover, in.**
Exposed to weather or in contact with ground	Walls	No. 14 and No. 18 bars: tendons larger than 1-1/2 in, diameter	1 1/2
		No. 11 bar and smaller, W31 or D31 wire, and smaller, tendons with strands 1-1/2 in. diameter and smaller	3/4
	All other	No. 14 and No. 18 bars: tendons larger than 1-1/2 in. diameter	2
		No. 6 and No. 11 bars: tendons larger than 5/8 in. diameter through 1-1/2 in. diameter	1 1/2
		No. 5 bars, W31 or D31 wire, and smaller, tendons with strands 5/8 in. diameter and smaller	1 1/4
Not exposed to weather or in contact with ground	Slabs, joists, and walls	No. 14 and No. 18 bars: tendons larger than 1-1/2 in, diameter	1 1/4
		Tendons and strands 1-1/2 in. diameter and smaller	3/4
		No. 11 bar, W31 or D31 wire, and smaller	5/8
	Beams, columns, pedestals, and tension ties	Primary Reinforcement	Greater of d_b and 5/8 and need not exceed 1-1/2
		Stirrups, ties, spirals, and hoops	3/8

Despite all minimum cover specifications, concrete cover should be appropriate for the environmental condition in which it will be located. It is considered good practice to exceed the minimum cover requirement and to utilize alternative corrosion protection measures in severe environments (ACI 2014).

2.3.2 Mixture Designs

The use of appropriate mixture designs for the given environment that the structure will be subjected to is a critical preventive step to reducing the risk of corrosion related damage in reinforced concrete. Most aspects of the mixture design will not serve to reduce the corrosion rate of the steel but will aid in providing the steel with necessary protection from chlorides introduced from the outside environment by decreasing porosity and permeability. Achieving a mixture design that is suitable for use in highly corrosive areas can be done by reducing the water-cementitious materials ratio (*w/cm*) and including the use of pozzolans such as fly ash, granulated blast furnace slag, or silica fume, non-porous aggregate, and corrosion inhibitors (ACI 2010).

The *w/cm* has a direct effect on the porosity and permeability of the concrete. As a general rule, the lower the ratio the lower the porosity of the concrete. Concrete with low porosity is highly desirable when selecting a mixture design to mitigate corrosion due to the decrease in the ability for chloride ions to penetrate the concrete. Typical high-performance concrete has a *w/cm* between 0.40 and 0.30 (ACI 2010). The lower side of the spectrum should be considered in mixtures utilized in highly corrosive areas. Mixtures containing a ratio above a 0.45 should not be considered for use in reinforced concrete that is exposed to marine environments (Cicek 2014).

Pozzolans, such as fly ash, have had success in decreasing the permeability of concrete by developing a denser microstructure. It is important to note however that due to the extended length of the pozzolanic reaction the concrete may be more permeable to chloride ingress at early ages compared to a similar non fly ash mixture. For a fly ash mixture to be successful in decreasing permeability proper curing procedures must be utilized before the concrete is exposed to chlorides. If these curing procedures are done correctly the fly ash mixture will typically provide greater resistance to chloride ingress once matured (ACI 2010).

Aggregate selection generally does not have a large effect on the level of corrosion resistance the concrete is able to provide but for the greatest chloride resistance aggregates should be non-porous and free of any contaminants containing chlorides (ACI 2010). The size of aggregates can have an effect on chloride penetration to steel when coupled with certain design characteristics of the constructed member. The porosity of the cement paste is typically higher surrounding aggregates than it is in the majority of the paste. This causes issue if the concrete cover over the reinforcing steel is approximately the same as the size of the aggregate. In this case, the placement of an aggregate with one side exposed to the environment and the other in close proximity to the reinforcing steel can provide a pathway of low permeability that chloride ions can exploit to gain quick access to the steel (ACI 2010).

Aggregate selection can lead to other issues in concrete that may indirectly affect the corrosion resistance of concrete. The reaction between alkalis found in cement and silica found in aggregates leads to alkali-silica reaction (ASR). ASR produces a gel like substance across the surface of the aggregate. Similar to corrosion, the negative effect of

ASR is due to the change of volume the aggregate undergoes while reacting. This can lead to cracking, spalling, and reduction of strength of the concrete which typically allows chloride ions to readily access the reinforcement.

The selection of admixtures utilized to achieve desired performance of concrete is important in all aspects of mixture design and construction. For concrete in highly corrosive areas the required use of corrosion inhibiting admixture is a common practice. Corrosion inhibitors are a name given to a variety of admixtures that act to decrease the amount of corrosion or the corrosion rate undergone by metals in concrete. The most common and effective corrosion inhibitors contain the active ingredient calcium nitrite [$Ca(NO_2)_2$]. Corrosion inhibitors do not reduce the ability for chloride ions to penetrate the concrete. They only serve to decrease the impact of corrosion by limiting the creation of expansive corrosion products (Jaknavorian et al. 1995). The use of corrosion inhibitors is discussed in further depth in Section 2.3.3.3 of this report. Water-reducing admixtures can also be beneficial in creating a mixture that is workable while still maintaining a low *w/cm* (PCA 2019). Keeping the *w/cm* low will aid in reducing porosity and prevent chloride ingress (ACI 2010). Shrinkage reducing admixtures are used to reduce early age cracking by controlling drying shrinkage (PCA 2019). Since cracking typically leads to an increased rate of both chloride diffusion and corrosion, the use of shrinkage reducing admixtures may be beneficial in corrosion mitigation (ACI 2010). Some admixtures should be avoided entirely during reinforced concrete construction. Set accelerators containing calcium chloride [$CaCl_2$] should never be used as they themselves contain chlorides that will contaminate the concrete and promote early onset corrosion of the steel (ACI 2010).

2.3.3 Construction

The practices followed during construction of reinforced concrete can be an integral factor in enhancing the performance of reinforcing steel in corrosive environments. These approaches are plentiful in number and practices vary widely, but all incorporate one of three guiding principles: to design and construct reinforced members to maximize the natural protection provided by the concrete, to apply penetrating barriers or treatments to reduce exposure to chloride ions, or to prevent corrosion of the reinforcement directly (ACI 2010).

2.3.3.1 Provisions to Maximize Protection of Concrete

Structural design to provide drainage of storm water contaminated with chlorides off of and away from the structure is an imperative measure to be taken in corrosion prevention. Concrete is pervious and therefore any water that runs or pools on the surface will have the ability to penetrate the concrete (ACI 2010). The level of penetration is a factor of concrete properties, time, and environmental conditions, but even a onetime deposit of contaminants containing chlorides on the outer layer of the concrete will gradually penetrate the cover to the reinforcement through diffusion. Intermittent wetting of the surface can accelerate this process (ACI 2010).

The most widely utilized form of corrosion protection is increased cover (McDonald 2016). Cover is the thickness of concrete between the outermost surface of the concrete to the outermost surface of the closest piece reinforcing steel (ACI 2010). For chloride ingress from the environment, concrete cover is a primary defense to corrosion prevention. The thickness of cover required to provide corrosion resistance is determined by the environmental conditions and the permeability of the concrete (ACI

2010). By providing a greater amount of cover there is more concrete in place to protect reinforcing steel from chloride diffusion. However, the amount of cover that can be provided is typically limited by factors of design and constructability. When cover thickness is limited, the quality of the cover becomes much more important. Highly permeable concrete does not provide good protection from chloride ingress and corrosion. For this reason, cover that provides good protection typically has a low porosity achieved through use of a low *w/cm* or SCM's (ACI 2010). Construction practices such as proper consolidation, finishing, and curing can increase the corrosion protection offered by of the cover (ACI 2010). The onset of corrosion can damage the cover by producing cracks and spalls. When these forms of deterioration manifest, the corrosion protection provided by the cover is greatly reduced (ACI 2010). The time to spalling of the concrete cover is a function of the ratio of cover to bar diameter, spacing, and concrete strength (ACI 2010). Although the necessity of adequate cover is well known and accepted, testing conducted to quantify the effectiveness of cover at protecting the reinforcement from corrosion has shown it is an unpredictable relationship. Estimates predict that it is somewhere between a little greater than linear and as much as the squared of the cover thickness (ACI 2010).

Cracking is an inevitable reality of concrete. Cracks create pathways in the concrete that facilitate the quick and easy penetration of chloride ions to the reinforcing steel. Although all cracks allow for the transmission of chlorides, the orientation of the crack is important in predicting the effect it will have on corrosion. Cracks that are perpendicular to the reinforcing steel are viewed as being less problematic than parallel cracks because the chlorides can only access a relatively small and isolated section of the steel (ACI 2010). These perpendicular cracks will often result in corrosion of the reinforcement no more than three bar diameters in length (ACI 2010). Parallel cracks cause greater concern due to the possibility of corrosion across a large length of reinforcing steel (ACI 2010). Predicting where controlled cracks will form is an important design consideration based upon the loading and geometrical properties of the section. Controlled cracks are often small and are intentionally oriented perpendicular to the reinforcing steel. Design should incorporate the effect of controlled cracks by offering enough reinforcing steel to account of the loss in concrete strength from the crack. Uncontrolled cracking is often the result of settlement, shrinkage, or overloading. These cracks are often larger than controlled cracks. The possibility for uncontrolled cracking should be included in the design by putting highly susceptible areas in convenient to repair locations and by constructing control joints in an attempt to direct and/or control the cracking (ACI 2010).

2.3.3.2 Prevention of Chloride Ingress and Corrosion

The prevention of chloride ingress is accomplished through preventive measures to keep contaminated water and air from accessing the concrete. Water in particular provides an efficient means of transport for chlorides, and generally, if it is kept away from the concrete its chloride contaminants will also (ACI 2010). Waterproof membranes form a physical barrier between the concrete and the environment. Some can even offer this protection in the event that a small hole or bubble is present (ACI 2010). For this reason, they are very effective at preventing water and chloride exposure. Waterproof membranes fall into one of two main categories, preformed sheets and liquid-applied materials (ACI 2010). Both types of membranes performance are based largely upon the workmanship and quality of the installation. For this reason, both field and laboratory testing to evaluate the benefits of each type compared to the other have been inconclusive (ACI 2010). Preformed sheets have uniform quality due to the controlled environment they are manufactured in. However, they can be harder to install properly then liquid-applied materials (ACI 2010). Liquid-applied materials are typically cheaper and easier to install but their performance is based completely upon the quality of the workmanship used to install it (ACI 2010). For both types of membranes, the most common issue impacting their performance after installation is blistering. Blistering is formed by the expansion of gases and moisture that was trapped underneath the membrane during installation. It is commonly a function of the moisture content and porosity of the concrete as well as atmospheric conditions at the time of installation. Blistering can be avoided if the membrane is installed when atmospheric conditions are suitable while the concrete is curing and the concrete surface is free of excess moisture and contaminants (ACI 2010).

Overlays are used to achieve the same principle of corrosion protection as a waterproof membrane. They are thin layers of specialty concrete that serve as a physical barrier between the reinforced concrete and the environment (ACI 2010). The most common application of overlays is seen on concrete bridge decks. There are many types of overlays that can be utilized, but there are two types that are designed specifically to provide corrosion protection. These are silica fume-modified and latex-modified concrete overlays (ACI 2010). Silica fume-modified concrete overlays achieve corrosion protection through a combination of high-range water-reducer and silica fume. The resulting mixture has a low permeability that resists chloride ingress to the deck concrete below (ACI 2010). Latex-modified concrete overlays are conventional portland cement with polymeric latex added into the mixture water. The polymers from the latex provide an increased binding ability, lowered allowable *w/cm*, and both an increased durability and resistance to penetration by chloride ions (ACI 2010). Although performance of latex-modified overlays is very good they are highly susceptible to plastic-shrinkage cracking. To reduce shrinkage cracking and to achieve the highest levels of performance and corrosion resistance from the overlay proper curing procedures should be implemented (ACI 2010).

To completely prevent corrosion of reinforcing steel in concrete, the use of noncorrosive steels or chloride penetration resistant coatings are the most common practices utilized (ACI 2010). The substitution of typical carbon reinforcing steel for a type of noncorrosive steel, such as stainless steel, can be a beneficial alternative for special applications in highly corrosive and wet environments. The higher cost of stainless steel makes replacement of all reinforcing steel a less utilized corrosion prevention measure, but it is commonly used as hardware for securing precast panels (ACI 2010). A less costly alternative is to use stainless steel-clad reinforcing bars. Testing conducted by the FHWA while examining the time to corrosion of these bars found they reduced the frequency of corrosion related cracking but did not eliminate it completely (ACI 2010). The reason for this could be either that the stainless steel corroded or that the cladding contained defects and did not offer full coverage protection of the carbon steel beneath (ACI 2010).

The application of external coatings to reinforcement bars have become commonly utilized as a preventative measure against corrosion. The most common coating material utilized is epoxy (ACI 2010). Over 600,000 tons of epoxy coated reinforcement is manufactured in the United States and Canada every year (McDonald 2016). By epoxy coating steel a physical barrier is created that prevents interaction of oxygen, moisture, chlorides, and electrical currents with the bar, thus reducing corrosion potential (ACI 2010). A principle concern when utilizing epoxy coated rebar is to prevent damage to the coating during manufacturing, transportation, storage, and installation of the bars. It is common for coatings to be damaged and exposing the highly corrosive steel underneath (ACI 2010). Special care must be taken before installation to repair any parts of the coating that may have been damaged (McDonald 2016). Epoxy coated ties and chairs should be utilized in conjunction with the coated rebar to prevent damage to the epoxy coating during installation and to further reduce the risk of corrosion to all steel which is to be embedded in the concrete (ACI 2010). Other forms of coatings are also used to prevent corrosion. Metallic coatings of nickel, cadmium, and zinc all offer corrosion prevention to reinforcing bars. However, only galvanized zinc bars are readily available (ACI 2010). When utilizing zinc coated reinforcement all reinforcement bars and associated steel assembly components in the concrete need to be coated to prevent galvanic coupling that could result between coated and uncoated steel (ACI 2010).

2.3.3.3 Corrosion Inhibitors

The history of corrosion related deterioration of reinforced concrete structures has led to the creation and technological advancement of corrosion preventing admixtures. When many reinforced concrete structures where created in the 1950’s, the potential for reinforcing steel to corrode was not understood (Cicek 2014). It wasn’t until the 1970’s, when corrosion deterioration began to manifest, that corrosion prevention measures started to become apparent in the form of structure design and concrete mixture design (Cicek 2014). In 1978, the first corrosion inhibiting admixtures became available for usage as DCI corrosion inhibitor (Jeknavorian et al. 1995). The ability for these admixtures to inhibit corrosion is due to the active ingredient calcium nitrite (Lane et al. 2003). Calcium nitrite has the ability to repair iron oxidization located on the surface of the steel. Doing this prevents the development of expansive corrosion products that are at primary fault for concrete deterioration such as spalling and cracking (Jeknavorian et al. 1995, ACI 2010). This process is the result of corrosive ferrous ions being converted into ferric oxide with the addition of calcium nitrite. This conversion can be expressed as the following formula (Jeknavorian et al. 1995):

$$2\mathrm{Fe}^{2+} + 2\mathrm{OH}^{-} + 2\mathrm{NO_2}^{-} \rightarrow 2\mathrm{NO} + \mathrm{Fe_2O_3} + \mathrm{H_2O}$$

Calcium nitrite admixtures are added to concrete during batching in the form of a liquid solution. The required dosage to provide sufficient protection is dependent upon the chloride concentration that the concrete structure will be exposed to (Jeknavorian et al. 1995). The manufacturer suggested DCI dosage rate for various chloride exposure levels is shown in Table 2.3 (GCP Applied Technologies 2020).

Table 2.3: DCI dosage rates for various levels of chloride exposure

DCI, gal/yd^3 (L/m^3)	**Chloride, lb/yd^3 (kg/m^3)**
2.0 (9.8)	6.0 (3.6)
2.5 (12.3)	8.0 (4.7)
3.0 (14.8)	9.9 (5.9)
3.5 (17.5)	11.5 (6.8)
4.0 (19.7)	13.0 (7.7)
4.5 (22.2)	14.1 (8.3)
5.0 (24.6)	15.0 (8.9)
5.5 (27.1)	15.6 (9.2)
6.0 (29.5)	16.0 (9.5)

Calcium nitrite corrosion inhibitors can be utilized in almost all steel reinforced, prestressed, or post tensioned concrete that are expected to come in contact with chlorides from marine environments or deicing salts during their service life (W.R. Grace & Co. 2013). The use of sufficient amounts of calcium nitrite has shown to increase service life of concrete by reducing corrosion potential and keeping corrosion rates low (W.R. Grace & Co. 2013).

The addition of calcium nitrite to a mixture does produce several side effects that will affect the characteristics of the concrete. Even in low concentrations, it produces an accelerating effect that reduces setting times. The use of a set retarding admixture can be utilized to counteract this, but in instances of cold weather concreting the accelerating effects can be beneficial (W.R. Grace & Co. 2013). Along with this, calcium nitrite may moderately reduce entrained air content which must be accounted for to reduce deterioration from freezing and thawing (W.R. Grace & Co. 2013). Use of various cements, aggregates, and additional admixtures can result in different effects when combined with corrosion inhibitor. For this reason, it is almost always suggested that multiple trial batches be produced prior to the first placement of calcium nitrite concrete so that impacts on performance are anticipated.

2.4 Corrosion Policies

To protect investments into infrastructure, it is common for State Highway Agencies and other municipalities to specify the use of corrosion prevention practices in new concrete construction in corrosive areas. The most common corrosive areas are marine environments where chlorides can be transferred through both water and air. Implementing these practices is the first proactive step to ensuring that structures reach their intended service life and avoid costly maintenance. Some common design and material requirements are the specification of Class AA mixtures, maximum *w/cm*, use of pozzolans to reduce the permeability of the concrete, increased cover depth, use of epoxy coated rebar, and corrosion inhibiting admixtures. The end goal of these specifications is for structures to see an increased service life with lower maintenance costs to offset the initially higher cost of implementing these mitigation practices.

2.4.1 NCDOT Prevention Methods

The NCDOT's Structures Management Unit (SMU) Manual provides the specific corrosion protection measures that are required in all concrete construction throughout the state. The currently utilized corrosion policy was implemented in 1999 and is applicable to all new construction, rehabilitation, and repair. The chloride exposure risk of the location in which the structure is to be constructed dictates the level of corrosion protection measures necessary. The greatest need for implementation of these measures can be seen along the eastern side of the state that forms a coastline with the chloride rich waters of the Atlantic Ocean. The manual defines two areas spanning the entirety of the eastern coastline as chloride rich marine environments requiring special corrosion protection measures. The boundaries of these areas can be seen in Figure 12-29 of the SMU Manual as well as in Figure 2.4 of this report indicated by a blue and a red line (NCDOT 2019). All area east of the blue line but west of the red line is designated as a corrosive area. The area east of the red line is designated as highly corrosive area.

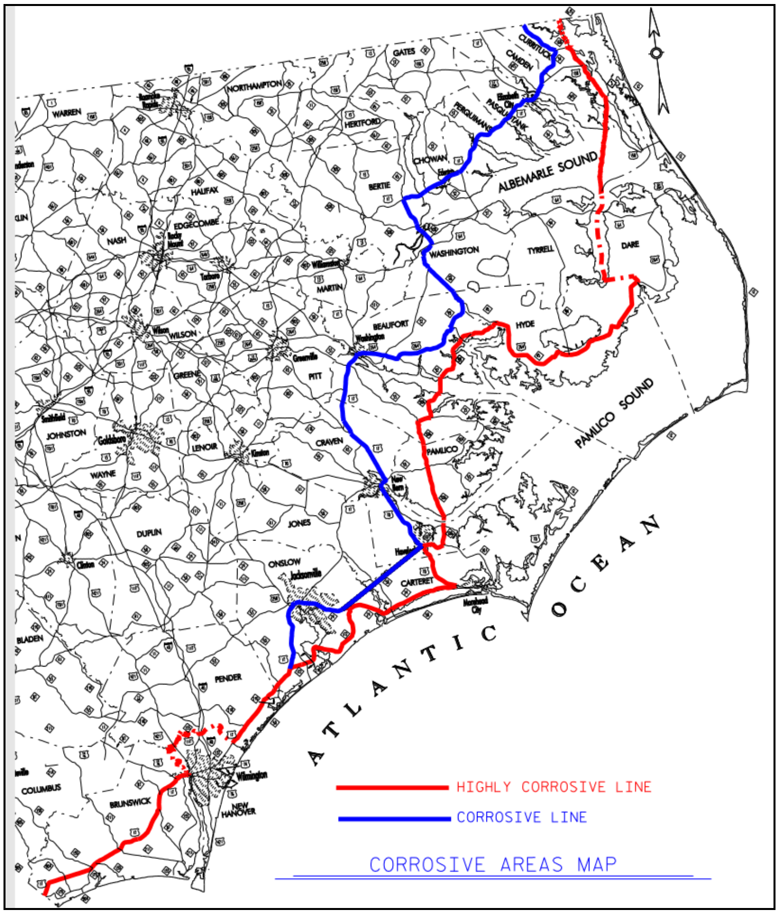


Figure 2.4: NCDOT corrosive areas map (NCDOT 2019)

Corrosion protection measures put in place for concrete structures west of the blue corrosive line are generally limited to bridge decks that see a large application of road salt or deicer application. In these areas, the use of some epoxy coated reinforcing steel is common as well as mineral admixtures to reduce concrete permeability. Corrosion protection for concrete in the corrosive area (on or east of the blue line) is only applicable to bridge members at stream crossings located within 15-feet of the mean high tide. Calcium nitrite corrosion inhibitor is specified for use in all bridge members and, given the situation, mineral admixtures may be required as well (NCDOT 2019). A minimum of one corrosion protection measure is required in all concrete construction in the highly corrosive area (on or east of the red line) regardless of any circumstances or proximity to water. The corrosion prevention measures that may be specified include (NCDOT 2019):

- Increased cover thickness
- Epoxy coated reinforcement steel
- Corrosion inhibiting admixtures
- Use of SCM’s (fly ash, silica fume, granulated blast furnace slag)
- Class AA concrete

Stay-in-place metal forms are not permitted for use in any of the corrosive site areas. If for constructability reasons metal forms need to be used, they must be removable (NCDOT 2019).

The need for corrosion prevention measures varies greatly depending on location, proximity to tide, and structural member type. For this reason, a comprehensive guide to these measures has been provided by the SMU in the form of the “Determination of Corrosive Protection” flow chart. The flow chart in Figure 12-30 of the SMU Manual is applicable to all concrete structures in the state, not only those in corrosive sites. A reconstructed version of this chart has been included and can be seen in Figure 2.5 (NCDOT 2019).

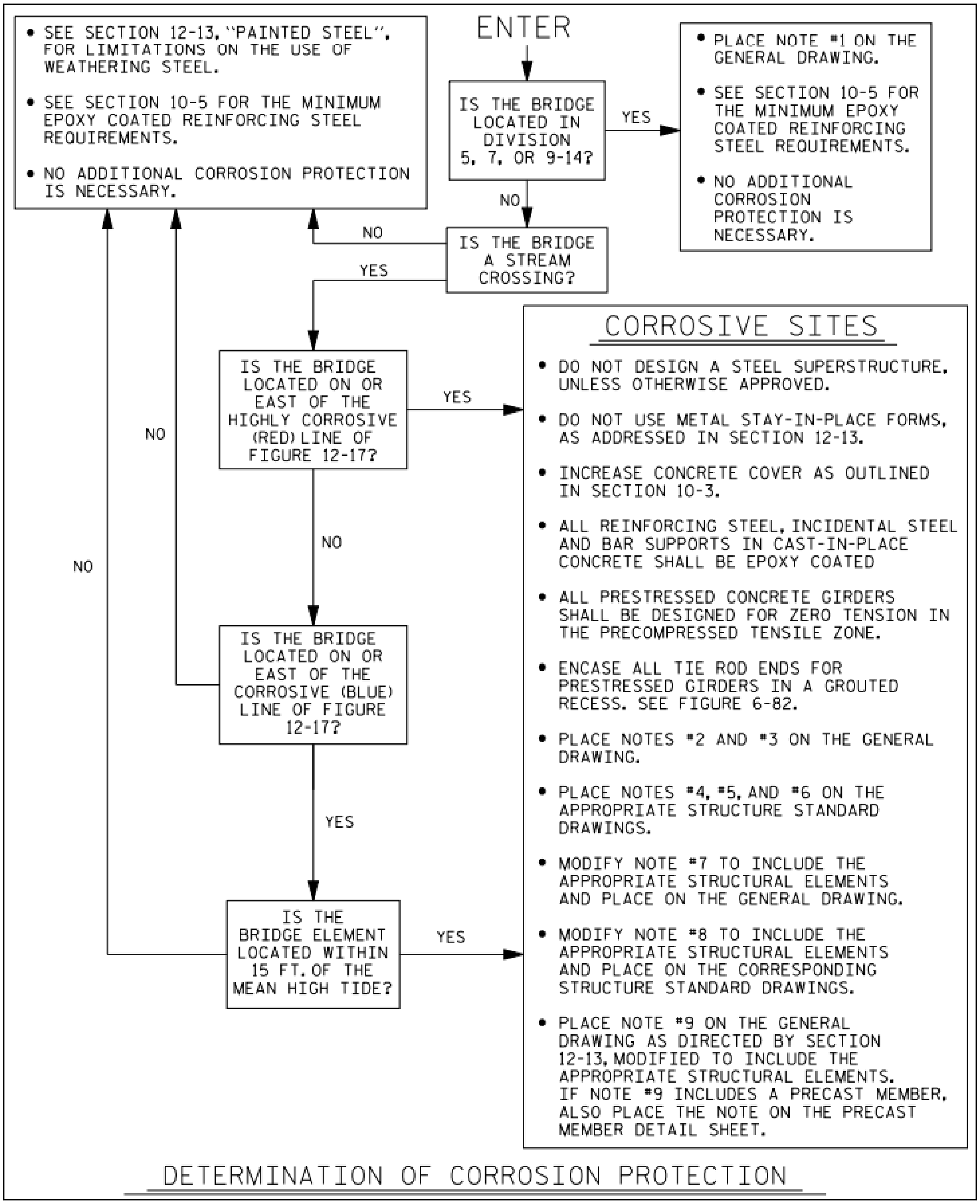


Figure 2.5: Determination of corrosion protection flow chart (NCDOT 2019)

The notes that are applicable to the determination of corrosion protection flow chart seen in Figure 2.5 have been included as follows:

Note #1: The class AA concrete in the bridge deck shall contain fly ash or ground granulated blast furnace slag at the substitution rate specified in Article 1024-1 and in accordance with Articles 1024-5 and 1024-6 of the Standard Specifications. No payment will be made for this substitution as it is considered incidental to the cost of the Reinforced Concrete Deck Slab.

Note #2: All metallized surfaces shall receive a seal coating as specified in the Special Provision for Thermal Sprayed Coatings (Metallization).

Note #3: Class AA concrete shall be used in all cast-in-place columns, bent caps, pile caps, and footings, and shall contain calcium nitrite corrosion inhibitor. For Calcium Nitrite Corrosion Inhibitor, see Special Provisions.

Note #4: Prestressed concrete girders are designed for 0 psi (0 MPa) tension in the precompressed tensile zone under all loading conditions.

Note #5: Precast panels shall be designed for an allowable tensile stress of 0 psi (0 MPa) in the precompressed tensile zone under all loading conditions.

Note #6: The water/cement ratio for concrete piles shall not exceed 0.40.

Note #7: All bar supports used in the (barrier rail, parapet, sidewalk, deck, bent caps, columns, pile caps, footings) and all incidental reinforcing steel shall be epoxy coated in accordance with the Standard Specifications.

Note #8: Prestressed concrete (girders, precast deck panels, cored slab units, piles) shall contain calcium nitrite corrosion inhibitor. See Special Provisions for Calcium Nitrite Corrosion Inhibitor.

Note #9: The concrete in the (columns, bent caps, pile caps, footings, and/or piles) of Bent No. ____ shall contain silica fume. Silica Fume shall be substituted for 5% of the portland cement by weight. If the option of Article 1024-1 of the Standard Specifications to partially substitute Class F fly ash for portland cement is exercised, then the rate of flay ash substitution shall be reduced to 1.0 lb (1.0 kg) of fly ash per 1.0 lb (1.0 kg). No payment will be made for this substitution as it is considered incidental to the various pay items.

2.4.2 Other State Highway Agency Prevention Methods

Corrosion deterioration in coastal areas is a well-known problem that every state highway agency with assets in these areas faces and aims to address. Within the continental United States there are twenty-one states that have boarders with coastlines along the chloride rich waters of the Atlantic Ocean, Pacific Ocean, and the Gulf of Mexico. SHAs in Florida, Georgia, and South Carolina each have individual specifications for how to mitigate corrosion related deterioration. These specifications are outlined in a bridge/structures design manual that is published by each SHA. These states policies were chosen to compare with North Carolinas due to their close geographical proximity and relatively advanced specifications. By comparing the specifications of these states with North Carolinas, an assessment can be made as to which policies utilize the most prevention methods. These specifications for each state are summarized in Table 2.4. A ‘YES’ or ‘NO’ in this table indicates if the use of these prevention methods are specifically stated as allowed for use in each states respective design manual. A ‘YES’ or ‘NO’ does not indicate that these methods are utilized in every situation or not used at all. It is often left to the engineer’s discretion as to which measures to utilize.

Table 2.4: Corrosion protection methods utilized by state highway agencies

State	**Corrosive Zones**	**Reinforcement**			**Admixtures**					**Design**	
		Epoxy	Galvanized	Stainless	CI	Fly Ash	Silica Fume	Slag	Metakaolin	Increased Cover	Max. w/cm
Florida	Extremely Corrosive, Moderately Corrosive, Slightly Corrosive	NO	NO	NO	YES	YES	YES	NO	YES	YES	NO
North Carolina	Highly Corrosive, Corrosive	YES	NO	NO	YES	YES	YES	YES	NO	YES	YES
South Carolina	Coastal Counties	NO	YES	NO	NO	NO	NO	NO	NO	NO	NO
Virginia	None	NO	NO	YES	YES	NO	NO	NO	NO	NO	NO

Florida has a large coastline on both the Atlantic Ocean and the Gulf of Mexico. Due to this, the FDOT has a very detailed specification outlining requirements for protection measures and environmental conditions. The FDOT Structures Design

Guidelines allow for the District Material Engineer or an Environmental/Geotechnical Consultant to determine the environmental classification applicable to all new bridge sites (FDOT 2019). Bridge locations are classified as being slightly aggressive, moderately aggressive, or extremely aggressive. Depending on the situation, the superstructure and the substructure may be designated as separate classifications, but the substructure will never be classified as less severe than the superstructure (FDOT 2019). Requirements for these classifications are as listed (FDOT 2019):

> *1. For structures over or within 2,500 feet of a body of water with chloride concentrations in excess of 6,000 ppm, both superstructure and substructure will be classified as extremely aggressive.*
>
> *2. For structures over any water with chloride concentrations of 2,000 to 6,000 ppm, the substructure will be classified as extremely aggressive. Superstructures located at 12 feet or less above the mean high-water elevation will be classified as extremely aggressive. Superstructures located at an elevation greater than 12 feet above the mean high-water elevation will be classified as moderately aggressive.*
>
> *3. For structures within 2,500 feet of any body of water with a chloride concentration of 2,000 to 6,000 ppm, but not directly over the body of water, the superstructure will be classified as moderately aggressive. The substructure will follow the non-marine criteria.*

It has been found that for superstructures within 2,500 feet of the coastline there is an increased rate of chloride intrusion at a concrete depth of two inches (FDOT 2019). Outside of this distance chloride intrusion decreases rapidly. This understanding is utilized when developing environmental classification for bridges. The rate of chloride intrusion and its relation to environmental classification can be seen in Table 2.5 (FDOT 2019).

Table 2.5: FDOT chloride intrusion rate relation to environmental classification

Chloride Intrusion Rate	Classification
≥ 0.016 lbs/cy/year	Extremely Aggressive
< 0.016 lbs/cy/year	Moderately Aggressive

For each environmental classification, there are specific corrosion prevention measures required. Design and construction measures include specific concrete class and increased concrete cover. There is no limit or range for the *w/cm* that can be utilized. The combined or individual use of calcium nitrite, silica fume, metakaolin, or fly ash are commonly specified as admixtures. With use of these preventative measures, all structures are designed with provisions to allow them to exceed the LRFD 75-year service life (FDOT 2019).

South Carolina DOT's policy does not specify what specific prevention measures can or cannot be utilized on most elements of its bridges. Instead, in coastal counties the State Bridge Design Engineer is given the ability to determine if there is a need for corrosion protection and what methods should be utilized to create the protection system. There is no guidance offered for what conditions or levels of chlorides constitute a corrosive environment. The only specified corrosion protection system is the use of Class 4000 concrete with galvanized reinforcement bars on bridge decks (SCDOT 2006).

2.5 Corrosion Testing

Testing to demine the effects of corrosion requires an analysis of both the corrosion rate and the potential for corrosion. Both play a large role in the magnitude of the electrochemical reaction that develops when a metal is subjected to an electrolyte solution (Poursaee 2016). The potential for corrosion is better known as the half-cell potential, which is a thermodynamic measurement of the ability to separate electrons from the metal (Poursaee 2016). This is different from the corrosion rate which is determined through the corrosion current and cannot be directly measured but estimated from the corrosion potential (Poursaee 2016).

2.5.1 Laboratory Testing

To determine if corrosion is in fact due to chloride ingress, powder samples of the concrete structure can be taken and tested for chloride content in the laboratory. A common way to test hardened concrete for this is by utilizing a rapid chloride test (RCT). This test requires that a powder sample be taken from drilling into hardened concrete. It is than mixed with an acidic extraction liquid to separate any chloride ions. 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 depth of the same location allow for a plot to be generated that depicts the chloride profile. By understanding the chloride concentration within the concrete, a case can be made for if corrosion is a result of chloride ingress. If the chloride concentration is above the corrosion threshold value (CTV) than it is very likely that corrosion is occurring or will occur. Corrosion thresholds can vary wildly based upon different cement types and exposure environments. For this reason, it is important to determine a suitable CTV before the chloride concentration is considered to be the cause of corrosion (Frederiksen 2000).

Using these measurements of chloride concentrations at various depths, a diffusion coefficient can be calculated. This diffusion coefficient is instrumental in service-life prediction models, such as Life-365, by quantifying how resistant the structure is to chloride ingress. This is explained in further depth in Section 2.6.

2.5.2 Field Testing

It is difficult to visually assess the corrosion of reinforcing steel until its effects on the concrete have expressed themselves in the form of cracking, delamination, spalling, or staining. For this reason, the most common methods to assess corrosion and chloride penetration are destructive and require coring or drilling to the depth of the reinforcement. When destructive testing is not an option, condition assessment of reinforcing steel in concrete can be completed utilizing a non-destructive device.

A commonly utilized device for measuring corrosion rate and half-cell potential of reinforcing steel in concrete is the GalvaPulse manufactured by Germann Instruments. It utilizes a nondestructive polarization technique that works by inducing a short anodic current pulse into the steel (Germann Instruments 2020). By analyzing the polarization resistance over a given time and applied current it is able to estimate the corrosion rate with the Stern Geary equation for active corrosion and Faradays law of electrochemical equivalence (PCTE). It has many advantages to other methods such as speed, reliability on even wet concrete, easily operated, and measurements are possible on curved and uneven surfaces. A limitation of this equipment is that a connection must be made to the reinforcing steel to polarize the metal. In many cases this requires destructively removing concrete cover to access the reinforcing steel. From the point of connection to the steel a relatively small area can be tested for corrosion rate. This makes it unideal for testing large area due to the need to create multiple access points to the reinforcing steel.

The iCOR device is a more recently developed non-destructive evaluation device produced by Giatec Scientific Inc. The iCOR has the ability to detect corrosion, measure corrosion rate, determine half-cell potential, measure surface resistivity, and evaluate corrosion potential (Giatec Scientific Inc. 2020). This device is unique in that it does not require a connection to the reinforcing steel to be made like is necessary to produce the same results from similar corrosion testing devices. Being that it is a nondestructive testing (NDT) device it can be utilized in a wide variety of ways and on critical elements, such as prestressed concrete, that a destructive testing device could not. The iCOR does not have the ability to test epoxy-coated reinforcement for corrosion rate or potential.

The iCOR device functions by interpreting the electrical response of the reinforcing steel to a directed current pulse. Four prongs on the Connectionless Electrical Pulse Response Analysis (CEPRA) device are utilized to polarize the reinforcing steel (Giatec Scientific Inc. 2019). The basic concept behind the device is to have two prongs produce an AC current with a sweeping frequency while the other two measure the voltage response from the steel. Steel that is corroding produces a different response than non-corroding steel. This difference is illustrated in Figure 2.6 where it is seen that the voltage of the non-corroding bar decreases with high frequency while the corroding bars voltage stays constant at all frequencies (Giatec Scientific Inc. 2019).

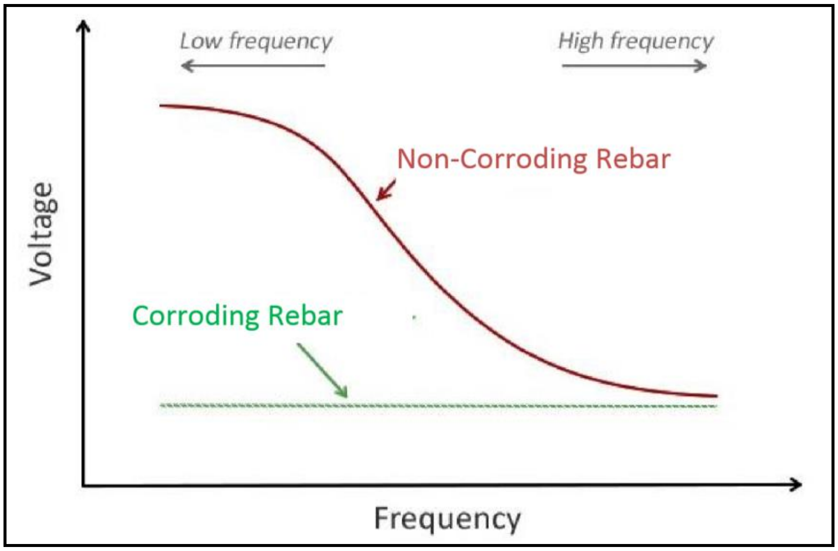


Figure 2.6: Effect of frequency on voltage of corroding and non-corroding steel

This analysis of the effect of frequency on voltage of corroding and non-corroding reinforcement is how the iCOR device produces a measurement of the corrosion processes happing within the concrete structure. The corrosion rate measurement produced by this device is reported in units of both µA/cm^2 or µm/yr. The classification of these corrosion rates can be seen in Table 2.6 (Giatec Scientific Inc. 2020).

Table 2.6: Corrosion rate classification

Color Code	**Corrosion Rate**		**Classification**
	µA/cm^2	**µm/yr**	
🟩 Green	<1	<10	Passive/Low
🟨 Yellow	1-3	10-30	Moderate
🟧 Orange	3-10	30-100	High
🟥 Red	>10	>100	Severe

Due to the ability to measure corrosion rate in a truly nondestructive method the iCOR was selected as the device used in all field work associated with this project. The use of the GalvaPulse would require multiple points of damage to the bridge and the potential for detrimental effects resulting from damage to a prestressed stand would limit corrosion rate mapping to only cast in place concrete elements.

2.5.3 Techniques to Verify Presence of Corrosion Inhibitor

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 employ these methods as a form of quality control 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 to do due to the difficulty of bringing analytical instruments into the field. Previous methods used to test for corrosion inhibitor have been simplified to address this issue. These simplified methods can detect the presence of calcium nitrite but cannot report the concentration (Jeknavorian et al. 1995). The procedure for this method involves placing a small sample of fresh concrete into a container and mixing with a reagent by shaking vigorously. After approximately five minutes, the solution will turn pink, indicating the presence of calcium nitrite (Jeknavorian et al. 1995).

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 being analyzed calorimetrically (Jeknavorian et al. 1995). This method is still widely utilized, but two other methods have since been developed. One method utilizes ion chromatography while the other relies on a polarographic analysis (Jeknavorian et al. 1995). These methods all have advantages and disadvantages based upon their degree of accuracy, ease of use, speed, and cost of equipment. These are summarized in Table 2.7 (Jeknavorian et al. 1995).

Table 2.7: Comparison of nitrite detection methods

Method	Procedure Overview	Advantages	Disadvantages
Ion Chromatography	Concrete extract is placed into anion exchange column where sample ions (chloride, nitrate, nitrite, sulfate) selectively interact with eluent ions for exchange sites on the resin before being eluted from the column. The extracted ions are detected with a conductivity detector.	• High accuracy • Detects other soluble ions (nitrate, chloride, sulfate) • No pretreatment process	• Moderately expensive equipment • System equilibrium is time consuming
Polarography	The selective electrochemical reduction of nitrite-diphenylamine complex is the bases for determining nitrite content. A polarography measures the current produced which can be compared to a standard solution to calculate calcium nitrite content.	• Most sensitive • Fast analysis time • Moderate equipment cost	• Easily contaminated • Assumes current response if due strictly to nitrite
Colorimetry	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 then measured by a spectrophotometer.	• Ease of use • Inexpensive equipment	• Dilutions are time consuming • Assumes photometric response is due strictly to nitrite

2.6 Corrosion Modeling

The term service life commonly utilized to express how durable a structure is. However, these terms differ in meaning as durability is a measure of performance over a given time while service life is a measure of time within a given level of performance (ACI 2017). Service life prediction modeling is a tool that quantifies the impact on performance that deterioration mechanisms will have on given materials or design attributes (ACI 2017). It can be utilized either in the design phase to model effects of deterioration or as a tool to evaluate performance long after construction. The use of these modeling techniques is an important step to developing sustainable and economic concrete structures against factors that are known to limit service life. Since corrosion is known to be a common deterioration method in coastal environments, service life prediction models that emphasis its impacts have been developed. These models allow for 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).

2.6.1 Overview

Many methods are utilized to model the effect of chloride ingress on corrosion. The purpose of these modeling efforts is generally to determine the service life of a concrete structure until a corrosion related failure occurs. Most methods utilize diffusion coefficients and CTV’s to determine the rate at which chloride ingress occurs and time for corrosion to propagate (Bhattacharjee and M. 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 valid. Modeling is important because it allows for the ability to forecast the implications of utilizing different mixture designs and corrosion prevention methods in an easy and low-cost way.

2.6.2 Methods

Vast numbers of researchers and developers in the field of concrete infrastructure have developed methods of service life modeling of corrosion deterioration. Of these methods several have been made available for commercial use. Some of the widespread, commercially available, software that is available include Stadium and Life-365. The means of analysis employed by these models varies greatly, and consequently, there can be large variations in results (Bentz and Thomas 2018).

Life-365 was developed as a way to plan and design concrete structures that may be exposed to chlorides and impacted by corrosion (Bentz and Thomas 2018). It is often utilized to analyze the benefits specific corrosion prevention measures have on the service life of a structure. It assumes that corrosion due to chloride ingress is the failure mode for which to determine the end of the service life. Like many models, its analysis is based upon the error function solution of Fick’s Second Law of Diffusion. 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 longevity of reinforced concrete. It accounts for some of the complex phenomenon 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). By making assumptions on the cost of initial construction and repair costs over the service life, the software is able to conduct a life cycle cost analysis that may be used to compare any number of different corrosion prevention methods.

STADIUM (Software for Transport and Degradation in Unsaturated Materials) is developed by SIMCO Technologies, who is an engineering firm dedicated to durability of concrete structures. Their STADIUM software is marketed as the only accurate solution to predicting long term performance of reinforced concrete structures recognized by the U.S. Department of Defense (SIMCO Technologies 2020). It utilizes finite element analysis to predict concrete deterioration over the time period before corrosion of reinforcing steel corrodes. The modeling method is not based upon Fick's Second Law of Diffusion and instead is based on, "ionic transport and reaction modeling in saturated and unsaturated concrete" (SIMCO Technologies 2020). This method emphasizes the unique interaction that takes place between contaminants and hardened cement paste that is dependent upon the type of cement, SCM's, and aggregates utilized (SIMCO Technologies 2020). It allows for inputs to analyze the influence of the geometry of the structure, materials utilized, exposure conditions, multiple degradation phenomena, and current repairs in place (Samson 2014). The advanced nature of this software, and the precision of its required inputs, can make it expensive and time intensive to utilize effectively.

Compared to STADIUM, Life-365 is more cost effective and user-friendly modeling option able to analyze critical factors known to influence corrosion. The cost savings of utilizing Life-365 as opposed to STADIUM are substantial, since Life-365 is available for use as a free software program and STADIUM's licensing costs (as well as costs associated with the supporting tests to obtain the required inputs) can be too high for routine testing use. Although Life-365 has some known limitations (many of which are accounted for by STADIUM), its use has proven to be sufficient in many corrosion related studies and will be utilized for all service life prediction in this thesis.

2.6.3 Life-365

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). Master Builders Technologies, GCP Applied Technologies, and the Silica Fume Association, with the intention to develop a standardized model to predict corrosion deterioration, funded the consortium. This failure mechanism is the basic assumption of the software and is therefore not applicable in all applications, such as carbonation due to reduction of pH level in concrete (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 allows for chlorides to readily access the reinforcing steel and cause early onset of corrosion. Cracking is an unpredictable, situationally based condition and for this reason, it would be difficult to model accurately. It is however an intuitive and customizable tool that has been used successfully in many applications to evaluate concrete mixtures and corrosion mitigation techniques.

Life-365 defines the service life of a structure by the time total time necessary for chlorides to penetrate and for corrosion to first begin, and then to 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 influenced by 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 initiation period is increases with larger cover and threshold values and decreases with larger chloride concentration and diffusion coefficients (Hodhod and Ahmed 2013). Because of how this modeling system estimates service life, determining an accurate diffusion coefficient is critical to achieve the best results from it.

Lifecycle cost prediction by Life-365 is accomplished following the procedure in ASTM E917-05, “Standard Practice for Measuring Life-Cycle Costs of Buildings and Building Systems” (Ehlen et al. 2009). This life cycle cost (LCC) method sums the relevant costs from initial purchase to decommissioning of a building of building system over a pre-determined time period (ASTM 2005). Life-365 LCC analysis estimates the costs of initial construction, operational barriers, and repairs over the design service life (Ehlen et al. 2009). The software accounts for inflation and discount rates by utilizing published values used by United States government agencies to conduct LCC analysis but can be changed by the user to suit their needs (Ehlen et al. 2009).

CHAPTER 3: METHODOLOGY

3.1 Introduction

To assess if the current corrosion mitigation policy in place by the NCDOT is adequately supporting the desired service life of bridges, the current condition of bridges constructed using this policy must be evaluated. Bridges selected for assessment as part of this study have all met several criteria for age, construction, and environmental location that facilitate the evaluation of potential correlations between the levels of corrosion prevention methods utilized and their effectiveness in a given environment. The condition assessment of all bridges selected includes both field and laboratory testing. This process consists of a visual inspection and the determination of the current corrosion rates, concrete resistivity, chloride content, diffusion coefficients, and concentration of corrosion inhibitor. The results from evaluation of these condition assessments will support service life modeling efforts and estimation of the remaining service life of the bridges. Service life modeling aims to address whether the policy is sufficient or not, if the areas designated as corrosive and highly corrosive are warranted, if sufficient amounts of calcium nitrite have been utilized, and predictions on if bridges will meet their intended service life of 50-years.

3.1.1 Bridge Selection Protocol

The selection of appropriate bridges to support this analysis is critical to producing test results that adequately represent the condition of bridges constructed under NCDOT's current corrosion mitigation policy. Several criteria were utilized to select bridges that have several desirable traits but have been exposed to varying environmental conditions within the designated corrosion zones along the NC coastal area, as well as just outside the currently designated corrosion zones. Doing so will allow correlations to be made between the effectiveness of the corrosion mitigation methods utilized for the given environmental conditions and the performance of each bridge. Along with environmental conditions, an ideal bridge for selection should have been constructed within ten to fifteen years so that environmental conditions known to promote corrosion would have had time to begin to impact the reinforced concrete. Commonalities between the bridges, such as construction types, accessibility, and proximity to water were also considered. All bridges within the highly corrosive zone and the corrosive zone were initially considered for selection, but only bridges located within 20 miles west of the blue corrosive line (shown in Figure 2.4) were determined to be most appropriate for this study.

3.1.2 Bridge Selection Process

The bridge selection process began with the research team identifying all bridges within both the highly corrosive and corrosive zones that were constructed under the corrosion mitigation policy. A total number of 90 bridges have been constructed since 1999 within the highly corrosive zone and corrosive zone. The location of these bridges can be seen in Figure 3.1. In Figure 3.1, blue marks indicate bridges constructed in the highly corrosive zone and red marks indicate bridges constructed in the corrosive zone. The criteria were then refined to include bridges that were within the desired age of 10 to 15 years old and also within the corrosive zones. The locations of these bridges can be seen in Figure 3.2. From these bridges, five were selected based upon having similar construction, reasonable accessibility, and proximity to water. This information was determined through review of recent inspection reports and the original construction drawings for each bridge. Two of the bridges selected are located in the corrosive zone while three are located in the highly corrosive zone. The proximity between the bridges was also considered to allow efficiency in field testing.

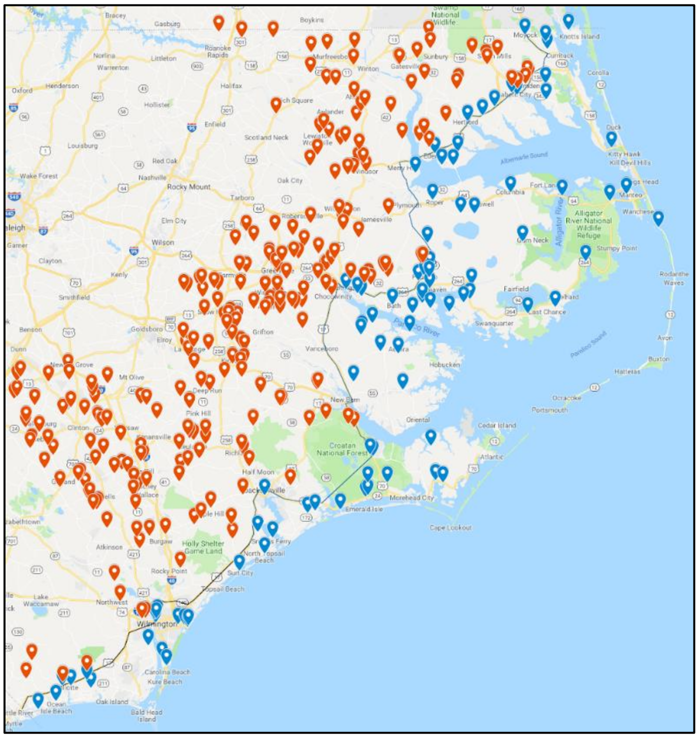


Figure 3.1: All bridges currently located within the corrosive zones

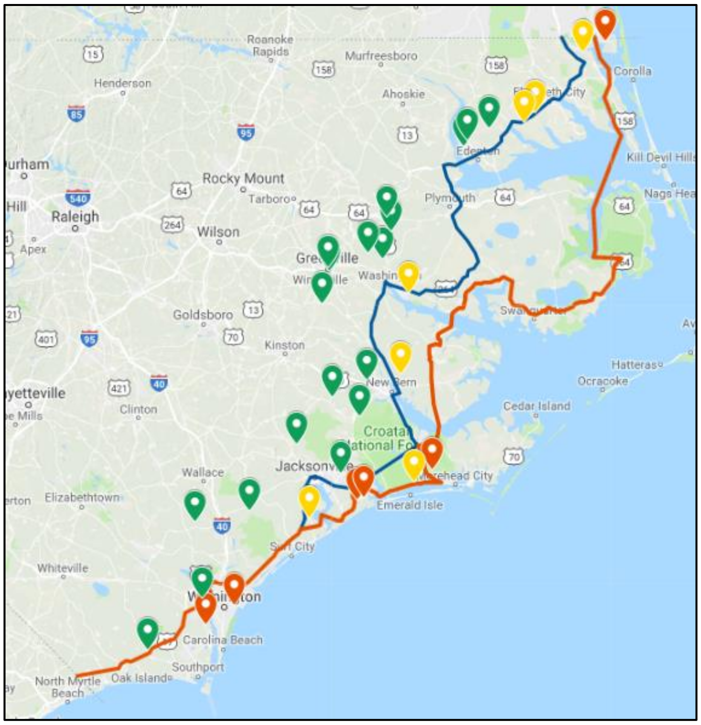


Figure 3.2: All bridges meeting selection criteria

Bridges selected for analysis were all within close proximity, no more than 30-miles, to Jacksonville, NC, Wilmington, NC, and the Virginia-North Carolina boarder. The location of the bridges analyzed in the Jacksonville, NC area, Wilmington, NC area, and near the Virginia boarder can be seen in Figure 3.3, 3.4 and 3.5 respectively. The six-digit number displayed on these figures is the NCDOT structure. As directed by members of the NCDOT project steering committee, no bridges were selected for analysis outside of the corrosive zone due to their lack of corrosion related design provisions.

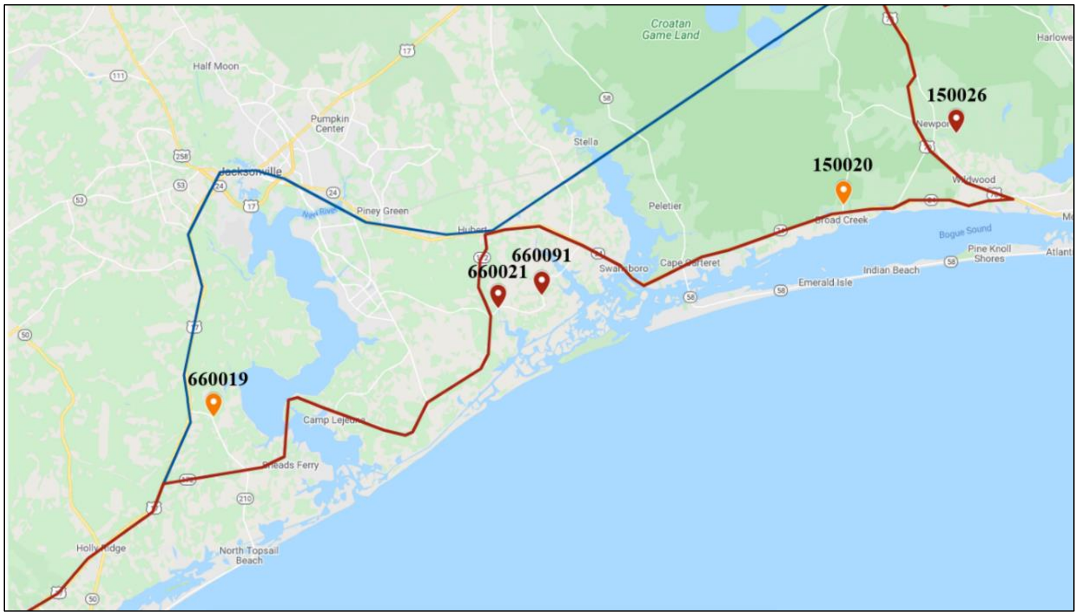


Figure 3.3: Location of bridges selected in Jacksonville, NC

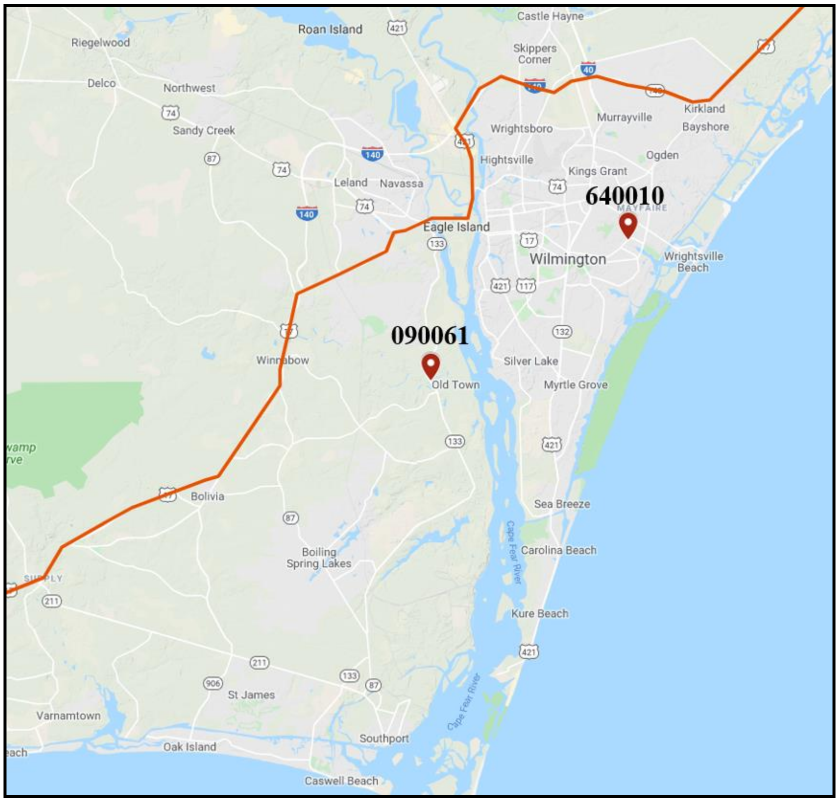


Figure 3.4: Location of bridges selected in Wilmington, NC

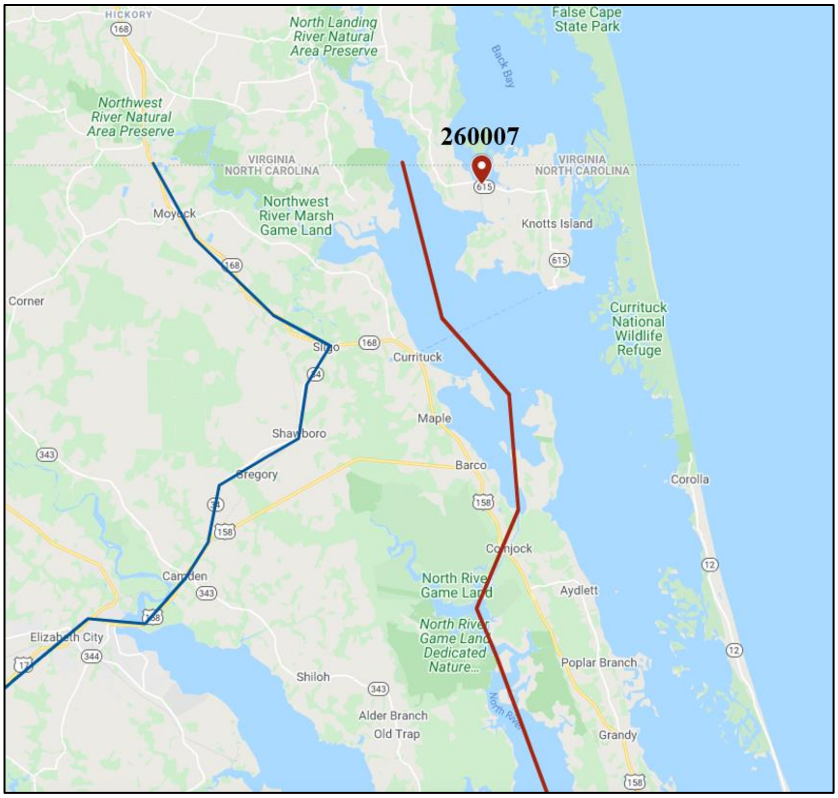


Figure 3.5: Location of Bridge selected near Virginia boarder

3.1.3 Description of Bridges Selected

A total of eight bridges were selected for testing utilizing the protocol and selection process outlined in Chapter 3.1.1 and 3.1.2 of this report. These five bridges are labeled as structure number (SN) 150020, 660019, 260007, 640010, 660021, 660091, 090061, and 150026 and their locations are shown in Figures 3.3, 3.4 and 3.5. Table 3.1 summarizes many characteristics of the bridges that were selected for the study. The bridges are organized into two groups based upon their corrosive zone designation. The two bridges in the corrosive zone are listed first and are followed by the six bridges in the highly corrosive zone. Within these two groups the bridges are listed in ascending order dependent upon their locations distance from a main body of water.

Table 3.1: Selected bridge characteristics

Structure Number	Location	Age (yr)	Corrosive Zone	Super-Structure Type	Steel Type	Sub-Structure Type	Additives/ Pozzolans	Height above Tide	Overlay
150020	East Prong of Broad Creek	12	Corrosive	Precast prestressed concrete cored slab	Epoxy	Prestressed concrete piles, Epoxy coated reinforced concrete pile cap	30.0% Fly Ash, 3.0 gal/CY Calcium Nitrite	15.2ft	None
660019	Stones Creek	11	Corrosive	Precast prestressed concrete cored slab	Epoxy	RC EBs on H-piles, RC Interior bent on PPC piles	5.0% Silica Fume, 3.0 gal/CY Calcium Nitrite	19.8ft	Epoxy reinforced concrete
260007	Corey’s Ditch	13	Highly Corrosive	Precast prestressed concrete cored slab	Epoxy	RC End and interior bents on steel pipe piles	3.0 gal/CY Calcium Nitrite, 43.0% Slag	8.5ft	Asphalt
640010	Bradley Creek	13	Highly Corrosive	Precast prestressed concrete cored slab	Epoxy	RC EB's on 16" Prestressed Concrete Piles	*Cast in Place*: 5.0% Silica Fume, 25.0% Fly Ash, 3.0 gal/CY Calcium Nitrite *Precast:* 5.0% Silica Fume, 3.5 gal/CY Calcium Nitrite	15.2ft	Asphalt

Table 3.1 continued: Selected bridge characteristics

Structure Number	Location	Age (yr)	Corrosive Zone	Super-Structure Type	Steel Type	Sub-Structure Type	Additives/ Pozzolans	Height above Tide	Overlay
660021	Bear Creek	15	Highly Corrosive	Precast prestressed concrete cored slab	Epoxy	RC End and interior bents on PPC piles	3.5 gal/CY Calcium Nitrite	12.7ft	Asphalt
660091	Parrot Swamp	13	Highly Corrosive	Precast prestressed concrete cored slab	Epoxy	RC EBs on H-piles	5.0% Silica Fume, 3.0 gal/CY Calcium Nitrite	13.9ft	Asphalt
090061	Town Creek	14	Highly Corrosive	Precast prestressed concrete cored slab	Epoxy	RC EB's on Prestressed Concrete Piles	5.0% Silica Fume, 3.0 gal/CY Calcium Nitrite	12.5ft	Asphalt
150026	Deep Creek	14	Highly Corrosive	Precast prestressed concrete cored slab	Epoxy	Prestressed concrete piles, Epoxy coated reinforced concrete pile cap	3.0 gal/CY Calcium Nitrite, 30.0% Fly Ash	7.6ft	Asphalt

Images sourced from recent NCDOT inspection reports of these bridges are shown in Figures 3.6 through 3.13. These images provide an overview of each bridge's site, accessibility, and general structure.

Figure 3.6: Bridge 150020 from north side

Figure 3.7: Bridge 660019 from north side

Figure 3.8: Bridge 260007 from west side

Figure 3.9: Bridge 640010 from north side

Figure 3.10: Bridge 660021 Bent 1 from north side

Figure 3.11: Bridge 660091 from west side

Figure 3.12: Bridge 090061 from east side

Figure 3.13: Bridge 150026 from south side

3.2 Field Testing Procedures

Field testing and visual observations of the selected bridges were completed during three visits to the NC coast. The first field visit took place in August 2019 to evaluate bridges SN 150020 and 660091. The second field visit took place in November 2019 to evaluate bridges SN 660019, 090061, and 640010. The third field visit took place in February 2020 to evaluate bridges SN 150026, 260007, 660021 and to reevaluate 150020. Elements of the bridges were selected for testing based upon accessibility, proximity to the water (potential chloride exposure), and construction type. These constraints primarily resulted in the substructure of the bridge (bent piers, bent caps, and end bents) being tested. Bridge decks could only be tested if there was no form of overlay on them. This was only the case on one bridge included in the study (SN 150020).

The work performed during each field visit included the same scope of field testing and observations. These included, a visual survey, NDT 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 that were looked for were 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) it is unlikely that many signs of corrosion have manifested to create visual distress at the time of this study.

3.2.2 Corrosion Rate and Concrete Resistivity

The testing of the current corrosion rate and concrete resistivity were completed simultaneously with the Giatec iCOR NDT device. The specific functions of the Giatec iCOR are explained in further depth in Chapter 2.5.2 of this thesis. The required apparatus to conduct this testing was provided by the manufacturer and included the iCOR measuring device, contact sponges, conductive gel, spray bottle, verification kit, and data recording app installed on a tablet. A pachometer was also utilized to identify the location of reinforcing steel. Testing was performed in accordance to the manufacturer suggested procedures outlined in the included user manual.

To begin this testing procedure, a flat reinforced concrete surface was selected to map the corrosion rates and concrete resistivity. Between the three field visits conducted the method of determining test locations varied. On the first field visit, locations were selected strategically and mapped with a high level of precision. The second and third field visits employed a two-stage method of loosely mapping a large area on the structure at first and then mapping areas indicating high corrosion rates in further detail. Although sampling procedures had varied, the testing methodology across both field visits remained unchanged.

Testing at each location began with initializing and synchronizing the measurement device and tablet app. The measurement device and app were connected to each other via Bluetooth. The accuracy of the iCOR measurement device was than verified using the manufacturer suggested methodology and the provided verification kit. Once a successful verification was completed the electrodes were prepared to begin testing. The preparation involved placing one to two drops of conductivity gel into the base of each electrode. Contact sponges were made moist with water and then inserted into each of the electrodes. There was no excess water flowing from the contact sponges, only enough moisture to keep them damp over the course of several tests.

After the device was physically initiated, a new project was created within the app and the measurement parameters were configured to include grid size, cover thickness, rebar size, and units of measurement. After all parameters were entered, a test grid was generated. From this grid a specific node was selected for testing. To correlate nodes to the testing area, the embedded reinforcement at the location was mapped utilizing a pachometer. Locations of reinforcing steel were mapped on the surface of the concrete with the use of chalk. With the iCOR measurement device pressed firmly against the concrete surface the corresponding grid point was selected on the tablet to provide access to the measurement page. The ‘Measure’ button was then pressed, and the test began. The test took several seconds to complete, and when complete, the results were instantaneously presented on the tablet screen. For each corrosion rate measurement, a fitted curve for the voltage measured over the time of the test was displayed along with its R^2 value. This R^2 value indicates how scattered the data points are which can lead to an interpretation of it the test was accurate. This process was repeated for the remaining locations on the grid to create a visual mapping of the corrosion rates within the area.

3.2.3 Powder Sample Acquisition

Powder samples were removed from several locations on each bridge using a rotary hammer drill. Sampling locations were determined based upon proximity to water, bridge element, accessibility, and areas with a high corrosion rate determined by prior testing with the Giatec iCOR NDT device. Prior to drilling, reinforcing steel locations were mapped with a pachometer to ensure that the hole avoided reinforcing steel. This served to avoid contamination of the powder samples, unnecessary damage to the bridge, and damage to the drill from contacting reinforcing or prestressed steel strands. Drilling locations were each located within close proximity to where corrosion rate mapping was conducted so results from the two tests could be correlated. Powder acquired from each drilled hole location was returned to the UNC Charlotte laboratory for testing for either chloride content or corrosion inhibitor concentration. The sampling procedures and criteria for drill location for each of these are similar and utilize much of the same equipment.

The evaluation of chloride content at varying depths from the surface of the concrete allows for an analysis of the diffusion process of the concrete to be made. This analysis results in the calculation of a diffusion coefficient for the concrete. The diffusion coefficient’s importance in understanding chloride transport and in life cycle modeling is discussed in further depth in Chapter 2.1.1 of this report. 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. For example, this results in a powder sample “at three inches of depth” being comprised of the powder from depths of 2 ½-inches to 3 ½-inches, and so forth. To avoid the possibility of contamination from previously drilled depths, powder samples were taken using two drill bits of varying size. 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 a 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 the use of a vacuum or blast of compressed air before the smaller ¾-inch bit is used to drill from a depth of ½-inches to 1 ½-inches in the center of the pilot hole. 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 drill from the depth of ½-inches to 1 ½-inches that the smaller sampling bit just traveled. The remaining dust was cleaned away with the vacuum. This process of drilling a larger pilot hole before the smaller sampling hole was continued until the desired sampling depth was reached and is illustrated in Figure 3.14. In this illustration, the first three samples for 0-inch depth, 1-inch depth, and 2-inch depth are shown to have been taken (the remaining samples at depths of 3-inches, 4-inches, and 5-inches are not shown but follow the same pattern).

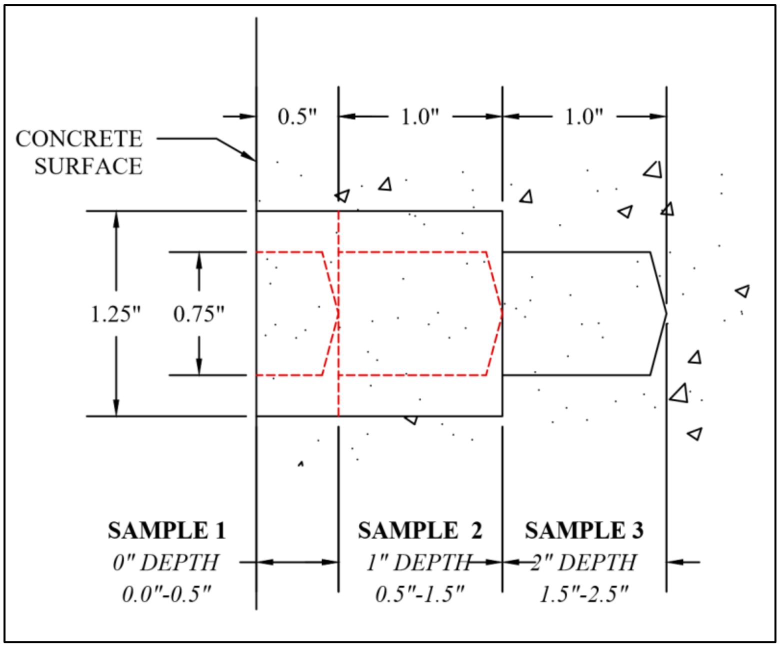


Figure 3.14: Powder sample acquisition process

The two-drill procedure utilized for this work is generally preferred over utilizing a single size drill bit because it reduces the risk of contamination between specimens. If the same sized bit is utilized there is a risk of the bit scouring the inside of the hole at previously sampled depths creating a mixture of powder that is not representative of the desired depth into the concrete. To further avoid contamination, each drill bit and dust collection tray is carefully cleaned with the vacuum or compressed air blast, isopropyl alcohol, and disposable towels to remove any residual powder that may remain on it before and after each drilling. This cleaning process is particularly important on the sampling bit.

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.

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. During transport and prior to testing, all samples remained in their individually labeled polyethylene bag to avoid contamination or damage. The results of these tests will provide the insight necessary to evaluate the as built performance of the bridges structural concrete in terms of chloride transport ability and in place concentration of corrosion inhibitor.

3.3.1 Rapid Chloride Test

The evaluation of chloride concentration in concrete powder samples from the bridges was completed with a Rapid Chloride Test (RCT) manufactured by Germann Instruments. The equipment needed for the RCT test was provided in a kit by the manufacturer and included an electrode, an electrometer, calibration liquids (chloride concentrations of 0.005%, 0.020%, 0.050%, and 0.500%), an electrode wetting agent, an electrode cleaning agent, and test vials with 10mL of a proprietary acid based extraction fluid. The manufacturer provided procedure was utilized in all sample preparation and testing.

Powder samples were obtained directly from the bridge with the procedure specified in Section 3.2.2 at three to five separate depths in each location. Per the manufacturer's recommendations for testing hardened concrete, two samples were prepared and tested for each depth of powder taken. The results from these tests where averaged resulting in the reported chloride concentration for each depth. Each test required that a powder sample be measured to 1.5g (within a tolerance of ±2.0%) and added to an individual test vial containing a proprietary acid-based extraction fluid. The vials containing powder samples were shaken vigorously for five minutes before being allowed to stand over-night (approximately 20 hours ± 4 hours) to ensure that close to 100% of the chlorides in the sample were extracted. The lid on each vial was removed and reapplied after shaking to release any gas developed during the extraction process.

Prior to each day's testing each set of vials a calibration of the electrode was conducted. With use of the manufacturer provided calibration liquids, the electrode was placed into each one in which a voltage was measured. The calibration fluids are reported by the manufacturer to produce a voltage of 100mV in the 0.005% solution, 72mV in the 0.020% solution, 49mV in the 0.050% solution, and -5mV in the 0.500% solution. Some deviation from these voltages may occur but (per the device manufacturer) is not of concern as long as the slope of the calibration curve is approximately 1.0% chloride per 100mV. The calibration curve utilized to convert the voltage readings to percent chloride by concrete weight for each test is shown in Figure 3.15.

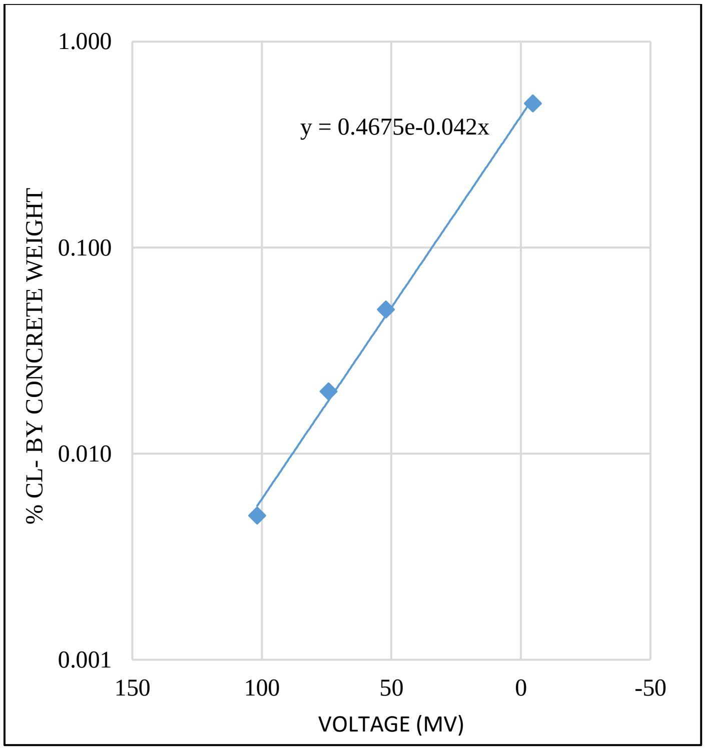


Figure 3.15: RCT calibration curve

Testing of the concrete powder samples was commenced upon completion of the electrode calibration. The electrode was cleaned with the electrode cleaning fluid (distilled water) and blotted dry before being submerged in the test vial. The electrode tip was fully submerged in the vial but not in contact with the bottom, avoiding damage caused by concrete powder granules that remain in the solution. The voltage was allowed to stabilize for two minutes before the measurement is recorded. The electrode was cleaned with the electrode cleaning fluid and blotted dry before being introduced to a new vial where the procedure was repeated. Once both test vials for each sample were tested, the mV readings for each test were averaged together and converted to chloride concentration (% chloride by concrete weight) by utilizing the calibration curve created before testing that day. Since chloride concentration is frequently reported in pounds of chloride per cubic yard of concrete the % chloride by concrete weight was multiplied by a factor of 38.15 (pcy/1% Cl) for normal weight concrete. This factor was obtained under the assumption that the concrete has a unit weight of 141.6 pcf (Cavalline et al. 2013).

3.3.2 Corrosion Inhibitor Detection

Corrosion inhibitor is dosed into the concrete mixture in varied concentrations dependent upon the mixture design and anticipated chloride exposure from the environment. Once concrete has hardened, this concentration of corrosion inhibitor can be determined 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 apparatuses necessary to conduct this test include equipment as well as reagents. The required equipment includes a hammer drill with ¾ inch bit, an analytical balance (accurate to ± 0.1mg), 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-Napthyl) 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 dilute 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 allow 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 µg/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 involved sample preparation, extraction, and nitrite determination. The powder samples acquired for analysis were taken following the procedure in Chapter 3.2.3 of this thesis. 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 that was representative of the drilled powder. The 2.0-gram sample was 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 though 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 allowing 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 converting the concentration from micrograms/mL into the more commonly utilized unit of lbs/yd^3 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/yd^3 for normal weight concrete was utilized (Cavalline et al. 2013). The fraction of nitrite in the sample was then calculated as:

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

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/yd^3 by the number of cubic feet in a cubic yard (27ft^3):

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

Using the unit weight and fraction of nitrite calculated from the previous equations the concentration of nitrite in of lbs/yd^3 of concrete was calculated by multiplying the unit weight by the fraction of nitrite:

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

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:

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

where 0.209 = Decimal percentage of NO_2^- in a corrosion inhibitor on average,
1.3 = Specific Gravity of CI,
8.33 lbs/gal = Weight of water per gallon.

3.4 Service Life Modeling

Modeling the effects of corrosion on bridge components selected for this analysis was performed using the 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). Input values specific to each structural element tested allow for the impact of environmental conditions, concrete performance properties, and corrosion mitigation practices to be explored. Data collected from field and laboratory testing was utilized whenever possible so that the modeling effort could be completed with as few assumptive inputs as possible.

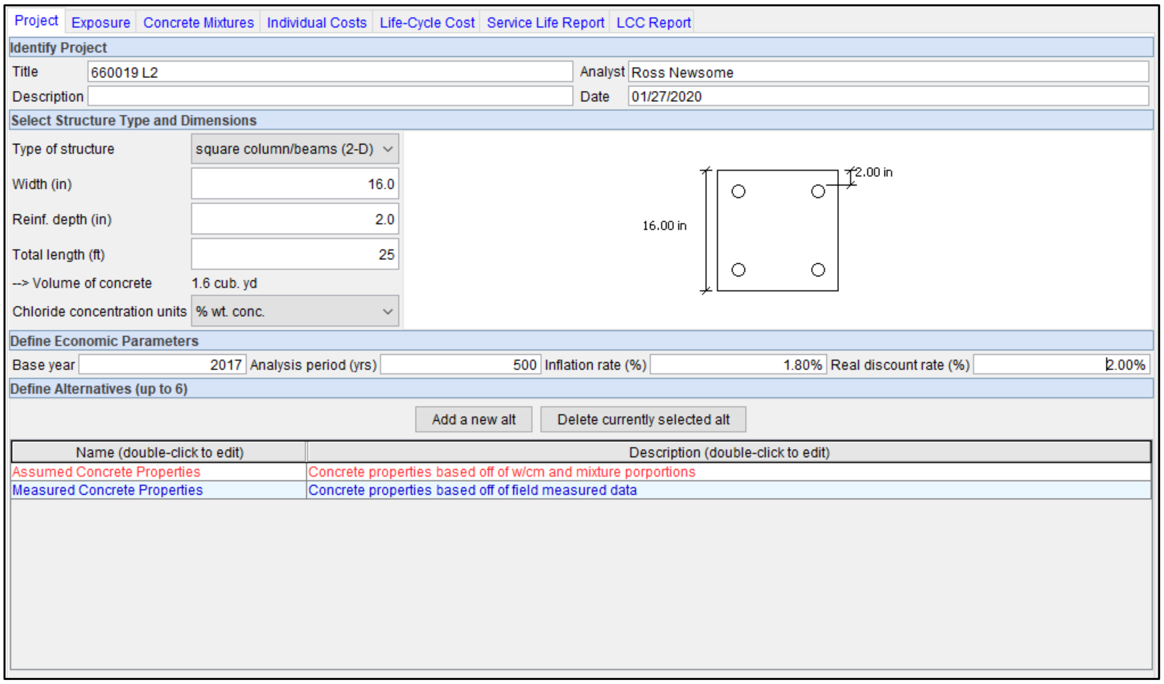
The process of utilizing Life-365 to model corrosion related deterioration is comprised of four steps. The first three steps are comprised of inputs necessary to complete the modeling process while the last one involves producing a reportable LCA. These steps include:

1. Define project
2. Define exposure
3. Define mixture designs
4. Compute service life

The methodology for completing these four steps is explained in greater detail in the following sections of this chapter.

3.4.1 Project Parameters

The first step of LCA modeling with Life-365 involves defining the project in terms of the structure type, units of measure, and analysis period. The interface utilized to do this can be seen in Figure 3.16. Every bridge element examined during field testing received its own project file and parameters specific to itself for LCA modeling.



Identify Project

Title 660019 L2

Analyst Ross Newsome

Description

Date 01/27/2020

Select Structure Type and Dimensions

Type of structure: square column/beams (2-D)

Width (in): 16.0

Reinf. depth (in): 2.0

Total length (ft): 25

--> Volume of concrete: 1.6 cub. yd

Chloride concentration units: % wt. conc.

Define Economic Parameters

Base year 2017

Analysis period (yrs) 500

Inflation rate (%) 1.80%

Real discount rate (%) 2.00%

Define Alternatives (up to 6)

Name (double-click to edit)	Description (double-click to edit)
Assumed Concrete Properties	Concrete properties based off of w/cm and mixture porportions
Measured Concrete Properties	Concrete properties based off of field measured data

Figure 3.16: Life-365 project interface

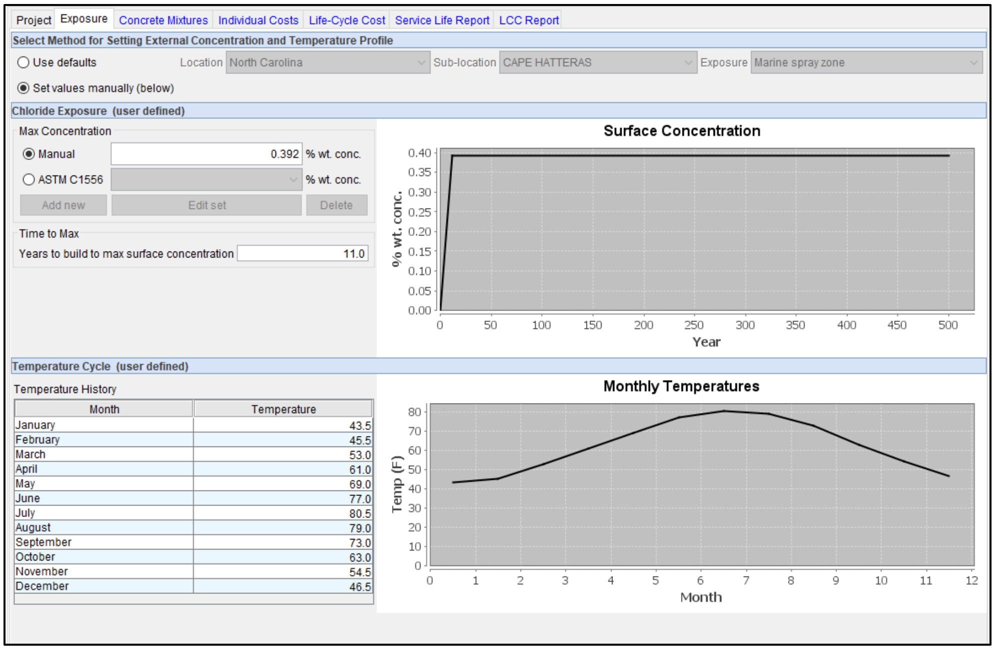
The structure types selected for analysis on this project were “columns” when modeling precast piles and “beams/girders” when modeling bent caps. The thickness of concrete cover that was utilized on these bridge elements was determined from the construction drawings that were provided to the research team by the NCDOT and confirmed in field with the use of a pachometer. In all cases a minimum cover of two inches was utilized.

The analysis period is the maximum amount of time, in years, the user would like to extend the LCA model. Life-365 allows for a maximum analysis period of 500 years. Since the interest of this study is to identify how long the concrete structures modeled can serve before chloride induced corrosion damage propagates there was no need to limit the analysis period. Therefore, a maximum analysis period of 500 years was utilized to essentially remove all time-based constraints from the LCA model.

Economic parameters were not entered for any of the modeling completed in this report. These parameters are only utilized when conducting a life cycle cost analysis (LCCA) with the Life-365 software. The research team felt that an attempt to perform an LCCA would involve an excessive number of assumptions related to construction, material, repair, and maintenance costs. Because of this it was decided to limit the scope of the modeling conducted with Life-365 to only include an LCA.

3.4.2 Exposure Conditions

The Life-365 interface allows for two methods of inputting exposure conditions for modeling allowing the user to choose between utilizing a “default” option or a “custom” option. The interface of the Life-365 exposure settings can be seen in Figure 3.17. Either of these input methods will allow for an LCA to be conducted by identifying two primary factors of exposure. These factors are average monthly temperature and surface chloride concentration. Temperature is factored into the model due to the chloride diffusion coefficient being a function of both time and temperature (Bentz and Thomas 2018). The surface chloride concentration is critical to quantifying the aggressiveness of the environment that the structure is exposed to.



Select Method for Setting External Concentration and Temperature Profile

○ Use defaults Location North Carolina Sub-location CAPE HATTERAS Exposure Marine spray zone

● Set values manually (below)

Chloride Exposure (user defined)

Max Concentration

● Manual 0.392 % wt. conc.

○ ASTM C1556 % wt. conc.

Time to Max

Years to build to max surface concentration 11.0

Temperature Cycle (user defined)

Temperature History

Month	Temperature
January	43.5
February	45.5
March	53.0
April	61.0
May	69.0
June	77.0
July	80.5
August	79.0
September	73.0
October	63.0
November	54.5
December	46.5

Figure 3.17: Life-365 exposure interface

The “default” method of exposure analysis is centered around geographic location and general exposure environments. The user can select the location (state) and sub-location (regional major city) of where the concrete structure under analysis is located. This allows for the average monthly temperature of the location to be determined based on yearly temperature profiles (Bentz and Thomas 2018). The “exposure” can be selected as one of several options to give an estimate of the surface chloride concentration. The possible options for exposure type and associated surface concentration for modeling a structure in a coastal environment can be seen in Table 3.2. Selecting these three parameters is the only inputs Life-365 considers when defining exposure with the “default” method.

Table 3.2: Default settings for exposure type and surface concentration

Exposure Type	Surface Concentration (% wt. conc.)
Marine Spray Zone	1.00
Marine Tidal Zone	0.80
Within 800m of the Ocean	0.60
Within 1.5km of the Ocean	0.60
Parking Garages	0.80
Rural Highway Bridges	0.56
Urban Highway Bridges	0.68

The "custom" method of defining exposure allows the user to input more structure specific information than is provided in the Life-365 model database. For this reason, it was decided that it would be most appropriate to utilize the "custom" method instead of the "default" method of defining exposure. Chloride exposure was defined manually based on the predicted surface concentration that was calculated simultaneously with the diffusion coefficient explained in Chapter 3.4.3 of this thesis. The chloride concentrations found at depths from one to five inches into the concrete were plotted and fitted with a curve that can be expressed with the following function. The value of the function where the depth (x) is equal to zero was determined to be the surface concentration.

$$C_{(x,t)} = C_0\left(1 - erf\left(\frac{x}{\sqrt{4 \cdot D \cdot t}}\right)\right)$$

where C_0 = initial chloride concentration measured,

x = the depth below the exposed surface,

D = the apparent chloride transport coefficient,

t = time.

In order to ensure that the surface concentration calculated by utilizing this method were relatively accurate the calculated values were compared with chloride concentration values that were measured from powder samples taken from a depth of zero to a half an inch into the concrete structure. These values as well as the percentage difference can be seen in Table 3.3. There is no clear pattern as to if one method predicted a high or low surface concentration. When compared to the values for surface concentration provided in Life-365's "default" settings for North Carolina marine tidal zones (0.8 % wt. conc.) and locations within 1.5km of the ocean (0.6 % wt. cont.) provided in Table 3.2, it appears that the modeling software overestimates the value for surface concentration in many cases.

Table 3.3: Surface chloride concentrations estimated through various methods

Structure Number	**Location**	**Surface Concentration (% wt. conc.)**			
		Published	**Measured**		
		Dependent Upon Exposure	**Powder Samples 0 - 0.5in**	**Calculated from Best Fit Curve**	**Percentage Difference**
150020	L1	0.60	0.282	0.129	-54%
	L2		0.310	0.225	-28%
660019	L1	0.80	0.291	0.246	-15%
	L2		0.341	0.392	15%
260007	L1	0.60	0.162	0.111	-31%
	L2		0.095	0.252	166%
640010	L1	0.60	-	0.038	-
	L2	0.80	0.295	0.371	26%
	L3	0.60	0.115	0.111	-4%
	L4	0.80	0.588	0.582	-1%
660021	L1	0.60	0.115	0.061	-47%
	L2		0.183	0.162	-12%
090061	L1	0.80	0.417	0.535	29%
	L2		0.759	0.830	9%
150026	L1	0.60	0.037	0.012	-68%
	L2	0.80	0.473	0.879	86%
	L3		0.357	0.281	-21%

Note: "-" Indicates that data was not collected.

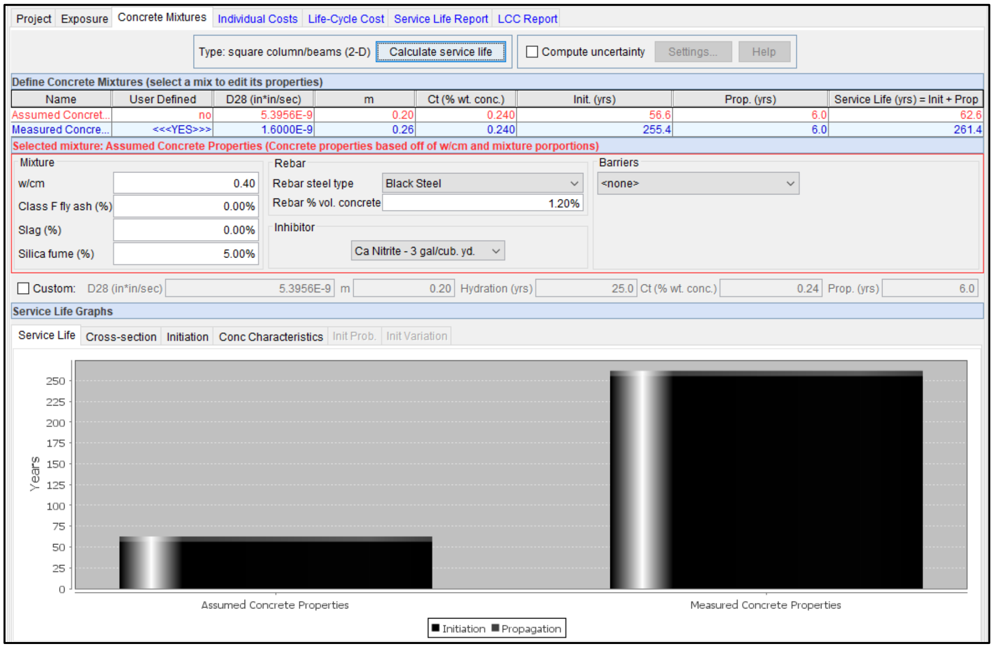
The annual exposure temperature cycle was defined by the most current average monthly temperature data available for three cities spanning the North Carolina coastline. Each bridge modeled during this project was assigned temperature data from the city in closest proximity to it. The three cities that were chosen were Jacksonville, Wilmington, and Nags Head. The monthly average temperatures of these cities can be seen in Table 3.4 (U.S. Climate Data).

Table 3.4: Monthly average temperatures utilized for LCA modeling

Month	**Average Temperature (^{0}F)**		
	Jacksonville	**Wilmington**	**Nags Head**
January	43.5	46.0	45.0
February	45.5	49.0	46.0
March	53.0	55.0	51.5
April	61.0	63.0	61.0
May	69.0	70.5	69.0
June	77.0	78.0	77.5
July	80.5	81.5	80.5
August	79.0	79.5	80.5
September	73.0	75.0	76.0
October	63.0	65.5	67.0
November	54.5	56.5	56.0
December	46.5	48.5	50.0

3.4.3 Concrete Properties

The Life-365 interface, as shown in Figure 3.18 allows for two methods of inputting concrete properties for modeling. These include a “default” method and a “custom” method. The default method allows for basic mixture design characteristics including w/cm and whether SCM’s such as slag, class F fly ash, or silica fume are to be utilized. It was determined that the “default” method would provide a very simplified model that would include many large assumptions about the concrete performance. To avoid making these assumptions and to utilize as much performance data that was gathered during the field visits the “custom” method of concrete property entry was selected for use. Using this method, it is necessary that values for diffusion coefficient at 28 days, diffusion decay index (m), hydration year, CTV, and propagation period be determined. The process for determining each of these values is explained in further depth in the following paragraphs.



Type: square column/beams (2-D)

☐ Compute uncertainty

Define Concrete Mixtures (select a mix to edit its properties)

Name	User Defined	D28 (in*in/sec)	m	Ct (% wt. conc.)	Init. (yrs)	Prop. (yrs)	Service Life (yrs) = Init + Prop
Assumed Concret...	no	5.3956E-9	0.20	0.240	56.6	6.0	62.6
Measured Concre...	<<<YES>>>	1.6000E-9	0.26	0.240	255.4	6.0	261.4

Selected mixture: Assumed Concrete Properties (Concrete properties based off of w/cm and mixture porportions)

Mixture
w/cm: 0.40
Class F fly ash (%): 0.00%
Slag (%): 0.00%
Silica fume (%): 5.00%

Rebar
Rebar steel type: Black Steel
Rebar % vol. concrete: 1.20%

Inhibitor
Ca Nitrite - 3 gal/cub. yd.

Barriers
<none>

☐ Custom: D28 (in*in/sec) 5.3956E-9 m 0.20 Hydration (yrs) 25.0 Ct (% wt. conc.) 0.24 Prop. (yrs) 6.0

Figure 3.18: Life-365 concrete properties interface

The diffusion coefficient is critical to the modeling effort because of its ability to quantify the rate at which chlorides can ingress into concrete. A diffusion coefficient for each bridge element tested for this project was calculated with use of Fick’s Second Law and the chloride concentrations measured at various depths into the concrete determined through field and laboratory testing. The methods of acquiring powder samples and determining the chloride concentrations at these various depths can be seen in Chapters

3.2.3 and 3.3.1 of this report. The program Mathcad was utilized to create a worksheet that given various inputs for chloride concentration at depths ranging from one to five inches would calculate the diffusion coefficient. Fick's equation that was utilized to calculate the diffusion coefficient using this method can be seen as follows (Bentz and Thomas 2018):

$$\frac{dC}{dt} = D \cdot \frac{d^2C}{dx^2}$$

where D = the apparent diffusion coefficient,
C = the chloride content,
x = the depth from the concrete surface,
t = time.

This equation provided the diffusion coefficient of the concrete at the age in which it was tested. In the case of this report this age is between ten to fifteen years old dependent upon the bridge analyzed. However, the diffusion coefficient at 28 days is necessary for service life modeling with Life-365. To correct for the impact of time and to calculate what the diffusion coefficient at 28 days would be the following equation was utilized (Bentz and Thomas 2018).

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

where $D(t)$ = diffusion coefficient at time t,
D_{ref} = diffusion coefficient at time t_{ref} (28 days),
m = diffusion decay index.

Using this equation, the reference diffusion coefficient (D_{ref}) set at 28 days was determined. The time (t) that was utilized was the age of each bridge in days along with the reference time (t_{ref}) of 28 days. Due to lack of knowledge of when chloride exposure began for each bridge element during the construction process, the age of the bridge and the subsequent beginning of exposure was assumed to be the completion date of the bridge. The diffusion decay index (m) that was utilized is a constant value that is dependent upon if portland cement concrete, fly ash concrete, or slag cement concrete are utilized. Values for m that have been proposed by Bamforth (1999) can be seen in Table 3.5 and widely reviewed (Bentz and Thomas 2018). However, these values do not indicate the amount of fly ash or slag that is used in fly ash or slag concrete mixtures. This shortfall has prompted the creators of Life-365 to recommend a conservative approach to calculating m for mixtures containing either fly ash or slag which considers their replacement rates in the concrete. This formula was utilized in determining the diffusion decay indexes for mixtures containing fly ash in this analysis and can be seen as follows (Bentz and Thomas 2018). Mixtures not containing fly ash or slag utilized the m value proposed by Bamforth (1999) for portland cement concrete (0.264).

$$m = 0.2 + 0.4(\%FA/50 + \%SG/50)$$

Table 3.5: Diffusion decay index (m) values for various types of concrete

Concrete Mixture	***m***
Portland Cement Concrete	0.264
Fly Ash Concrete	0.700
Slag Cement Concrete	0.620

The chloride threshold value (C_t) required for this analysis quantifies the amount of chlorides that must be present at the depth of the steel to initiate corrosion. This threshold value is widely discussed in published literature and a wide variety of threshold values have been proposed. However, due to the multitude of external influences on this value, to propose a single value for which this threshold should be would be invalid. For this reason, Life-365 recommends a conservative chloride threshold value of 0.05 percent by weight of concrete (% wt. conc.) be utilized for typical portland cement concrete (Bentz and Thomas 2018). The only factor considered by Life-365 to impact the value for Ct is the presence of calcium nitrite corrosion inhibitors. These corrosion inhibitors are dosed in units of gallons per cubic yard of concrete (gal/CY) and the impact on the chloride threshold value per various dosages can be seen in Table 3.6 (Bentz and Thomas 2018). Per the NCDOT specification, all precast and cast in place concrete bridge members that were evaluated for this study contained three gal/CY of calcium nitrite corrosion inhibitor and thus a threshold value of 0.24 was utilized for LCA modeling.

Table 3.6: Corrosion inhibitor dosages impact on chloride threshold value

Dosage (gal/CY)	**Threshold, C_t (% wt. conc.)**
2	0.15
3	0.24
4	0.32
5	0.37
6	0.40

The duration of the hydration process has an impact on the diffusion coefficient decay process. The diffusion coefficient decay process is the phenomenon in concrete that throughout the hydration process the diffusion coefficient will decrease with time. This is due to the microstructure of the concrete continually developing during hydration and thus becoming less permeable. With the equation developed to model the diffusion coefficient decay process it allows the diffusion coefficient to reach a value of zero at a given point. Since assuming the diffusion coefficient could ever reach zero would not be valid, Life-365 assumes that the decay process ends when hydration is complete and the diffusion coefficient at this point is kept constant throughout the remainder of the analysis period. The length of the hydration process is recommended to be set to 25 years and therefore this hydration duration was utilized in all LCA modeling (Bentz and Thomas 2018).

The values for diffusion coefficient at 28 days, hydration years, diffusion decay index, and chloride threshold value are utilized to calculate the initiation period (t_i) of the concrete structure based upon given exposure conditions. This is only part of the service life as the propagation period (t_p) must also be added (Service Life = $t_i + t_p$). The propagation period is defined as the time for corrosion to reach an unacceptable level and warrant repair or replacement (Bentz and Thomas 2018). This time is dependent upon the type of reinforcing steel utilized in the structure. Life-365 recommends a propagation period of six years be utilized for concrete containing uncoated black steel and twenty years for concrete containing epoxy coated steel (Bentz and Thomas 2018). This propagation period is simply added to the initiation period at the end of the LCA to provide an estimate for the total service life.

3.4.4 LCA Modeling

A Service Life Report can be produced when all parameters have been defined for project parameters, exposure conditions, and concrete properties. This report will produce four figures that display the impact of chloride ingress and corrosion on the reinforced concrete component. The service life graph displays the estimated service life of each concrete mixture based of a calculated initiation period and propagation period. The service life is achieved by adding the initiation period to the propagation period. An example of a service life graph comparing two different concrete mixtures can be seen in Figure 3.19 (Bentz and Thomas 2018).

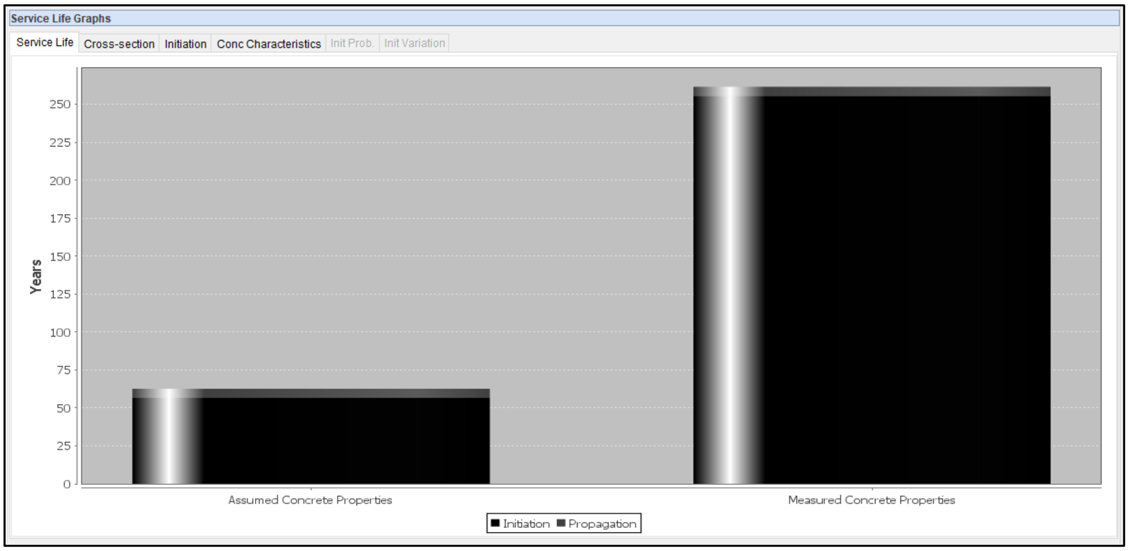
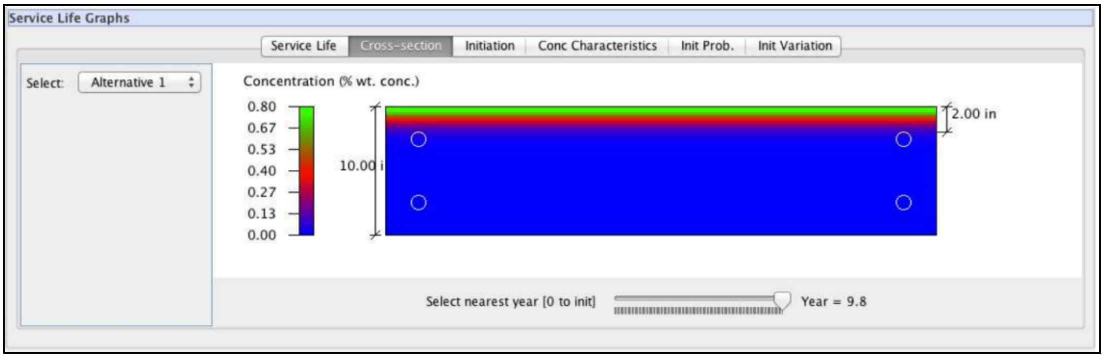


Figure 3.19: Life-365 service life graph

A figure depicting the cross section of the reinforced concrete element and the concentration of chlorides at the point of initiation of corrosion. This is presented in Figure 3.20 where the chloride concentration scale is on the right-hand side of the graph and the modeled reinforced concrete element is color coordinated to depict the chloride concentrations throughout it at the time of initiation (Bentz and Thomas 2018).



Select: Alternative 1

Select nearest year [0 to init] Year = 9.8

Figure 3.20: Life-365 chloride concentration cross section

The impact of chloride concentration on time of initiation is displayed in two ways that are displayed in Figure 3.21. The graph on the left shows the concentration of chlorides at the time of initiation by depth of the structure while the graph on the right shows the concentration of chlorides at the rebar depth up to the point of initiation (Bentz and Thomas 2018). The reinforcement depth is displayed as a vertical dashed line in the graph on the right.

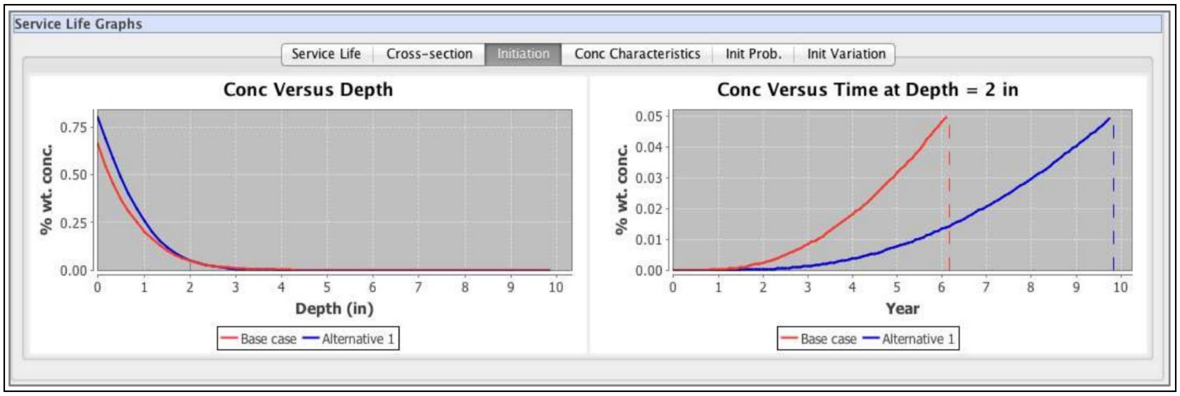


Figure 3.21: Life-365 concentration of chlorides at time of initiation

Two additional graphs are produced that indicate the performance level of the concrete mixture. These can be seen in Figure 3.22 where the graph on the left depicts the diffusion of chloride ions into the concrete over time and the graph on the right depicts how the concrete surface chloride levels change over time (Bentz and Thomas 2018).

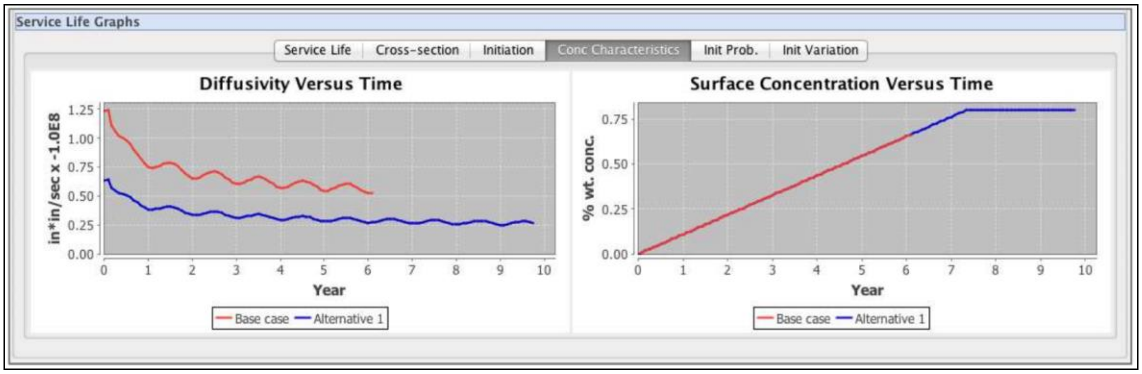


Figure 3.22: Life-365 concrete diffusivity and surface concentration over time

CHAPTER 4: TEST RESULTS

4.1 Rapid Chloride Test

The use of Rapid Chloride Tests (RCT) was employed to determine the concentration of chlorides at various depths into the concrete elements selected for testing. The tests were conducted on powder samples acquired using the procedure outlined in Chapter 3.2.3 of this thesis. RCT tests were conducted on two 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. The following is an analysis of the chloride concentrations measured at various depths and the corresponding chloride profiles for each test location.

Bridge 150020 was located in the corrosive zone crossing over the East Prong of Broad Creek. Access to the end bents was obstructed by rip rap and the embankment elevation so the prestressed pile substructure of Bent 1 were evaluated. Powder samples were acquired from two piles at similar elevations. The piles were not located in the water but close enough to it that they most likely experience infrequent wetting only during large storm events. The two locations tested (L1 and L2) are labeled in Figure 4.1. The RCT results indicating the chloride concentrations detected are shown in Table 4.1 and the resulting chloride profile is shown in Figure 4.2.

Figure 4.1: RCT test locations L1 and L2 on bridge 150020

Table 4.1: RCT Results for test locations L1 and L2 on bridge 150020

Structure Number	**Location**	**Depth (in)**	**Chloride Concentration**	
			% Conc. Wt.	**pcy**
150020	L1	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	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

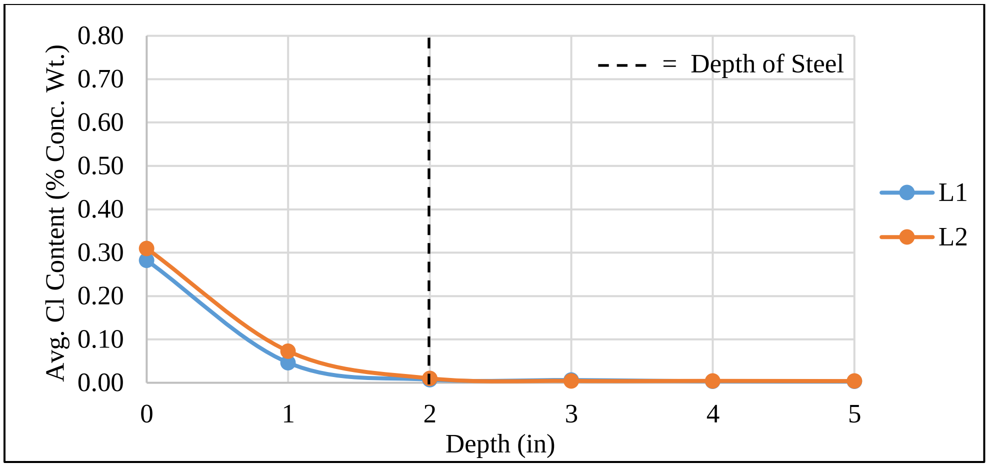


Figure 4.2: Chloride profile for locations L1 and L2 on bridge 150020

Bridge 660019 was located in the corrosive zone crossing over Stones Creek. Access to the end bents was obstructed by rip rap so selected components of the prestressed pile substructure of Bent 1 were evaluated. Powder samples were acquired from two piles at similar elevations within the tidal zone where daily tidal changes submerge and expose the concrete. A maximum powder sample depth of 3.0-inches was acquired due to the presence of reinforcing steel at the drilled locations. The two locations tested (L1 and L2) are labeled in Figure 4.3. The RCT results indicating the chloride concentrations detected are shown in Table 4.2 and the resulting chloride profile is shown in Figure 4.4.

Figure 4.3: RCT test locations L1 and L2 on bridge 660019

Table 4.2: RCT Results for test locations L1 and L2 on bridge 660019

Structure Number	**Location**	**Depth (in)**	**Chloride Concentration**	
			% Conc. Wt.	**pcy**
660019	L1	0	0.291	11.096
		1	0.051	1.963
		2	0.003	0.109
		3	0.003	0.097
	L2	0	0.341	13.001
		1	0.081	3.096
		2	0.004	0.167
		3	0.004	0.171

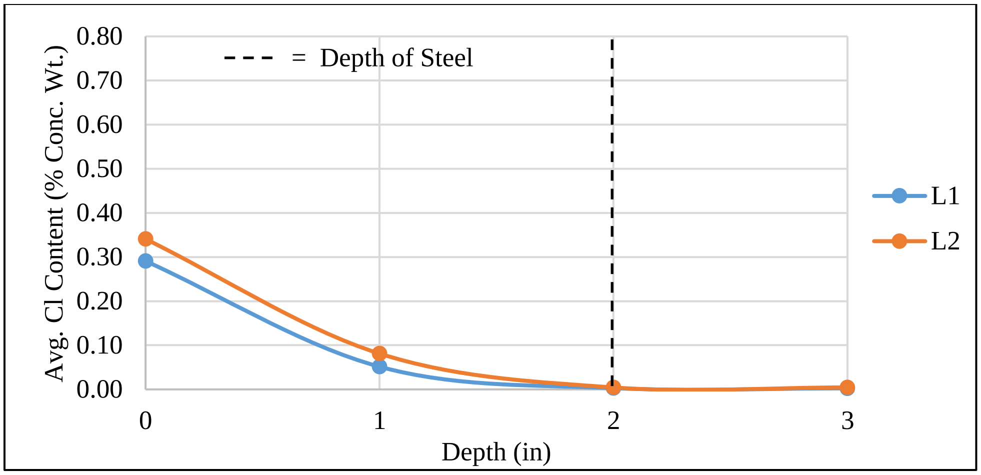


Figure 4.4: Chloride profile for locations L1 and L2 on bridge 660019

Bridge 260007 was located in the highly corrosive zone crossing over Corey's Ditch. Two powder samples were acquired from the Bent 3 Cap at the same elevation. The Bent Cap appears to be high enough above the tide where the water level would reach it only on a rare occasion. The two locations tested (L1 and L2) are labeled in Figure 4.5. The RCT results indicating the chloride concentrations detected are shown in Table 4.3 and the resulting chloride profile is shown in Figure 4.6.

Figure 4.5: RCT test locations L1 and L2 on bridge 260007

Table 4.3: RCT Results for test locations L1 and L2 on bridge 260007

Structure Number	**Location**	**Depth (in)**	**Chloride Concentration**	
			% Conc. Wt.	**pcy**
260007	L1	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
		5	0.002	0.069
	L2	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

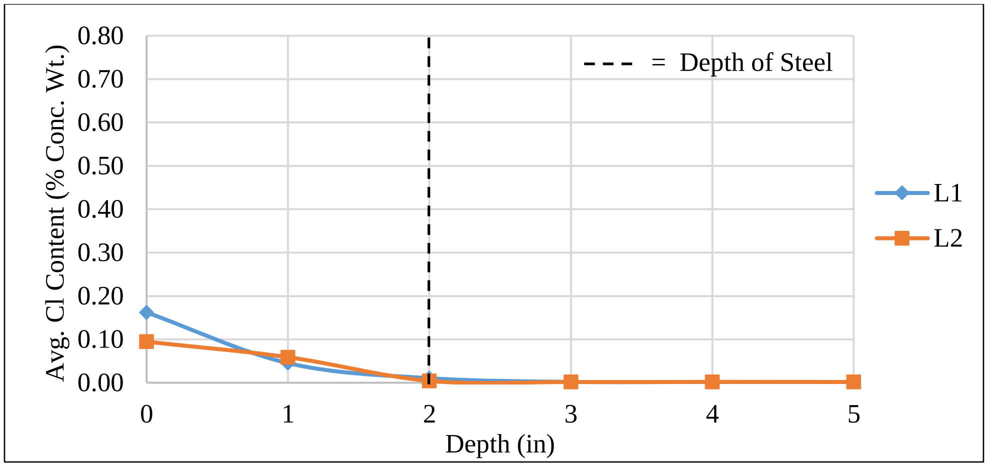


Figure 4.6: Chloride profile for locations L1 and L2 on bridge 260007

Bridge 640010 was located in the highly corrosive zone crossing over Bradley Creek. Four locations were evaluated including the Bent Cap and a prestressed pile from both Bent 1 and Bent 3. Powder samples acquired from the two piles showed evidence of being in the tidal zone where tidal changes frequently submerge and expose the concrete. The four locations tested (L1, L2, L3 and L4) are labeled in Figure 4.7 and 4.8. The RCT results indicating the chloride concentrations detected are shown in Table 4.4 and the resulting chloride profile is shown in Figure 4.9.

Figure 4.7: RCT test locations L1 and L2 on bridge 640010

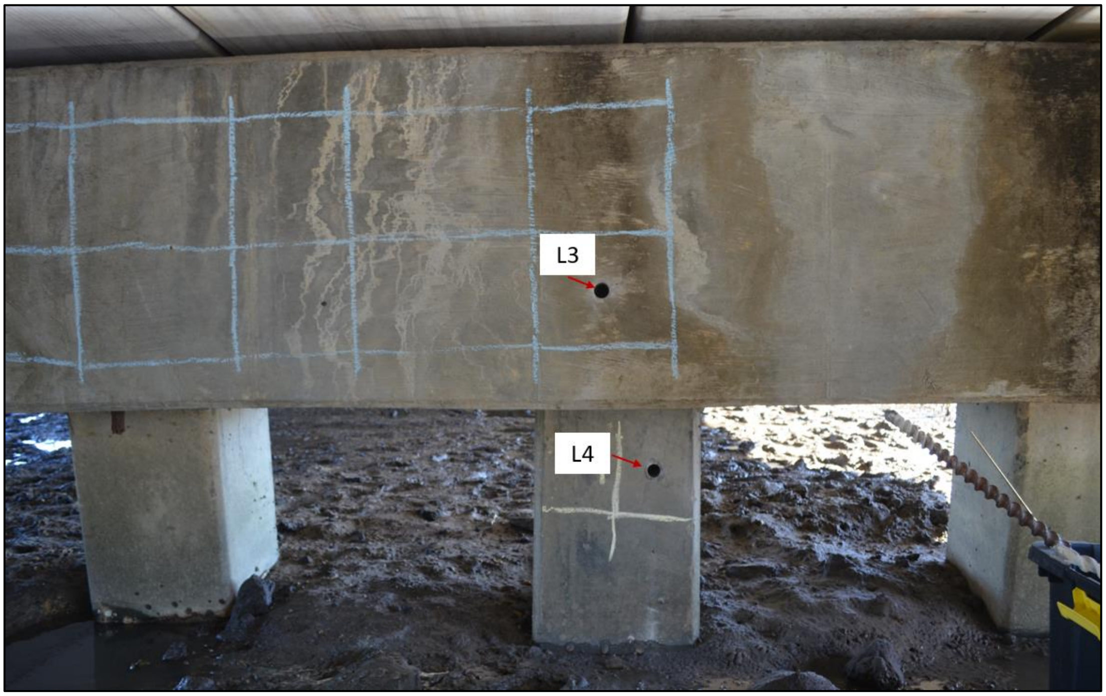


Figure 4.8: RCT test locations L3 and L4 on bridge 640010

Table 4.4: RCT Results for test locations L1, L2, L3 and L4 on bridge 640010

Structure Number	**Location**	**Depth (in)**	**Chloride Concentration**	
			% Conc. Wt.	**pcy**
640010	L1	1	0.018	0.692
		2	0.004	0.139
		3	0.004	0.150
		4	0.004	0.145
		5	0.003	0.132
	L2	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	0	0.115	4.395
		1	0.041	1.574
		2	0.007	0.281
		3	0.004	0.142
		4	0.003	0.125
		5	0.004	0.157
	L4	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

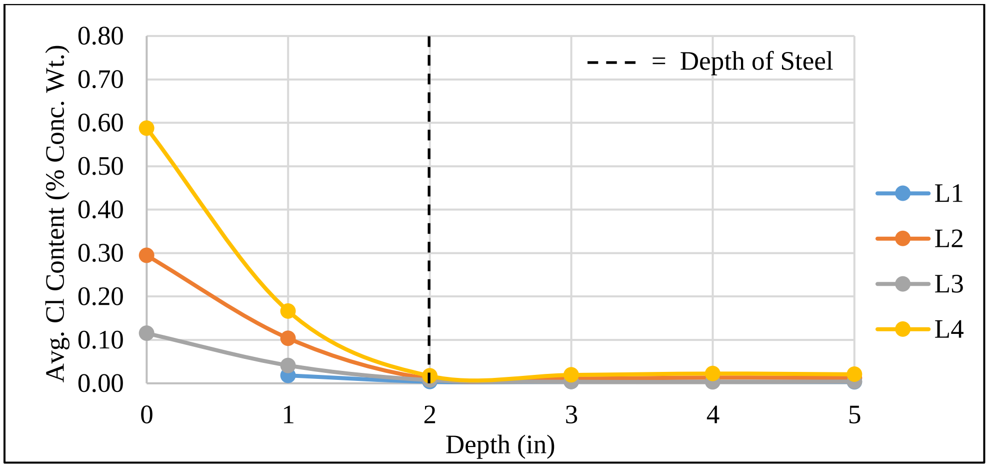


Figure 4.9: Chloride profile for locations L1, L2, L3 and L4 on bridge 640010

Bridge 660021 was located in the highly corrosive zone crossing over Bear Creek. The prestressed pile substructure of Bent 1 was evaluated. Powder samples were acquired from two piles at the same elevation approximately one foot above the tidal zone where daily tidal changes submerge and expose the concrete. The two locations tested (L1 and L2) are labeled in Figure 4.10. The RCT results indicating the chloride concentrations detected are shown in Table 4.5 and the resulting chloride profile is shown in Figure 4.11.

Figure 4.10: RCT test locations L1 and L2 on bridge 660021

Table 4.5: RCT Results for test locations L1 and L2 on bridge 660021

Structure Number	**Location**	**Depth (in)**	**Chloride Concentration**	
			% Conc. Wt.	**pcy**
660021	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
	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

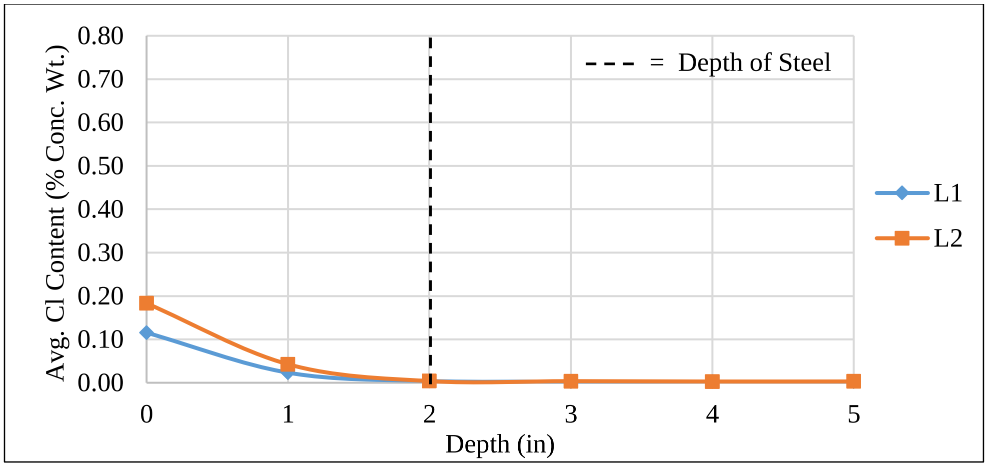


Figure 4.11: Chloride profile for locations L1 and L2 on bridge 660021

Bridge 660091 was located in the highly corrosive zone crossing over Parrot Swamp. As it was a single span bridge, two locations on End Bent 1 at similar elevations were selected for sampling. These locations would have experienced wetting from the waterway on a very rare occurrence if at all. The two locations tested (L1 and L2) are labeled in Figure 4.12. The RCT results indicating the chloride concentrations detected are shown in Table 4.6 and the resulting chloride profile is shown in Figure 4.13.

Figure 4.12: RCT test locations L1 and L2 on bridge 660091

Table 4.6: RCT Results for test locations L1 and L2 on bridge 660091

Structure Number	**Location**	**Depth (in)**	**Chloride Concentration**	
			% Conc. Wt.	**pcy**
660091	L1	1	0.008	0.316
		2	0.007	0.248
		3	0.007	0.252
	L2	1	0.011	0.436
		2	0.009	0.326
		3	0.011	0.408

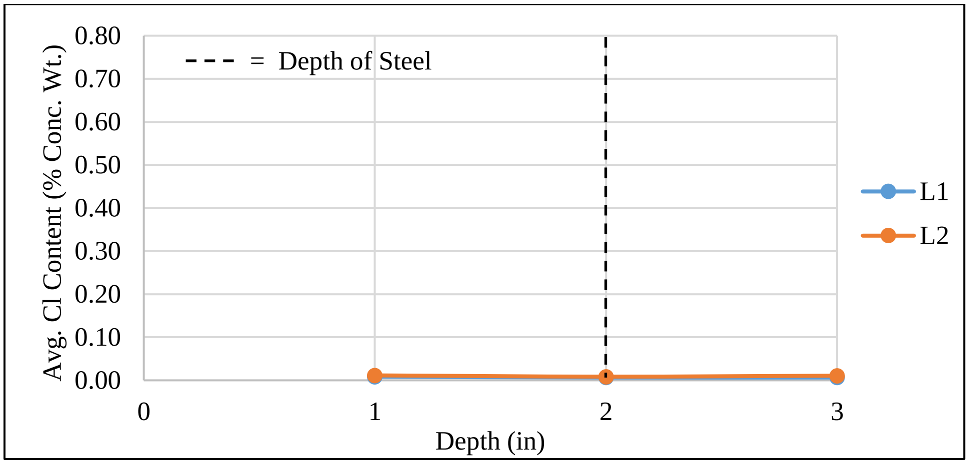


Figure 4.13: Chloride profile for locations L1 and L2 on bridge 660091

Bridge 090061 was located in the highly corrosive zone crossing over Town Creek. The prestressed pile substructure of Bent 4 was evaluated. Powder samples were acquired from two piles at different elevations within the tidal zone where daily tidal changes submerge and expose the concrete. The two locations tested (L1 and L2) are labeled in Figure 4.14. L1 is located approximately one foot below L2 due to the rising tide restricting access to complete the second drilling at the same elevation. The RCT results indicating the chloride concentrations detected are shown in Table 4.7 and the resulting chloride profile is shown in Figure 4.15.

Figure 4.14: RCT test locations L1 and L2 on bridge 090061

Table 4.7: RCT Results for test locations L1 and L2 on bridge 090061

Structure Number	Location	Depth (in)	Chloride Concentration	
			% Conc. Wt.	pcy
090061	L1	0	0.417	15.894
		1	0.405	15.463
		2	0.256	9.762
		3	0.164	6.268
		4	0.102	3.907
		5	0.065	2.498
	L2	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

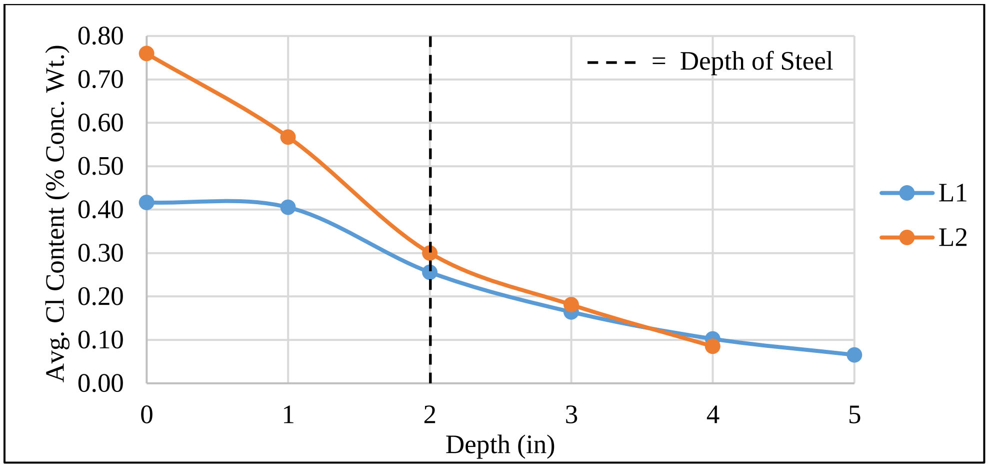


Figure 4.15: Chloride profile for locations L1 and L2 on bridge 090061

Bridge 150026 was located in the highly corrosive zone crossing over Deep Creek. One location on the End Bent 2 and two locations on Bent 1 were chosen for analysis. The two locations selected on the Bent 1 cap where of the same elevation within the tidal zone where daily tidal changes submerge and expose the concrete. The location tested on the End Bent (L1) is labeled in Figure 4.16. The locations tested on Bent 1 (L2 and L3) are labeled in Figure 4.17. The prestressed piles of Bent 1 were not candidates for testing due to the high tide having completely submerged them. The RCT results indicating the chloride concentrations detected are shown in Table 4.8 and the resulting chloride profile is shown in Figure 4.18.

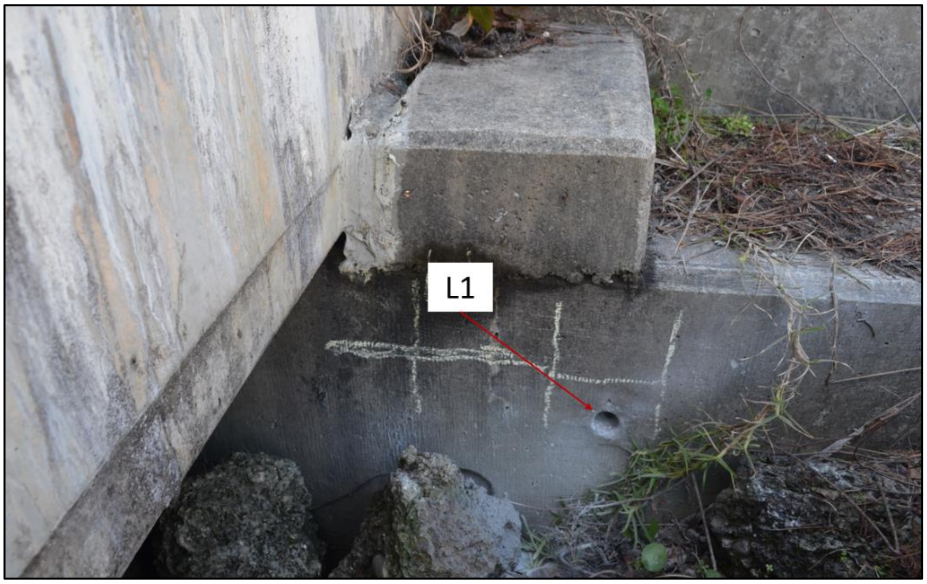


Figure 4.16: RCT test location L1 on bridge 150026

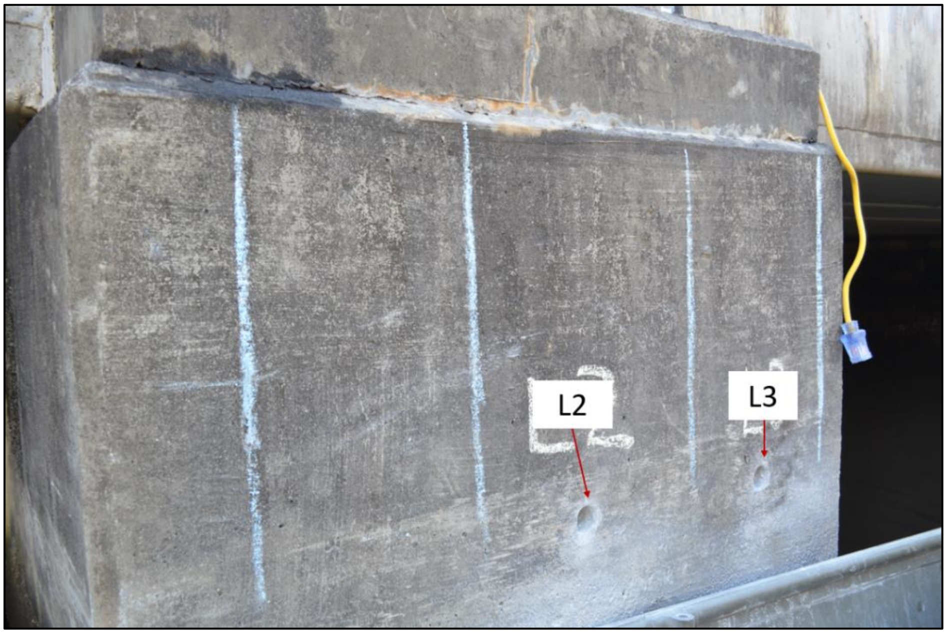


Figure 4.17: RCT test locations L2 and L3 on bridge 150026

Table 4.8: RCT Results for test locations L1, L2 and L3 on bridge 150026

Structure Number	**Location**	**Depth (in)**	**Chloride Concentration**	
			% Conc. Wt.	**pcy**
150026	L1	0	0.037	1.406
		1	0.011	0.415
		2	0.006	0.222
		3	0.004	0.155
		4	0.004	0.161
		5	0.004	0.153
	L2	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
		5	0.004	0.142
	L3	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

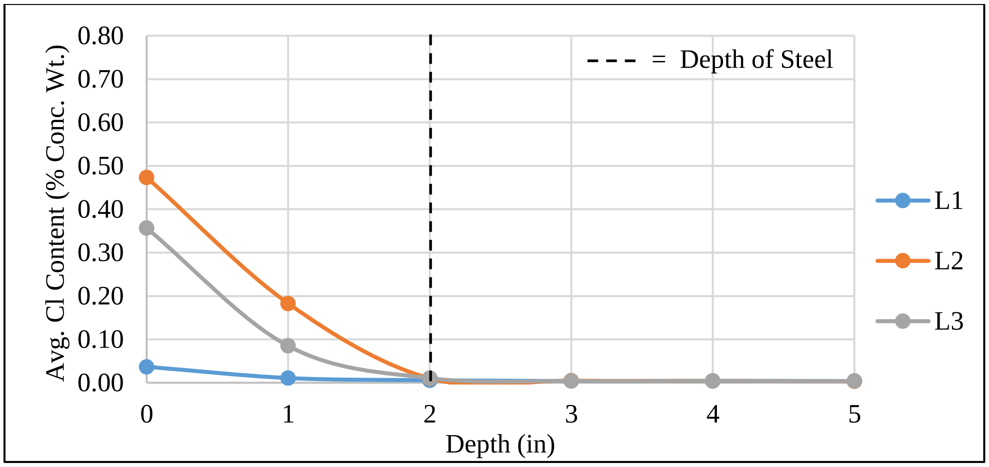


Figure 4.18: Chloride profile for locations L1, L2 and L3 on bridge 150026

A common trend seen between the results of all RCT tests conducted is the quick decline of chloride concentration between zero and two-inches into the element followed by the leveling out of the concentration until the maximum depth of five-inches was reached. This indicates that there has not been much chloride penetration after the two-inch depth and that there are low levels of background chlorides in all bridges. These trends were both anticipated based on previous research on NC bridges (Cavalline et al. 2013, Tempest et al. 2017). The use of pozzolans in almost all concrete mixtures should result in a low diffusion coefficient indicating that chlorides will not easily penetrate deep below the exposed surface. The low-level background chloride concentrations are typical as chlorides can be introduced to the mixture unknowingly through use of certain aggregates or through the batching process. Since the background concentrations are significantly lower than typical chloride threshold values there is little to no risk of them initiating corrosion.

The only bridge evaluated that did not conform to the trends discussed in the preceding paragraph was 660091. The chloride concentrations found are relatively low and remain constant or increased across each depth sampled. This bridge was the first bridge evaluated for the project and the process of acquiring powder samples was conducted with a less refined method at this point in the project. The method utilized one drill with a single sized drill bit to acquire samples from all depths. During later stages of the project, a more refined method involving two drills and two sized drill bits was applied for all remaining bridges visited. This method provided a higher level of consistency while sampling and produced more reliable RCT results. These methods are compared and explained in further depth in Chapter 3.2.3 of this thesis. It is unclear as to if the results of the RCT tests from bridge 660091 are valid due to the high probability of contamination from utilizing the one drill method. Due to this, the ability to compute meaningful diffusion coefficients and surface concentrations to utilize while conducting service life modeling for this bridge was impacted.

4.2 Diffusion Coefficient and Surface Concentration Calculations

The results of chloride concentrations, measured at various depths into the concrete, through RCT tests were utilized to calculate the diffusion coefficient and estimated surface concentration of each bridge location evaluated. The procedure for calculating these two properties is explained in further depth in Chapters 3.4.2 and 3.4.3 of this thesis, respectively. To expedite these calculations MathCad documents (sheets) were created to input pertinent information and automatically calculate the diffusion coefficient. These MathCad sheets can be seen in Appendix B of this thesis. A summary of the surface concentration and diffusion coefficient information calculated for this thesis is included in Table 4.9.

Table 4.9: Summary of measured diffusion coefficients and surface concentrations

Structure Number	**Location**	**Corrosive Zone**	**Location Description**	**Surface Concentration (% wt. conc.)**	**Diffusion Coefficient (in^2/yr)**
150020	L1	Corrosive	Prestressed Pile Infrequent Wetting	0.129	0.054
	L2		Prestressed Pile Infrequent Wetting	0.225	0.047
660019	L1	Corrosive	Prestressed Pile Tidal Zone	0.246	0.032
	L2		Prestressed Pile Tidal Zone	0.392	0.031
260007	L1	Highly Corrosive	Bent Cap Rare Occurrence Wetting	0.111	0.048
	L2		Bent Cap Rare Occurrence Wetting	0.252	0.024
640010	L1	Highly Corrosive	Bent Cap Very Infrequent Wetting	0.038	0.073
	L2		Prestressed Pile Tidal Zone	0.371	0.033
	L3		Bent Cap Very Infrequent Wetting	0.111	0.048
	L4		Prestressed Pile Tidal Zone	0.582	0.034
660021	L1	Highly Corrosive	Prestressed Pile Infrequent Wetting	0.061	0.046
	L2		Prestressed Pile Infrequent Wetting	0.162	0.029
660091	L1	Highly Corrosive	End Bent Rare Occurrence Wetting	0.009	2.473
	L2		End Bent Rare Occurrence Wetting	0.011	21.597
090061	L1	Highly Corrosive	Prestressed Pile Tidal Zone	0.535	0.322
	L2		Prestressed Pile Tidal Zone	0.830	0.197
150026	L1	Highly Corrosive	End Bent Very Infrequent Wetting	0.012	0.628
	L2		Bent Cap Tidal Zone	0.879	0.024
	L3		Bent Cap Tidal Zone	0.281	0.036

Common values for diffusion coefficients for generic portland cement concrete range from 0.3 to 0.6 in^2/yr. In almost all cases the measured diffusion coefficient was much lower than this range. These results are in line with those that could be expected from high quality concrete containing one or more pozzolans within it, and from precast concrete members that benefit from a highly controlled production environment. Previously conducted studies of the Virginia Dare Bridge, located on the Outer Banks of North Carolina, explored the impact of silica fume and fly ash use on chloride diffusion coefficient rates. Field gathered data from the study indicated that bridge pier concrete containing a tertiary blend of 20% Class F fly ash and 5% silica fume resulted in an apparent diffusion coefficient of 0.043 in^2/year (Tempest et al. 2017). This value of diffusion coefficient is within the same order of magnitude as the results calculated for similar concrete structures and mixture designs evaluated during this study. The low diffusion properties of the concrete utilized on the bridges observed leads to the assumption that they will not experience excessive chloride loading and will meet or exceed their designed service life of 50-years.

There is not a published expected value for the surface concentration of concrete bridge elements exposed to chloride rich waters. There are, however, assumptive values provided by the Life-365 modeling software ranging from 0.6 % wt. conc. (22.9 lb/CY) to 0.8 % wt. conc. (26.5 lb/CY) for structures within 1.5km of the ocean and marine tidal zones respectively. These surface concentration values for structures within 1.5 km of the ocean seem to be overestimates based upon the findings of this research study. The surface concentration values for tidal zones measured during this study are typically higher than the surface concentrations found on structures within 1.5km of the ocean. However, in many cases, the estimated values from Life-365 are also an overestimate compared to what was measured. Surface concentrations do appear to be lower in the corrosive zone than in the highly corrosive zone. However, due to the large variation in results gathered from the highly corrosive zone further research is recommended to develop a deeper understanding of how frequency of exposure to brackish waters impacts surface chloride concentrations.

Some test locations yielded diffusion coefficients that are higher than would be expected. This is the case with both locations sampled at bridge 660091 and with L1 at bridge 150026. At each of these locations, very low and consistent concentrations of chloride were detected across all depths sampled. It is likely that, due to these locations limited exposure to chloride-rich waters, minimal chloride loading has been experienced. The chloride concentrations detected are most likely background chlorides present in the mixture at the time of batching and construction. Diffusion coefficients were calculated based upon the curvature of the chloride profile developed as a result of RCT testing. Because these locations showed almost constant chloride concentrations across all depths, the chloride profiles did not contain sufficient curvature to produce meaningful results from the diffusion coefficient calculations, and ultimately resulted in artificially inflated values. Due to this, the diffusion coefficients calculated for bridge 660091 were much higher than any published values discovered while performing the literature review for this thesis and were therefore considered invalid. Because the use of an accurate diffusion coefficient is so critical to producing an accurate service life prediction model it was decided that this bridge would not be utilized in the modeling analysis of this thesis.

4.3 Service Life Modeling

The service life modeling was completed using the Life-365 software following the methodology outlined in Chapter 3.4 of this thesis. The service life was modeled using two separate methods for each bridge. Both methods were identical in regards to defining the geometry of the concrete element and its exposure conditions. The two methods deviated in how the concrete properties, related to corrosive site performance, were evaluated. The first method utilized Life-365 software to estimate the concretes characteristics based upon factors such as *w/cm*, percentage of fly ash, silica fume, or slag utilized, and the dosage of corrosion inhibitor. The second method utilized information gathered and calculated 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.

Defining the type of structural component and its geometry was the first step in the modeling process. In Table 4.10 the type of element, its dimensions, and the cover utilized on each component modeled are shown. The exposure conditions were input based upon as many measured values as possible. These conditions, which can be seen in Table 4.11, include the estimated surface concentration and the buildup period which is the age of the bridge. The average monthly temperature values were defined by regions surrounding one of three major North Carolina cities: Jacksonville, Wilmington, and Nags Head. The average temperature values utilized for these three regions can be seen in Table 3.5. These characteristics for both the type of component and the exposure conditions were held constant for each location throughout both methods of modeling. The vertical distance from high tide elevation for each location that is reported is an approximate value identified using the approximate elevation where high tide was observed on the day of testing. Many factors impact what the high tide elevation is on a day to day basis, so the reported value is only approximated to the nearest half a foot. A negative value indicates that the location was below where the high tide was observed.

Table 4.10: Geometry and element type inputs of modeled locations

Structure Number	**Location**	**Vertical Distance from High Tide Elevation (ft)**	**Bridge Element**	**Dimensions (in)**	**Cover (in)**
150020	L1	1.5	Prestressed Pile	20 x 20	2
	L2	1.5	Prestressed Pile	20 x 20	2
660019	L1	0.5	Prestressed Pile	16 x 16	2
	L2	0.5	Prestressed Pile	16 x 16	2
260007	L1	2.5	Bent 3 Cap	33 x 50	2
	L2	2.5	Bent 3 Cap	33 x 50	2
640010	L1	3.0	Bent 1 Cap	30 x 33	2
	L2	1.0	Prestressed Pile	16 x 16	2
	L3	3.0	Bent 3 Cap	30 x 33	2
	L4	1.5	Prestressed Pile	16 x 16	2
660021	L1	1.0	Prestressed Pile	12 x 12	2
	L2	1.0	Prestressed Pile	12 x 12	2
090061	L1	-1.0	Prestressed Pile	20 x 20	2
	L2	0.0	Prestressed Pile	20 x 20	2
150026	L1	2.5	End Bent	12 x 33	2
	L2	0.5	Bent 1 Cap	42 x 44	2
	L3	0.5	Bent 1 Cap	42 x 44	2

Table 4.11: Exposure condition inputs of modeled locations

Structure Number	**Location**	**Vertical Distance from High Tide Elevation (ft)**	**Buildup Period (yrs)**	**Surface Concentration (% wt. conc.)**	**Temperature Region**
150020	L1	1.5	12	0.129	Jacksonville
	L2	1.5		0.225	
660019	L1	0.5	11	0.246	Jacksonville
	L2	0.5		0.392	
260007	L1	2.5	13	0.111	Nags Head
	L2	2.5		0.252	
640010	L1	3.0	13	0.038	Wilmington
	L2	1.0		0.371	
	L3	3.0		0.111	
	L4	1.5		0.582	
660021	L1	1.0	15	0.061	Jacksonville
	L2	1.0		0.162	
090061	L1	-1.0	14	0.535	Wilmington
	L2	0.0		0.830	
150026	L1	2.5	14	0.012	Jacksonville
	L2	0.5		0.879	
	L3	0.5		0.281	

Both methods of defining the concrete characteristics relied upon mixture design properties and components such as *w/cm*, percentage of fly ash, silica fume, or slag utilized, and the dosage of corrosion inhibitors. These properties were known based upon mixture designs provided by the NCDOT research project steering committee and are shown in Table 4.12. The impact each of these pozzolans and the use of corrosion inhibitor is explained in depth in Chapter 3.4.3 of this thesis.

Table 4.12: Corrosion protection in concrete mixtures

Structure Number	Location	Corrosion Inhibitor	Fly Ash	Slag	Silica Fume
150020	L1	3.0 gal/CY	30%	-	-
	L2		30%	-	-
660019	L1	3.0 gal/CY	-	-	5%
	L2		-	-	5%
260007	L1	3.0 gal/CY	-	43%	-
	L2		-	43%	-
640010	L1	3.0 gal/CY	25%	-	5%
	L2	3.5 gal/CY	-	-	5%
	L3	3.0 gal/CY	25%	-	5%
	L4	3.5 gal/CY	-	-	5%
660021	L1	3.5 gal/CY	-	-	-
	L2		-	-	-
090061	L1	3.0 gal/CY	-	-	5%
	L2		-	-	5%
150026	L1	3.0 gal/CY	30%	-	-
	L2		30%	-	-
	L3		30%	-	-

When allowing Life-365 to assume concrete properties based upon the mixture proportions shown in Table 4.12, the diffusion coefficient is the only value that is calculated differently than when utilizing the field measured values. With all other values in the modeling process held constant, the diffusion coefficient calculation is the driving factor introducing variability between both methods of service life modeling. The input values used to define concrete properties using the field measured method and the Life-365 assumed method can be seen in Table 4.13. Input values for hydration period, propagation period, and chloride threshold value (Ct) were constant between both methods.

Table 4.13: Concrete property input values of modeled locations

Structure Number	**Location**	**Constant Values**			**Field Measured**		**Software Assumed**	
		Hydration (yrs)	**Propagation Period (yr)**	**Ct (% wt. conc.)**	**Diffusion Coefficient at 28 days (in^2/yr)**	***m***	**Diffusion Coefficient at 28 days (in^2/yr)**	***m***
150020	L1	25	6	0.24	0.498	0.44	0.388	0.44
	L2				0.435		0.388	
660019	L1			0.24	0.052	0.26	0.170	0.20
	L2				0.050		0.170	
260007	L1			0.24	0.473	0.45	0.388	0.45
	L2				0.236		0.388	
640010	L1			0.24	0.249	0.40	0.170	0.40
	L2			0.28	0.056	0.26	0.170	0.20
	L3			0.24	0.164	0.40	0.170	0.40
	L4			0.28	0.058	0.26	0.170	0.20
660021	L1			0.28	0.185	0.26	0.388	0.20
	L2				0.117		0.388	
090061	L1			0.24	0.558	0.26	0.170	0.20
	L2				0.341		0.170	
150026	L1			0.24	6.213	0.44	0.388	0.44
	L2				0.237		0.388	
	L3				0.356		0.388	

The results of the service life modeling process utilizing both methods have been summarized in Table 4.14. The total maintenance free bridge service life was reported as the minimum service life modeled. This is because the actual service life of the bridge is driven by the time until the first failure takes place. In some cases, there are large differences between the service lives predicted by each method for a single location. Being that the main difference between the two methods was how the diffusion coefficient was calculated, these large differences emphasize the importance of the diffusion coefficient in the modeling process. Due to the large amount of assumptions that must be made about concrete performance through the software assumed method it is typically believed that the field measured method produced the more representative results. Utilizing the field measured input method all structural concrete bridge elements observed were predicted to have a maintenance free service life greater than their design service life of 50-years.

Table 4.14: Summary of service life modeling

Structure Number	Location	Corrosive Zone	Location Description	Vertical Distance from High Tide Elevation (ft)	Distance from Open Water (mi)	Field Measured		Software Assumed	
						Component Service Life (yrs)	Total Maintenance Free Life (yrs)	Component Service Life (yrs)	Total Maintenance Free Life (yrs)
150020	L1	Corrosive	Prestressed Pile, Infrequent Wetting	1.5	1.0	506.0+	506.0+	506.0+	506.0+
	L2		Prestressed Pile, Infrequent Wetting	1.5		506.0+		506.0+	
660019	L1	Corrosive	Prestressed Pile, Tidal Zone	0.5	1.5	506.0+	261.6	506.0+	62.7
	L2		Prestressed Pile, Tidal Zone	0.5		261.6		62.7	
260007	L1	Highly Corrosive	Bent Cap, Rare Occurrence Wetting	2.5	0.0	506.0+	506.0+	506.0+	506.0+
	L2		Bent Cap, Rare Occurrence Wetting	2.5		506.0+		506.0+	
640010	L1	Highly Corrosive	Bent Cap, Very Infrequent Wetting	3.0	0.3	506.0+	150.4	506.0+	44.6
	L2		Prestressed Pile, Tidal Zone	1.0		391.7		100.5	
	L3		Bent Cap, Very Infrequent Wetting	3.0		506.0+		506.0+	
	L4		Prestressed Pile, Tidal Zone	1.5		150.4		44.6	
660021	L1	Highly Corrosive	Prestressed Pile, Infrequent Wetting	1.0	0.8	506.0+	506.0+	506.0+	506.0+
	L2		Prestressed Pile, Infrequent Wetting	1.0		506.0+		506.0+	
090061	L1	Highly Corrosive	Prestressed Pile, Tidal Zone	-1.0	1.6	42.3	24.8	24.8	24.8
	L2		Prestressed Pile, Tidal Zone	0.0		24.8		30.9	
150026	L1	Highly Corrosive	End Bent, Very Infrequent Wetting	2.5	4.2	506.0+	63.2	506.0+	41.7
	L2		Bent Cap, Tidal Zone	0.5		63.2		41.7	
	L3		Bent Cap, Tidal Zone	0.5		324.2		298.1	

Many of the locations modeled have very long service lives reported as 506.0+ years. This is the case for bridges 150020, 660021, 260007, and certain elements of most other bridges analyzed. This value is reported when the modeled service life surpasses the maximum allowable analysis period of 500-years in Life-365 with a six-year propagation period. Service lives of this length are generally unrealistic as other deterioration mechanisms or changes in service conditions (such as traffic capacity) will prevent a structure from safely and efficiently functioning for this long. However, because Life-365 only considers deterioration due to corrosion this implies that corrosion is unlikely to be a major issue for these elements of the structures analyzed. This bodes well for the NCDOT's corrosion policy because the structures considered as part of this work were constructed under this policy, and the design service life of 50-years should easily be achieved. These findings are believed to be representative of all bridges within the corrosive zones due to the sample population encompasses typical bridges in a wide range of locations spanning the entire coast of North Carolina.

Based upon the results of the modeling it appears that main factor impacting the service life is the frequency in which the concrete member is exposed to the chloride rich waters. In all cases where locations experienced infrequent, very infrequent, or rare occurrence wetting the maximum service life of 506.0+ years was reported by both modeling methods. This indicates that for locations where concrete elements that do not undergo heavy chloride loading from regular exposure to chloride rich waters the risk of corrosion related deterioration is highly improbable. In areas referred to as tidal zones where there is frequent wetting and drying of the concrete due to tidal induced changes in water level, the service lives are shown to be impacted. This frequent wetting and drying of the concrete has been known to enable chloride ingress due to absorptivity and sorptivity properties of concretes capillary pore structure (ACI 2010). Measured values of surface concentration of chlorides at these locations confirm that this phenomenon is true. The average surface concentration amongst all locations sampled was 0.307 % wt. conc. and all but one tidal zone location tested were shown to have a higher than average surface concentration. As a result of this, it was found that in areas containing the highest measured surface concentrations the modeled maintenance free service life was decreased.

There was no indication that the proximity to coastline or major bodies of water had an impact on the service lives of the bridges modeled. Bridge 260007 was the only bridge modeled that was directly located on the coastline and not a tributary stream or river. If proximity to a major body of water had an impact on service life, it would have likely been observed in this bridge. However, the bent cap modeled was predicted to have an exceptional service life of over 500 years. Similar trends are seen with most other bridges within both the corrosive and highly corrosive zones. These findings support the previous assertion that the factor impacting service life the most is the actual exposure to the chloride rich waters (even intermittently) and not proximity to them.

There is not a perceivable difference between service lives of bridges in both corrosive zones. The impact on service life appears to be more closely related to the exposure of the concrete to brackish waters. For this reason, the corrosion mitigation policy as it applies to members exposed to heavy chloride loading may need to be enhanced to ensure that the 50-year service life is met. However, the corrosion mitigation policy may be overly strict for members that are not frequently exposed to chlorides. It is recommended that further research be done to confirm this assertion.

4.4 Corrosion Inhibitor Concentration

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). The locations of the samples utilized are shown in Chapter 4.1 of this thesis.

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 R^2 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 4.15 and the resulting calibration curve is shown in Figure 4.19.

Table 4.15: 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

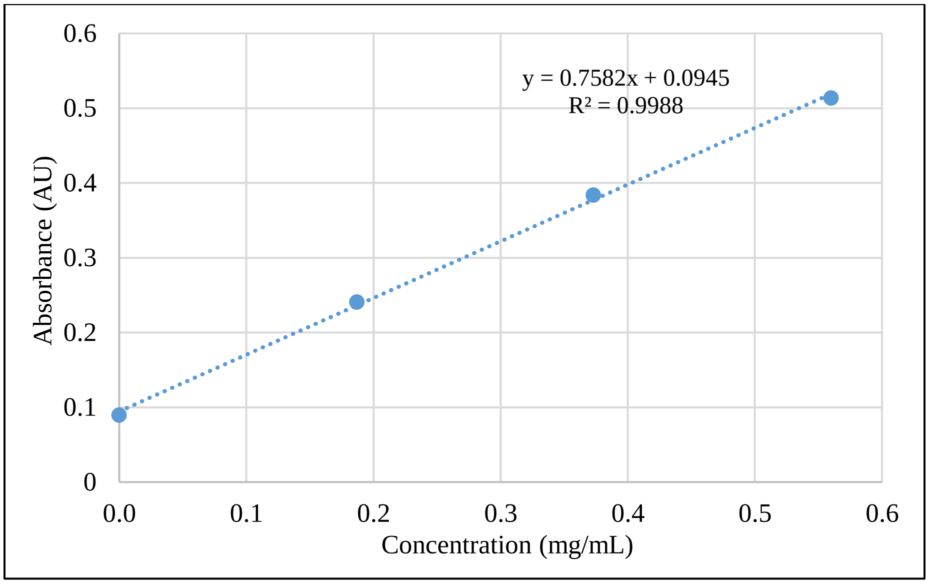


Figure 4.19: Calibration curve for calcium nitrite determination

Utilizing the equation resulting from 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 4.16. The report generated by the spectrophotometer software at the completion of this testing has been included in Appendix B of this thesis.

Table 4.16: 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	0.090
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

The concentration values resulting from the calibration curve equation are in units of mg/mL. However, the NCDOT specifies calcium nitrite concentration in units of gallons per cubic yard. To compare measured concentrations to the minimum specified concentration of 3.0 gal/CY these values were converted from mg/mL to gal/CY by utilizing the equations provided in Chapter 3.3.2 of this thesis. These concentration values in gal/CY can be seen in Table 4.17 as “Measured Concentration”.

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). This is the result of an analysis done in 1989 of 200 standard samples all containing 658 pcy of cement (Jeknavorian 2005). As a result of the uncertainty in recovery rate two values for the theoretical concentration are reported beside the "Measured Concentration" in Table 4.18. 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 only 85%. This is a more optimistic value which assumes that 15% of the nitrite was not extracted and thus the reported theoretical concentration will be the highest.

Table 4.17: 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	2.80
	5	2.84	2.96	3.35
660021-L1	2	3.87	4.03	4.55
	5	3.20	3.33	3.76
090061-L1	2	1.96	2.04	2.30
	5	3.05	3.18	3.59
150026-L2	2	2.79	2.90	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 4.20. 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 lowered concentration of calcium nitrite. It is believed that this is what caused the low concentrations of calcium nitrite on several of the tests, but the results remain valid based upon the ability to achieve a specified concentration from the companion sample. This same principle can be applied to samples where the calcium nitrite concentration was detected to be higher than the concentration reportedly utilized in the mixture designs. In these situations, a larger volume of paste would be included in the sample than aggregate and thus artificially increasing the concentration recovered.

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 indicates that the corrosion inhibitor admixture is being mixed uniformly within the concrete mixture, which would be expected. This is desirable to ensure that all embedded steel is afforded the same level of corrosion protection by the admixture.

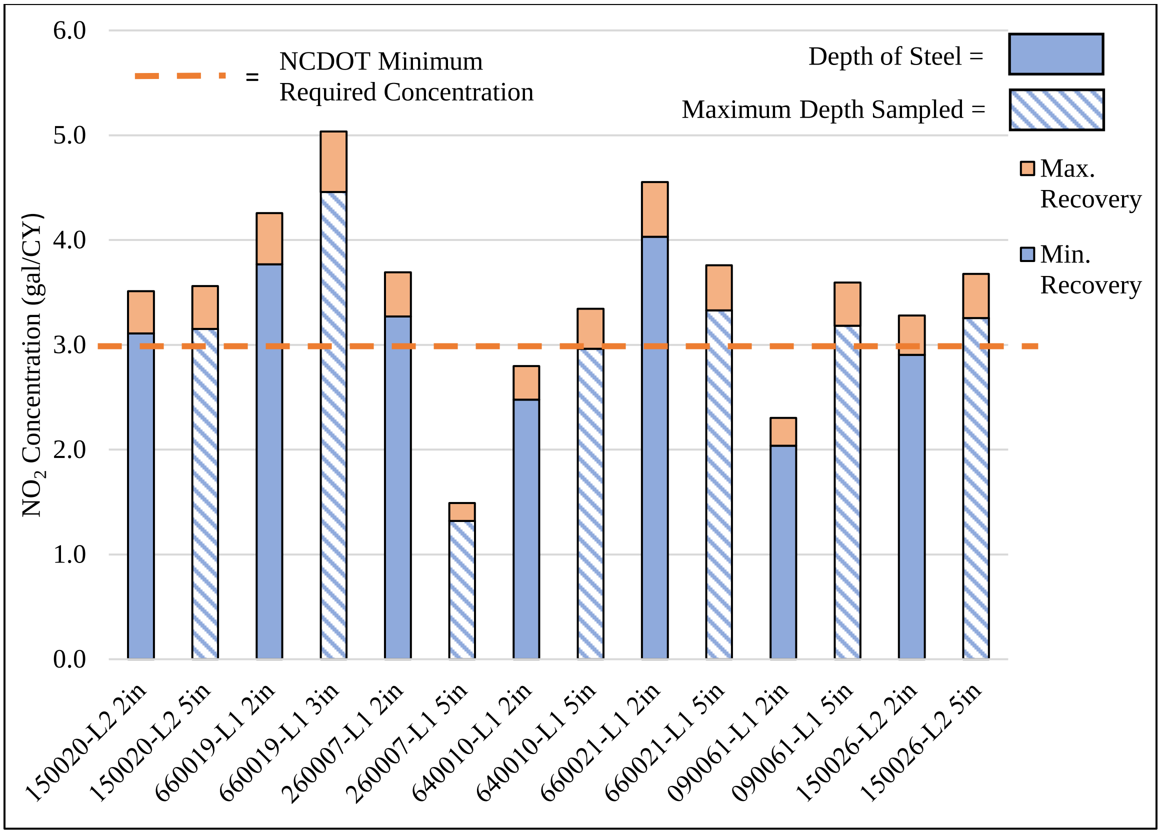


Figure 4.20: Theoretical calcium nitrite concentrations

As part of NCDOT quality assurance testing this same procedure (W.R. Grace Chemical Procedure #C-20.0) is utilized to verify that correct dosages of corrosion inhibitor have been utilized in concrete structures. The NCDOT provided test data for four bridge locations to correlate with the results of the testing previously discussed. This data as well as the laboratory measured concentration from the testing done for this thesis are included in Table 4.18. Having test data available from shortly after the concrete component was constructed allows for an analysis of if time has an impact on the concentration of calcium nitrite corrosion inhibitor left within the concrete. This analysis may not be valid however because the tests performed by the NCDOT are only linked to a mixture design utilized on the project. This introduces variability to the analysis because the powder sample taken by the NCDOT may not have been from the same element or location as was tested for this thesis.

Table 4.18: Calcium nitrite concentration as measured by UNCC and NCDOT

Bridge-Location	UNCC Measured		NCDOT Measured	Test Status	% Difference NCDOT Measured vs. Lab 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%

When comparing the nitrite concentrations determined from acceptance testing conducted by the NCDOT with the results from the UNCC laboratory, as shown in Table 4.18, there is no discernible trend indicating that the concentration will be impacted by time. However, due to the uncertainties discussed previously further research is recommended to confirm this assertion. It is also important to note that the NCDOT measured values reported in Table 4.18 are an average of several tests conducted on 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 location had at least one test indicating that the concentration was above 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 failed tests, indicate that this procedure for determining nitrite concentration is utilized as a simple validation that calcium nitrite is present within the concrete mixture. Further understanding of the NCDOT acceptance process is required to validate this assertion.

4.5 Corrosion Rate and Surface Resistivity

Field testing utilizing the Giatec iCOR nondestructive testing device allowed for the determination of active corrosion rates and surface resistivities in the areas were drilled powder samples were taken. Various other areas of each bridge were analyzed and mapped using the iCOR and in-depth reporting of those results can be seen in Violette (2020), "Evaluating Corrosive Site Performance and Policy with Concrete Admixtures." Having conducted this testing allowed for correlations to be made between the corrosion rate, surface resistivity, diffusion coefficient, and chloride concentration at the depth of steel (2-inches in all cases). Specifically, two correlations between 1.) corrosion rate and chloride concentration at the depth of steel and 2.) surface resistivity and diffusion coefficient were of interest. The rationale behind this is that theoretically 1.) there should be increased corrosion rates in areas where chloride concentration at the steel is highest and 2.) high surface resistivities in areas with low diffusion coefficients.

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 4.19 (Erdogdu, et al. 2004, Giatec Scientific Inc. 2020).

Table 4.19: Corrosion rate, surface resistivity, and diffusion classifications

Corrosion Rate (μm/yr)	Classification
< 10	Passive/Low
10 - 30	Moderate
30 - 100	High
> 100	Severe

Surface Resistivity (KΩ·cm)	Classification
> 100	Very High
50 - 100	High
10 - 50	Moderate
< 10	Low

Diffusion Coefficient (in^2/yr)	Classification
< 0.3	Low/Passive
0.3 - 0.6	Moderate
> 0.6	High

Field-testing results were compiled into a summary table and displayed in Table 4.20. Corrosion rates and surface resistivities are reported in two ways: as an average and as a worse case maximum and minimum value for corrosion rate and surface resistivity respectively. This was done because for both these characteristics four to six measurements were taken within close proximity of the location where the drilled powder samples were taken. No one measurement is able to be correlated to the exact location of the drilled samples because testing with the iCOR required a location directly over steel and drilling to acquire powder samples required avoiding the steel entirely. Therefore, an analysis area, not exceeding one foot in all directions, was utilized to represent the corrosion rate and surface resistivity of the drilled location. Measurements of corrosion rate and surface resistivity sometimes varied largely within this relatively small analysis area and it is important to consider both the average condition and the worst-case condition as this is where signs of corrosion related deterioration are likely to manifest.

Table 4.20: Summary of field-testing results

Structure Number	**Location**	**Corrosion Rate (μm/yr)**		**Surface Resistivity (kΩ·cm)**		**Diffusion Coefficient (in^2/yr)**	**2-in Chloride Concentration (pcy)**
		Avg.	**Max.**	**Avg.**	**Min.**		
150020	L1	5	11	190	109	0.054	0.28
	L2	4	7	518	407	0.047	0.39
660019	L1	56	90	81	49	0.032	0.11
	L2	38	136	96	51	0.031	0.17
260007	L1	22	52	187	67	0.048	0.40
	L2	31	52	137	67	0.024	0.16
640010	L1	3	7	361	172	0.073	0.14
	L2	42	77	118	91	0.033	0.39
	L3	3	5	103	75	0.048	0.28
	L4	-	-	-	-	0.034	0.65
660021	L1	4	7	212	200	0.046	0.16
	L2	5	10	176	158	0.029	0.15
090061	L1	120	274	28	7	0.322	9.76
	L2	154	251	23	7	0.197	11.45
150026	L1	16	52	254	134	0.628	0.22
	L2	14	28	305	199	0.024	0.39
	L3	9	28	341	199	0.036	0.40

Note: “-“ Indicates that the data was lost due to a technical issue

Correlation of the chloride concentration at the depth of steel and the corrosion rate is displayed in Figure 4.21. As previously discussed in Chapter 4.1 of this thesis, the chloride concentrations in the concrete decreased dramatically within the first one to two inches from the concrete surface. This resulted in a cluster of very low measurements for 2-inch chloride concentrations. This is cluster contains 88% of the sample population and is indicative of concrete that is very impermeable or does not receive heavy chloride loading. Only two readings, both from Bridge 090061, did not match this trend and are classified as high chloride concentrations. Average corrosion rates measured were generally low with 43% of locations being categorized as passive/low, 19% as moderate, 25% as high, and 13% as very high. The two corrosion rates that were classified as very high were from Bridge 090061 where the high chloride concentrations were also measured. Despite the majority of readings forming a cluster of low chloride concentrations and low corrosion rates, these two readings from bridge 090061 fit the theoretical trend that the corrosion rate will increase with an increased chloride concentration at the depth of steel. Further testing will be needed to confirm this as this trend it anchored only by two of sixteen data points.

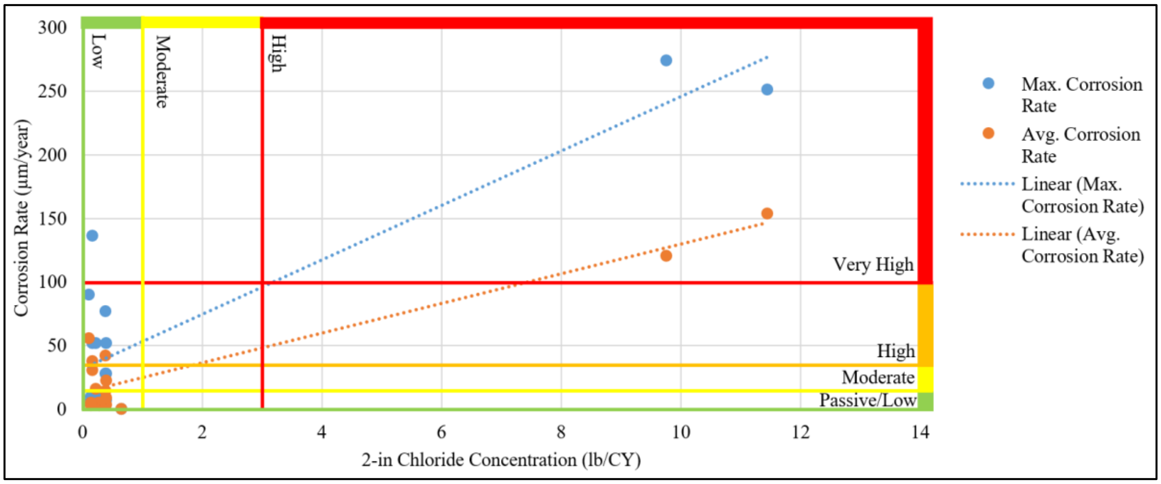


Figure 4.21: 2-in chloride concentration vs. corrosion rate

Correlation of the measured diffusion coefficient and locally measured surface concentration is displayed in Figure 4.22. Approximately 88% of the sample population is calculated to have a diffusion coefficient classified as passive or low. These low diffusion coefficients are indicative of concrete that is very impermeable which is anticipated as all but one of these mixtures contains silica fume, fly ash, or slag. Amongst these locations that have been tested to have low diffusion coefficients there is a wide range of surface resistivities. The average local surface resistivities measured range from the low 20’s to the 500’s of kohm·cm, with 75% of the sample population categorized as having a very high surface resistivity. The trend of increasing surface resistivities with relatively constant and low diffusion coefficients leads to the conclusion that no matter how high the surface resistivity the diffusion coefficient cannot decrease indefinitely. This is due to the nature of concrete’s capillary pore structure never being completely impermeable and therefore ensuring that the diffusion coefficient does not reach zero.

The data point for diffusion coefficient from 150026-L1 appears to be an outlier in this data set as its high surface resistivity should result in a low diffusion coefficient. However, at this location very low and consistent concentrations of chloride were detected across all depths sampled. It is likely that due to these locations limited exposure to chloride-rich waters minimal chloride loading has been experienced. The concentrations detected are most likely background chlorides present in the mixture. The diffusion coefficient was calculated based upon the trend of chloride concentrations decreasing as the depth into the concrete increases. Because these locations showed almost constant concentrations across all depths the diffusion coefficient calculations yielded what were artificially high values.

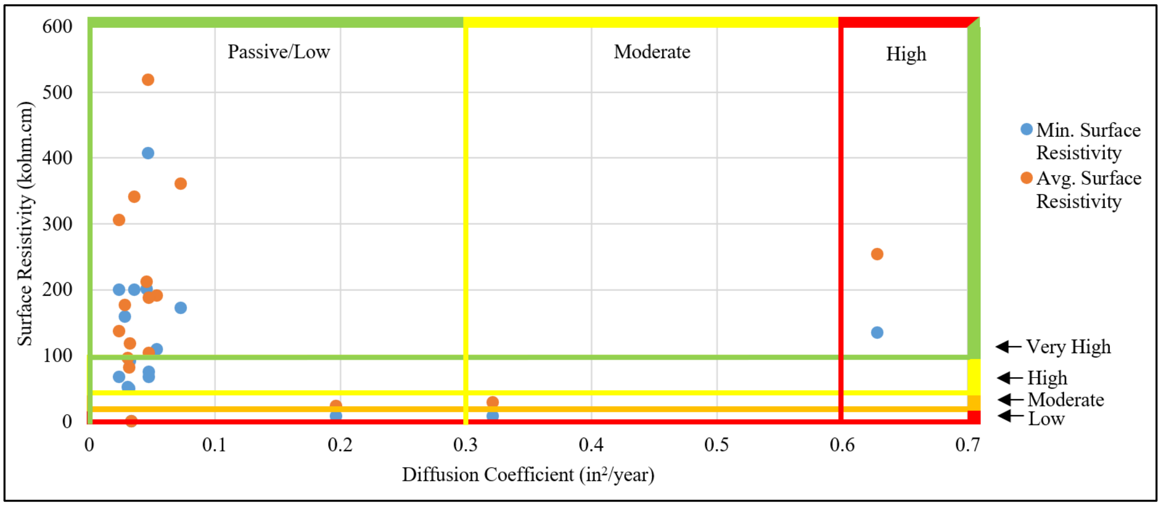


Figure 4.22: Diffusion coefficient vs. surface resistivity

A final analysis to correlate the field measured values for surface resistivity and corrosion rate at each location was conducted as seen in Figure 4.23. The data points for the average values as well as the worst-case scenario values fit a power trend indicating that as the surface resistivity of a location is increased the corrosion rate at the location is decreased, and vice versa. This is an expected result and serves as a good verification that the testing equipment and the results gathered from it are logical.

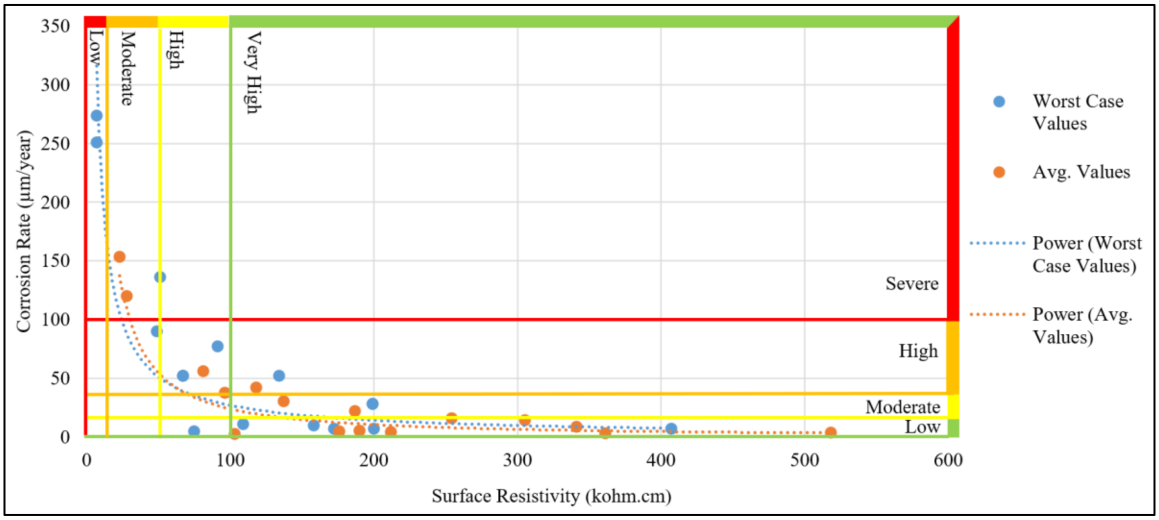


Figure 4.23: Surface resistivity vs. corrosion rate

The curve created by correlating surface resistivity with corrosion rate readings (Figure 4.23) is very similar to the relationship between surface resistivity to rapid chloride permeability tests (RCPT) that has been found in a previous research study for the Louisiana Transportation Research Center and is shown in Figure 4.24 (Rupnow and Icenogle 2011). RCPT testing is similar to surface resistivity testing as it is an electrical test utilized to evaluate the resistance of a concrete sample. The difference is that the surface resistivity meter measures the resistance of the sample containing its own pore solution and/or curing solution to electricity, while RCPT measures the resistance of a conditioned (vacuum saturated concrete sample) to chloride ion penetration using a salt solution.

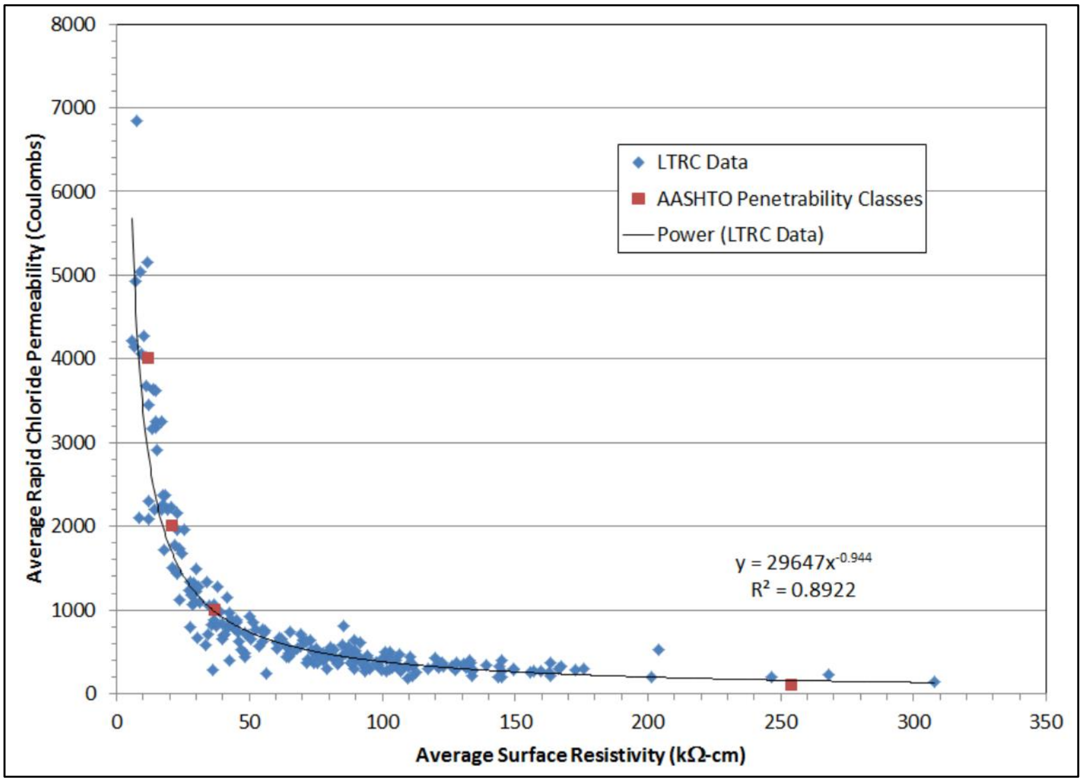


Figure 4.24: Surface resistivity vs. RCPT (Rupnow and Icenogle 2011)

Based upon the similar trends that surface resistivity has with both RCPT and corrosion rate, it is possible that there could be a relationship between corrosion rate and RCPT. If this relationship can be established, the determination of corrosion rates and chloride permeability (RCPT) could be accomplished with only surface resistivity measurements. This would result in significant cost savings in quality assurance and assessment due to the time and manpower that would be saved by only needing to utilize surface resistivity equipment, and the upfront equipment cost for the surface resistivity test is much lower than the other two tests as well. However, significant additional research to expand the sample population and collect more information would be needed to confirm that the trends between these tests can be correlated.

4.6 Summary of Findings

Findings from the field and laboratory research conducted for this thesis indicate that there are common trends between concrete performance and exposure and their impact on the maintenance free service lives of bridges within the corrosive zones. Specifically, conclusions form this study include:

- RCT tests indicated a rapid decline of chloride concentration between the surface and two inches into almost all elements. This rapid decline was always followed by the leveling out of the concentration to background levels over the next several inches until the maximum sampling depth was reached. This indicates that there has not been appreciable chloride penetration after the two-inch depth and that there are low levels of background chlorides in all bridges. Since the background concentrations are significantly lower than typical chloride threshold values there is little to no risk of them initiating corrosion.

- Common values for diffusion coefficients for generic portland cement concrete in chloride-contaminated environments range from 0.3 to 0.6 in^2/yr (Erdogdu, et al. 2004). In almost all cases the measured diffusion coefficient was much lower than this range. These results are consistent with those that could be expected from high quality concrete containing one or more pozzolans within it, or from precast concrete members fabricated under highly controlled conditions. Previously conducted studies of the Virginia Dare Bridge, located on the Outer Banks of North Carolina, explored the impact of silica fume and fly ash use on chloride diffusion coefficient rates. This field gathered data from bridge pier concrete, with similar mixture designs as seen in the bridges observed for this study, verified that the diffusion coefficients measured were within the same order of magnitude that would be anticipated from concrete with these exposure conditions (Tempest et al. 2017).
- Surface chloride concentrations calculated for concrete structural elements in tidal zones were found to be higher than in areas not subjected to frequent exposure to brackish waters.
- Surface chloride concentrations are shown to be lower for bridges located in the corrosive zone than for bridges located in the highly corrosive zone. However, due to the large variation in surface concentration results gathered over the course of this study further research is recommended in this area.
- Service life modeling results indicated that the main factor impacting the service life is the tendency for the concrete member to be exposed to (or intermittently exposed to) chloride rich waters. In all cases where locations experienced

infrequent wetting the maximum maintenance free service life that the model could predict, over 500-years, was reported by both modeling methods. This indicates that for locations where concrete elements that do not undergo heavy chloride loading from regular exposure to chloride rich waters the risk of corrosion related deterioration is highly unprobeable. In areas where there is frequent wetting and drying of the concrete due to tidal fluctuations the maintenance free service lives may be impacted.

- There was no indication that the proximity to coastline or major bodies of water had an impact on the service lives of the bridges included as part of this work.
- There is not a perceivable difference between service lives of bridges in both corrosive zones. The impact on service life appears to be more closely related to the exposure of the concrete to brackish waters of any kind.
- In most cases the theoretical calcium nitrite concentration detected from powder samples taken in the field met or exceeded the required NCDOT minimum concentration. This verifies that calcium nitrite is being utilized in the mixtures in approximately the correct dosage rates.
- 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 indicates that the corrosion inhibitor admixture is being mixed uniformly within the concrete mixture.
- When comparing the calcium nitrite concentrations determined from acceptance testing conducted by the NCDOT with the results from the UNCC laboratory

there is no discernible trend indicating that the concentration of corrosion inhibitor will be impacted by time. However, due to the uncertainties discussed in Chapter 4.4 of this thesis further research is recommended to confirm this assertion.

- Chloride concentrations measured at the depth of reinforcing steel resulted in “low” measurements of less than 1.0-pcy for 88% of the sample population.
- Average corrosion rates measured resulted in 43% of locations being categorized as passive/low, 19% as moderate, 25% as high, and 13% as very high.
- Average surface resistivities measured range from the low 20’s to the low 500’s with 75% of the sample population categorized as having a very high surface resistivity.
- Measured values indicate that as the surface resistivity measured at a location is increased the corrosion rate at the location is decreased, and vice versa.
- The curve created by correlating surface resistivity with corrosion rate readings (Figure 4.23) is very similar to the relationship between surface resistivity to rapid chloride permeability tests (RCPT) that has been found in a previous research study (Rupnow and Icenogle 2011). Based upon the similar trends that surface resistivity has with both RCPT and corrosion rate, it is possible that there could be a relationship between corrosion rate and RCPT. Further research to expand the sample population and collect more information would be needed to confirm that the trends between these tests can be correlated.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The NCDOT corrosion protection policy currently in place for corrosive zones is fairly comprehensive and the required provisions afford redundancy. It specifies use of one or more methods known to offer protection against corrosion, using several mechanisms, to concrete structural elements that are most likely to be exposed to chloride rich coastal waters. These methods almost always consist of a twofold approach utilizing one protection measure to make the concrete less permeable to chlorides (with the use of silica fume, fly ash, or slag) and a second protection measure to reduce the ability for reinforcing steel to begin corroding (with the use of epoxy coated steel and corrosion inhibitor).

Supporting the effectiveness of the current policy, all bridges visited during the project showed no visual signs, or very limited signs, of corrosion related distress after ten to fifteen years of service. Service life modeling predicting the future performance of these bridges indicated that in most cases the high level of corrosion protection offered by NCDOT’s current corrosion policy should result in a maintenance free service life often vastly extending beyond the designed service life of 50-years. All cases where the maintenance free service life was predicted to be lower than the design service life can be correlated to the tendency of the concrete structural elements to be in exposure to chloride rich waters. These areas, known as tidal zones, that undergo regular wetting and drying due to daily tidal shifts are well known to be most vulnerable to corrosion. The tendency of elements in the tidal zone to be predicted to fail to meet service life goals was found to hold true across both the corrosive and highly corrosive zones specified in the NCDOT Structure Design Manual. Between the two zones, regardless of a bridge's proximity to the coast, concrete components that were infrequently or rarely subjected to direct contact from coastal waters were predicted to have sufficient corrosion protection methods in place to avoid being at risk for corrosion related deterioration throughout their service life.

5.2 Recommendations to Current Specifications

The current specification for corrosion protection is outlined in depth in Chapter 2.4.1 of this thesis. The primary parts of the specification related to the study conducted for this thesis along with all findings and recommendations about each part are summarized as follows:

- Corrosive zone corrosion protection is utilized in varying degrees dependent upon the location of the bridge. The NCDOT specifies two corrosive zones shown in Figure 1.1 (NCDOT 2019). The corrosive zone is located on or east of the blue line and the highly corrosive zone is located on or east of the red line.
    - Findings/Recommendations: The designation of a corrosive and highly corrosive zone is an important characteristic the specification, requiring varying levels of corrosion protection dependent upon the level of environmental chloride exposure is important to allowing the policy to be appropriately utilized. Based upon the results of this study presented in this thesis, no statement can currently be made regarding whether the two corrosive zone system specified by the NCDOT policy is the most appropriate way to designate the necessary corrosion protection measures.

This is because the primary factor found to impact the predicted service life was the tendency of the elements to be exposed to chloride rich waters and results were indeterminant as to if the proximity to the coast had any correlation to predicted service life. During the literature review, it was found that several coastal states use salinity maps to assist in delineating highly corrosive and mildly corrosive zones. Future research to evaluate the salinity of the waters at each bridge location could allow for a sound scientific analysis of the environmental exposure differences between the two zones. Alternatively, salinity maps of the North Carolina coastline could be utilized to assist in delineating these zones based upon the water's salinity. Completion of a salinity study would allow for the NCDOT policy to be revised to include corrosive zone designations based upon chloride concentration of the water. This would be a similar approach to how the FDOT classifies corrosive zones. The FDOT corrosion protection policy is based upon measured salinity at the bridge location and specifies corrosion protection measures for the substructure and superstructure individually based upon chloride exposure risk. It is recommended that this policy be utilized as a reference for a revised NCDOT policy.

- Corrosion protection measures for both corrosive zones are limited to stream crossings. Bridges that do not serve as a stream crossing do not require special provisions for corrosion protection.

  - Findings/Recommendations: Constraining specifications for corrosion protection measures to only bridges located at stream crossings was determined to be an appropriate practice. This was due to the finding of substantial decreases in chloride ingress and surface concentration found on bridge elements only several feet from the tidal zone. It is unlikely that bridges not within proximity to water would experience high levels of chloride loading from the environment. Exceptions to this would be locations where bridges experiences heavy application of deicers and road salt during the winter months.
- For both corrosive zones, elements that undergo repeated wetting and drying due to tidal fluctuations require that 5.0% of the portland cement be substituted with silica fume (a proportional replacement of Class F fly ash may also be approved).
  - Finds/Recommendations: The specification of silica fume in tidal and splash zones was found to be a warranted corrosion protection measure due to the finding that these areas are most likely to have service lives impacted by corrosion deterioration. In most cases where concrete structural elements within the tidal zone were observed this corrosion protection measure of the policy was found to be effective in allowing bridges to exceed their maintenance free service life. However, of the eight concrete elements exposed to these conditions across four different bridges, two locations from the same bridge were predicted to have maintenance free service lives below their design service lives. This bridge observed also was determined to have a higher than anticipated

diffusion coefficient (>0.100 in^2/year) and as a result was observed to have substantial chloride loading within the tidal zone. Due to this, the bridge was predicted to have a maintenance free service life of only half of what it was designed for. It is unclear if this bridge is simply an outlier of the data, or if it is indicating a potential issue with the corrosion protection afforded by the policy. Further sampling of bridges within the tidal zone is recommended to further explore these findings.

- The following corrosion protection measures are applicable to the corrosive zone (on or east of blue line) for elements within 15 feet of the mean high tide and all concrete bridge elements at stream crossings in the highly corrosive zone (on or east of the red line):
    - *Class AA concrete for bridge decks shall contain fly ash or granulated blast furnace slag at the approved substitution rates.*
    - *Class AA concrete shall be used in all cast-in-place columns, bent caps, pile caps, and footings, and shall contain calcium nitrite corrosion inhibitor.*
    - *The water/cement ratio for concrete piles shall not exceed 0.40.*
    - *All bar supports used in the (barrier rail, parapet, sidewalk, deck, bent caps, columns, pile caps, footings) and all incidental reinforcing steel shall be epoxy coated.*
    - *Prestressed concrete (girders, precast deck panels, cored slab units, piles) shall contain calcium nitrite corrosion inhibitor.*

    - Findings/Recommendations: The methods of corrosion protection utilized in these areas were shown in this study to be adequate in reducing the risk of corrosion related deterioration. It was found that in structural concrete elements not located within the tidal zone these corrosion protection measures provided a conservative approach given the limited chloride loading they experienced from the environment. However, it is not recommended that the corrosion protection constraints of the policy be limited beyond their current state in any way.

The corrosion protection measures specified by this policy are very comprehensive and are based upon some of the best practices known in concrete construction. Based upon the results of the research conducted for this thesis the policy appears to provide sufficient protection for the bridges within both corrosive zones. In some cases the policy is likely more conservative than it needs to be. This appears to be the case when the policy requires full corrosion protection measures on concrete elements that are not exposed to repeated wetting and drying due to tidal fluctuations or will never experience direct exposure to brackish waters but are within 15 feet of the mean high tide. In these areas, sometimes as little as two feet from the mean high tide, there was shown to be very little chloride content within the concrete, indicating that there is not a high probability for chloride induced corrosion deterioration to manifest. However, although evidence suggests that corrosion policy is excessive for elements outside of the tidal zone, it is not suggested that it be pared back from the minimum standards that it currently specifies. This is because having a high level of corrosion protection in place serves to ensure that variations in exposure, concrete properties, or construction quality will not be detrimental to the concrete structural element meeting its design service life.

The corrosion policy identifies that elements exposed to repeated wetting and drying due to tidal fluctuations are particularly susceptible to corrosion deterioration. The policy addresses this by specifying that a minimum of 5.0% silica fume be utilized in these structural concrete elements. The research conducted in this study concluded that this is an effective corrosion protection measure because most of these concrete structural elements observed within the tidal zone were predicted to have maintenance free service lives extending beyond their design service lives. However, there were some areas exposed to repeated wetting and drying were the design service life of 50-years was not predicted to be met. The best example of this was bridge 090061 where two prestressed piles within the tidal zone were tested. The mixture design for these piles contained 5.0% silica fume and 3.0 gal/CY of corrosion inhibitor as specified by the policy. Despite these protection measures high levels of active corrosion were measured and the predicted maintenance free service lives modeled were fairly low at less than 25-years. Due to the limited number of samples obtained from bridges within the tidal zone it cannot be determined if this bridge is simply an outlier of the data, or if it is indicating a potential issue with the corrosion protection afforded by the policy. Further sampling of bridges within the tidal zone is recommended to further explore these findings.

One possible explanation for the finding that one bridge was not predicted to meet its designed maintenance free service life is that there was a quality related defect impacting the performance of this particular structure. Given that the same type of concrete structural elements (prestressed piles) in similar conditions with the same corrosion prevention methods were able to achieve desirable service lives, the possibility of a production related defect impacting the quality of the prestressed piles utilized on that bridge cannot be ruled out. Although precast components are typically subjected to higher levels of quality control than cast in place components construction defects, deviations from the plans, or the use of less than desirable concrete is still possible. This was made apparent when visual inspection of bridge 660091 revealed that a large honeycomb defect on a precast concrete cored slab unit had left a prestressed strand exposed in a limited area. This precast element was accepted for use even though this defect was left unrepaired which allowed it to be highly susceptible to corrosion deterioration. The addition of stricter acceptance processes for prestressed components could improve performance of these elements. By instituting additional quality acceptance measures the corrosion policy would be enhanced without the need to require that additional corrosion protection methods be utilized in the design and construction. This could include the addition of an acceptance specification for concrete surface resistivity to the corrosion policy. Surface resistivity has been linked to several key aspects of concrete performance including chloride permeability and enhanced durability against corrosion. Guidance on how surface resistivity could be incorporated into the policy is presented in Biggers (2019) and Cavalline et al. (2020).

The policy is providing sufficient protection as it currently is for the concrete bridge elements outside of the tidal zone that have been constructed in accordance with it. To enhance its effectiveness and how it is utilized NCDOT could consider further research to evaluate means to optimize, and potentially reduce, corrosion protection provisions for concrete elements outside of the tidal zone. Further research should be conducted to determine if the corrosion protection measures specified for concrete elements subjected to direct exposure to brackish waters within the tidal zone are providing sufficient protection. The addition of including quality standards within the policy regulating the acceptance of precast structural concrete elements could prove beneficial in ensuring that the policy is being implemented properly. Ultimately, the research conducted indicates that the policy is currently providing sufficient protection to the concrete elements observed and no critical recommendations to enhance it were identified as necessary based upon the results of this research project. However, this conclusion is based on only one element of a multi-method study. The results of other methods used to evaluate the policy were not completed at the time of publication, and therefore, additional findings could impact the conclusions set forth in this thesis.

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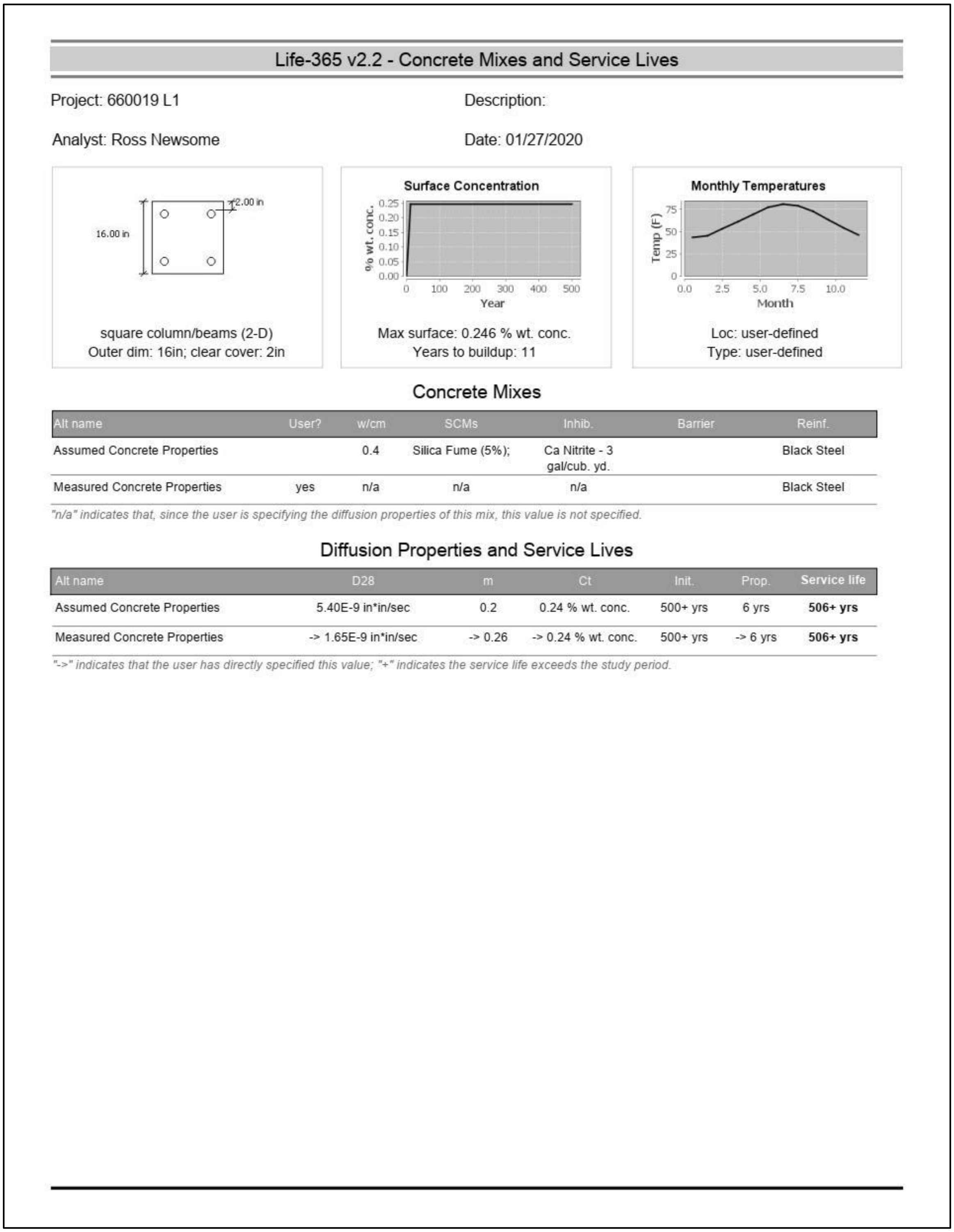
APPENDIX A: CORROSION MODELING SERVICE LIFE REPORTS



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

Project: 660019 L1

Description:

Analyst: Ross Newsome

Date: 01/27/2020

square column/beams (2-D)
Outer dim: 16in; clear cover: 2in

Max surface: 0.246 % wt. conc.
Years to buildup: 11

Loc: user-defined
Type: user-defined

Concrete Mixes

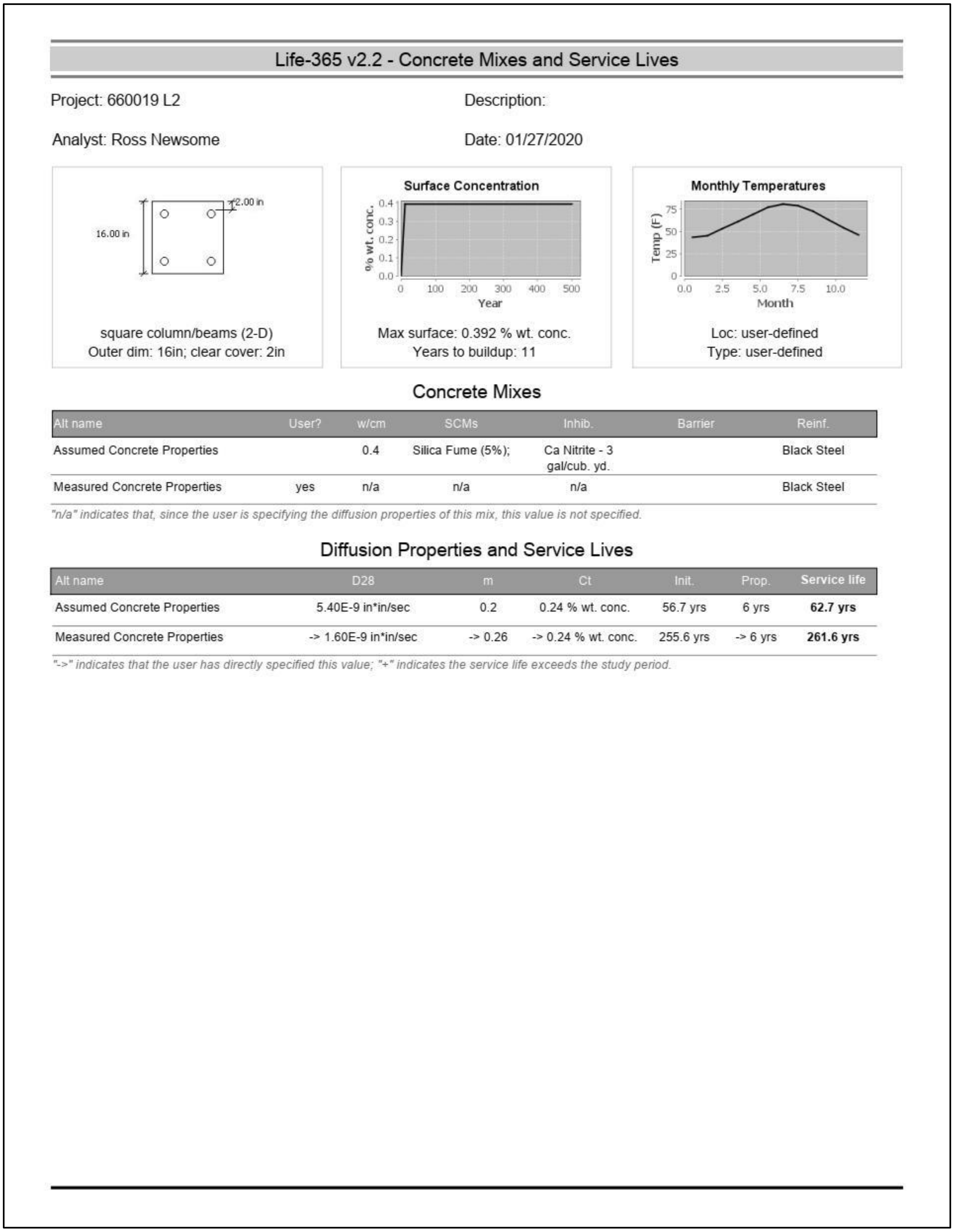
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4	Silica Fume (5%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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	5.40E-9 in*in/sec	0.2	0.24 % wt. conc.	500+ yrs	6 yrs	**506+ yrs**
Measured Concrete Properties	-> 1.65E-9 in*in/sec	-> 0.26	-> 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: 660019 L2

Description:

Analyst: Ross Newsome

Date: 01/27/2020

square column/beams (2-D)
Outer dim: 16in; clear cover: 2in

Max surface: 0.392 % wt. conc.
Years to buildup: 11

Loc: user-defined
Type: user-defined

Concrete Mixes

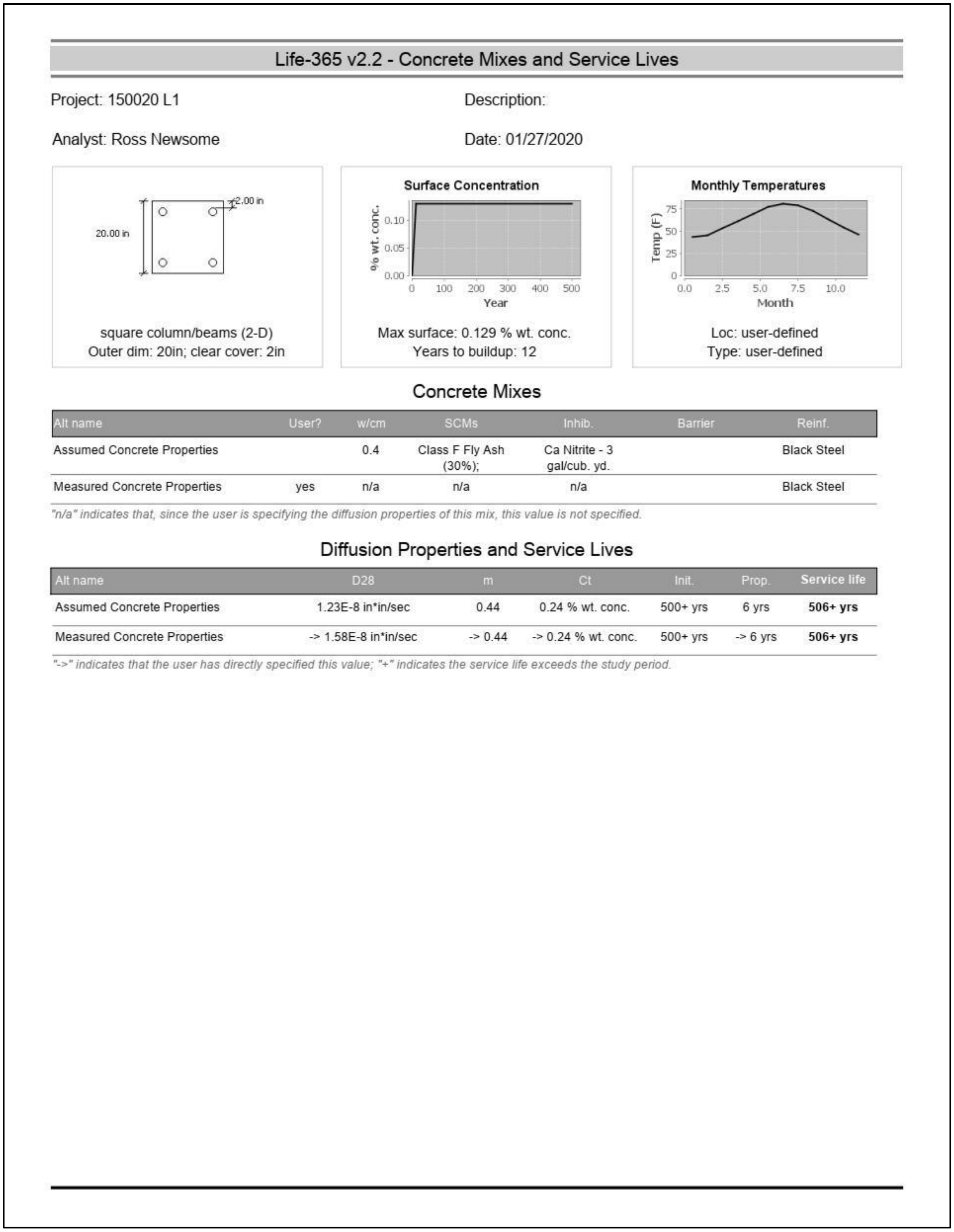
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4	Silica Fume (5%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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	5.40E-9 in*in/sec	0.2	0.24 % wt. conc.	56.7 yrs	6 yrs	**62.7 yrs**
Measured Concrete Properties	-> 1.60E-9 in*in/sec	-> 0.26	-> 0.24 % wt. conc.	255.6 yrs	-> 6 yrs	**261.6 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 L1

Description:

Analyst: Ross Newsome

Date: 01/27/2020

square column/beams (2-D)
Outer dim: 20in; clear cover: 2in

Max surface: 0.129 % wt. conc.
Years to buildup: 12

Loc: user-defined
Type: user-defined

Concrete Mixes

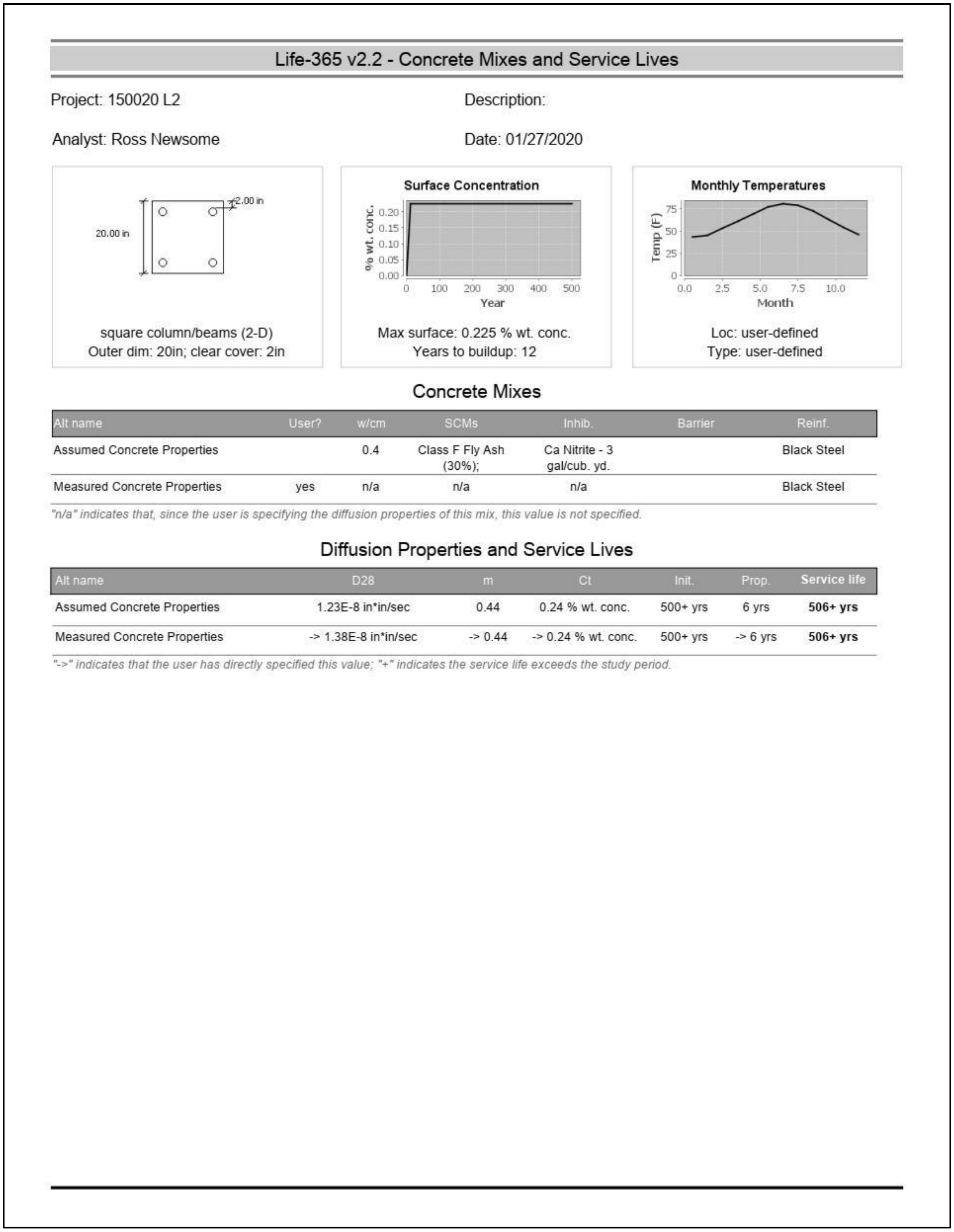
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4	Class F Fly Ash (30%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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	1.23E-8 in*in/sec	0.44	0.24 % wt. conc.	500+ yrs	6 yrs	**506+ yrs**
Measured Concrete Properties	-> 1.58E-8 in*in/sec	-> 0.44	-> 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: 150020 L2

Description:

Analyst: Ross Newsome

Date: 01/27/2020

square column/beams (2-D)
Outer dim: 20in; clear cover: 2in

Max surface: 0.225 % wt. conc.
Years to buildup: 12

Loc: user-defined
Type: user-defined

Concrete Mixes

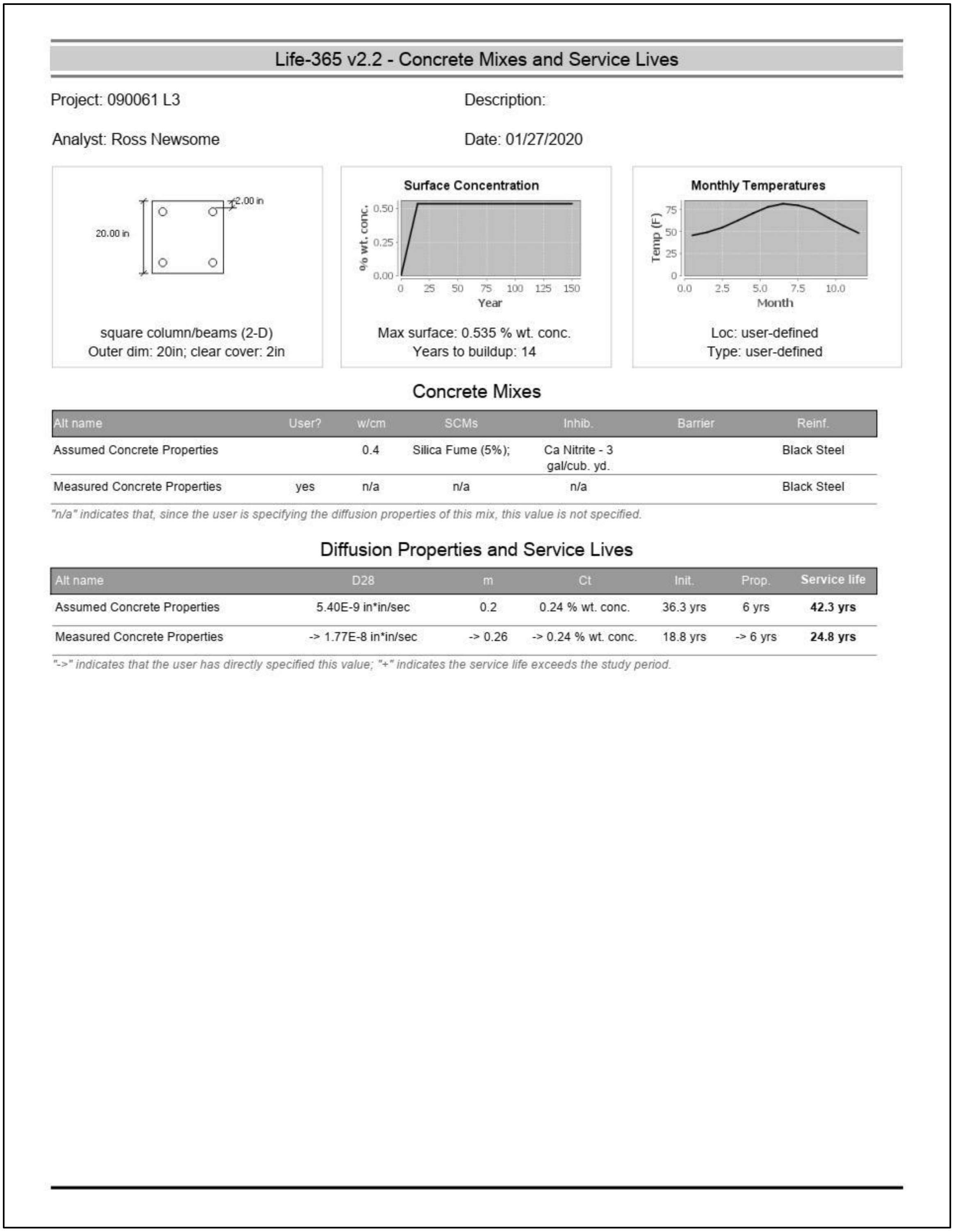
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4	Class F Fly Ash (30%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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	1.23E-8 in*in/sec	0.44	0.24 % wt. conc.	500+ yrs	6 yrs	**506+ yrs**
Measured Concrete Properties	-> 1.38E-8 in*in/sec	-> 0.44	-> 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: 090061 L3

Description:

Analyst: Ross Newsome

Date: 01/27/2020

square column/beams (2-D)
Outer dim: 20in; clear cover: 2in

Max surface: 0.535 % wt. conc.
Years to buildup: 14

Loc: user-defined
Type: user-defined

Concrete Mixes

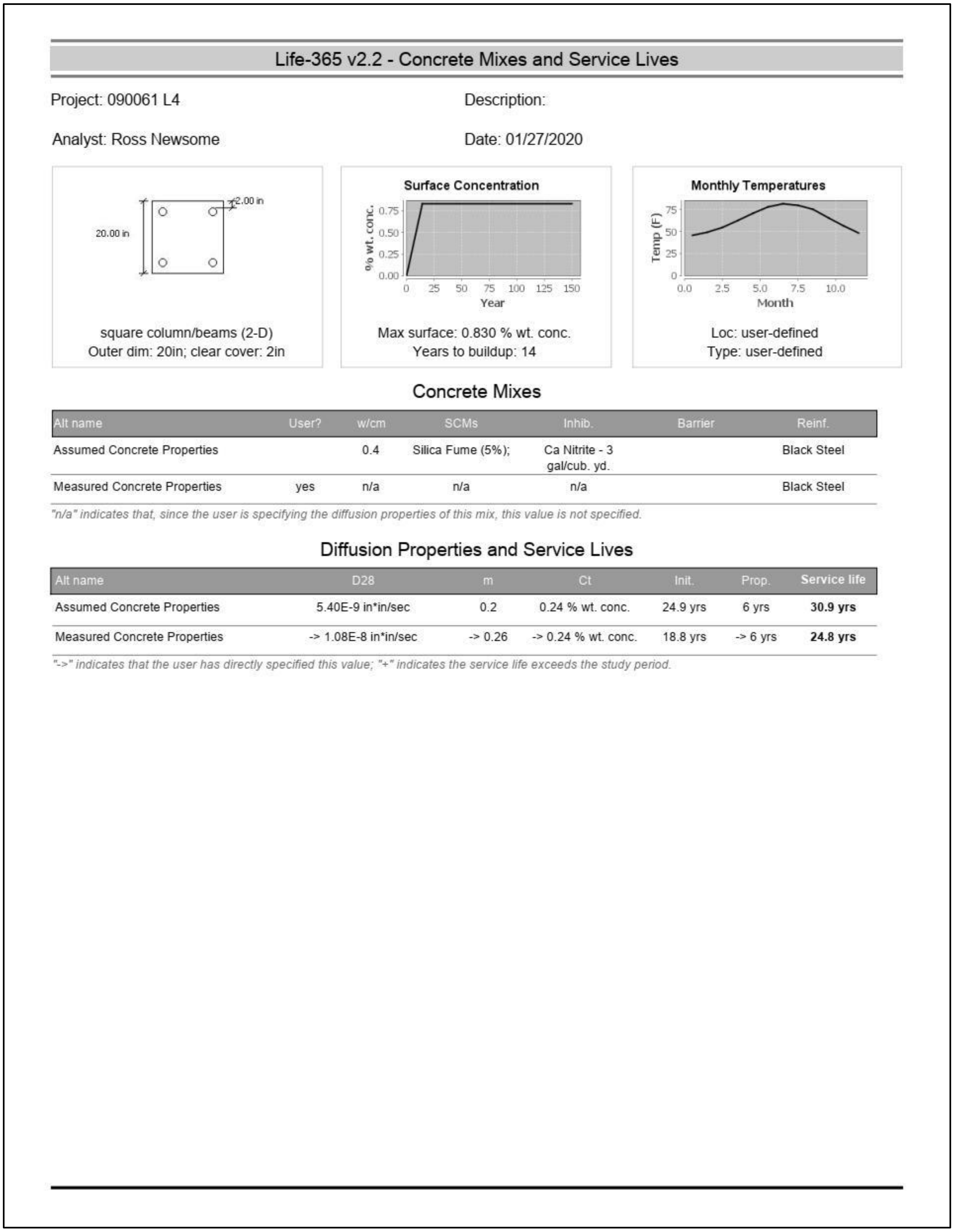
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4	Silica Fume (5%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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	5.40E-9 in*in/sec	0.2	0.24 % wt. conc.	36.3 yrs	6 yrs	**42.3 yrs**
Measured Concrete Properties	-> 1.77E-8 in*in/sec	-> 0.26	-> 0.24 % wt. conc.	18.8 yrs	-> 6 yrs	**24.8 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: 090061 L4

Description:

Analyst: Ross Newsome

Date: 01/27/2020

square column/beams (2-D)
Outer dim: 20in; clear cover: 2in

Max surface: 0.830 % wt. conc.
Years to buildup: 14

Loc: user-defined
Type: user-defined

Concrete Mixes

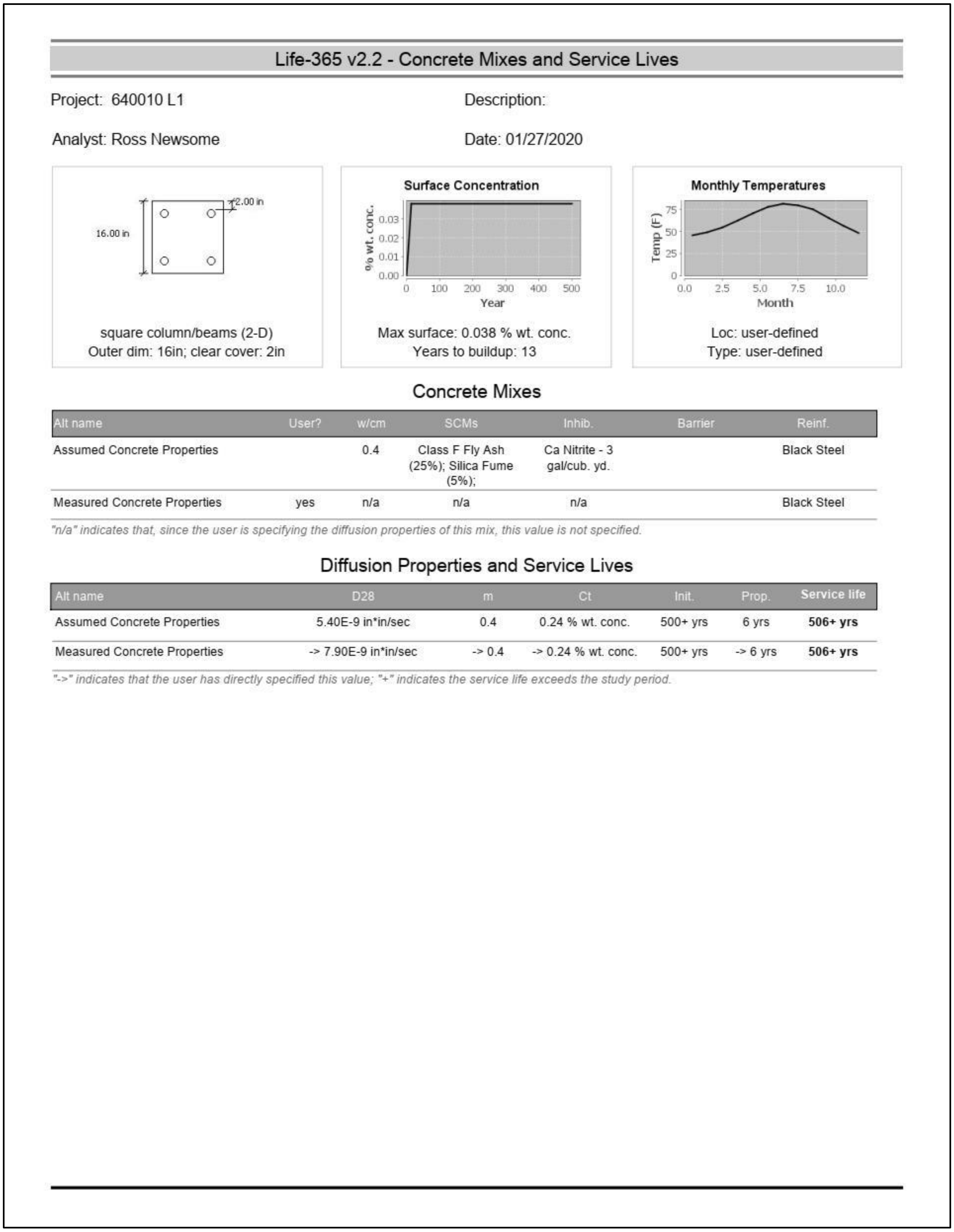
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4	Silica Fume (5%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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	5.40E-9 in*in/sec	0.2	0.24 % wt. conc.	24.9 yrs	6 yrs	**30.9 yrs**
Measured Concrete Properties	-> 1.08E-8 in*in/sec	-> 0.26	-> 0.24 % wt. conc.	18.8 yrs	-> 6 yrs	**24.8 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: 640010 L1

Description:

Analyst: Ross Newsome

Date: 01/27/2020

square column/beams (2-D)
Outer dim: 16in; clear cover: 2in

Max surface: 0.038 % wt. conc.
Years to buildup: 13

Loc: user-defined
Type: user-defined

Concrete Mixes

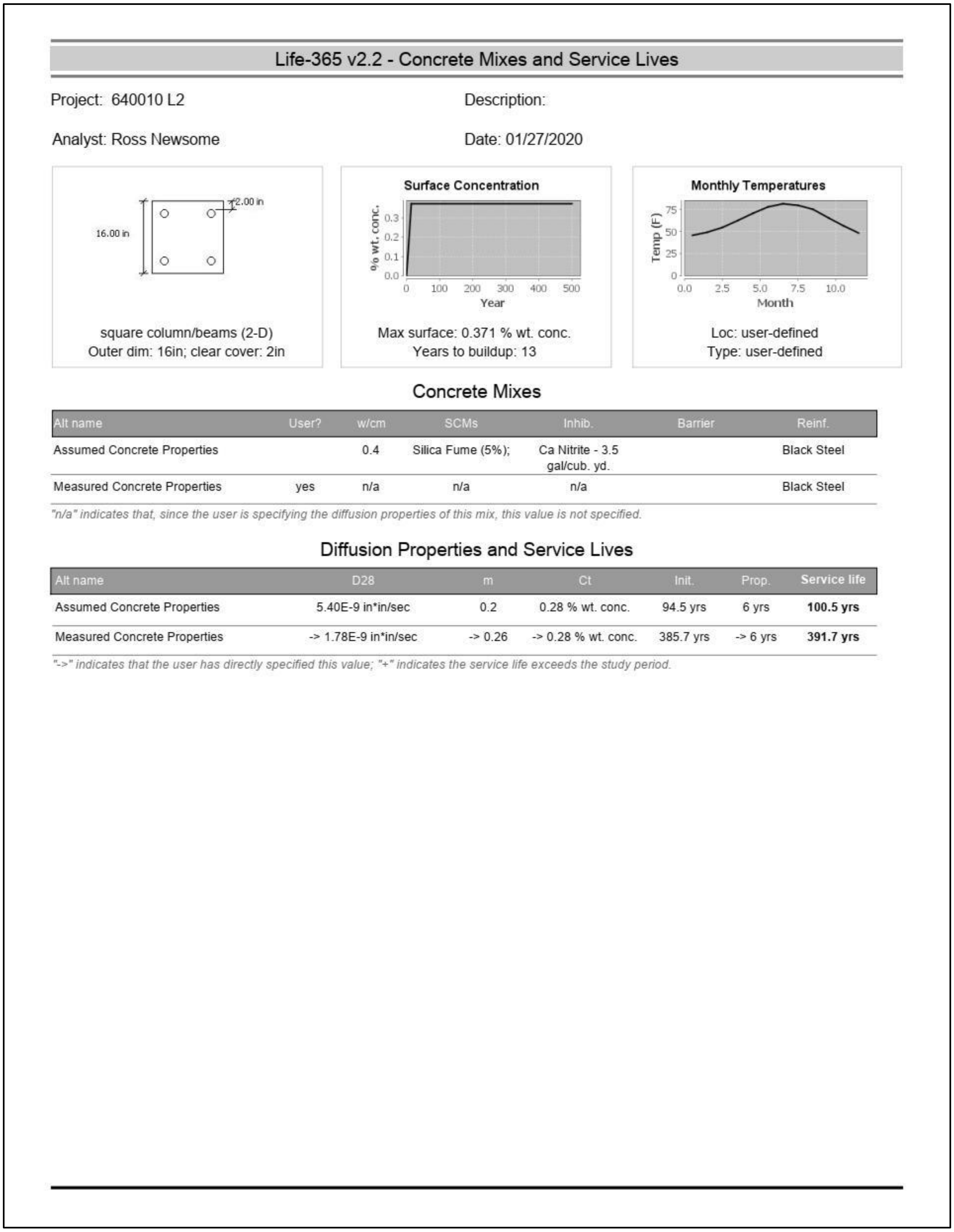
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4	Class F Fly Ash (25%); Silica Fume (5%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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 Concrete Properties	5.40E-9 in*in/sec	0.4	0.24 % wt. conc.	500+ yrs	6 yrs	**506+ yrs**
Measured Concrete Properties	-> 7.90E-9 in*in/sec	-> 0.4	-> 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: 640010 L2

Description:

Analyst: Ross Newsome

Date: 01/27/2020

square column/beams (2-D)
Outer dim: 16in; clear cover: 2in

Max surface: 0.371 % wt. conc.
Years to buildup: 13

Loc: user-defined
Type: user-defined

Concrete Mixes

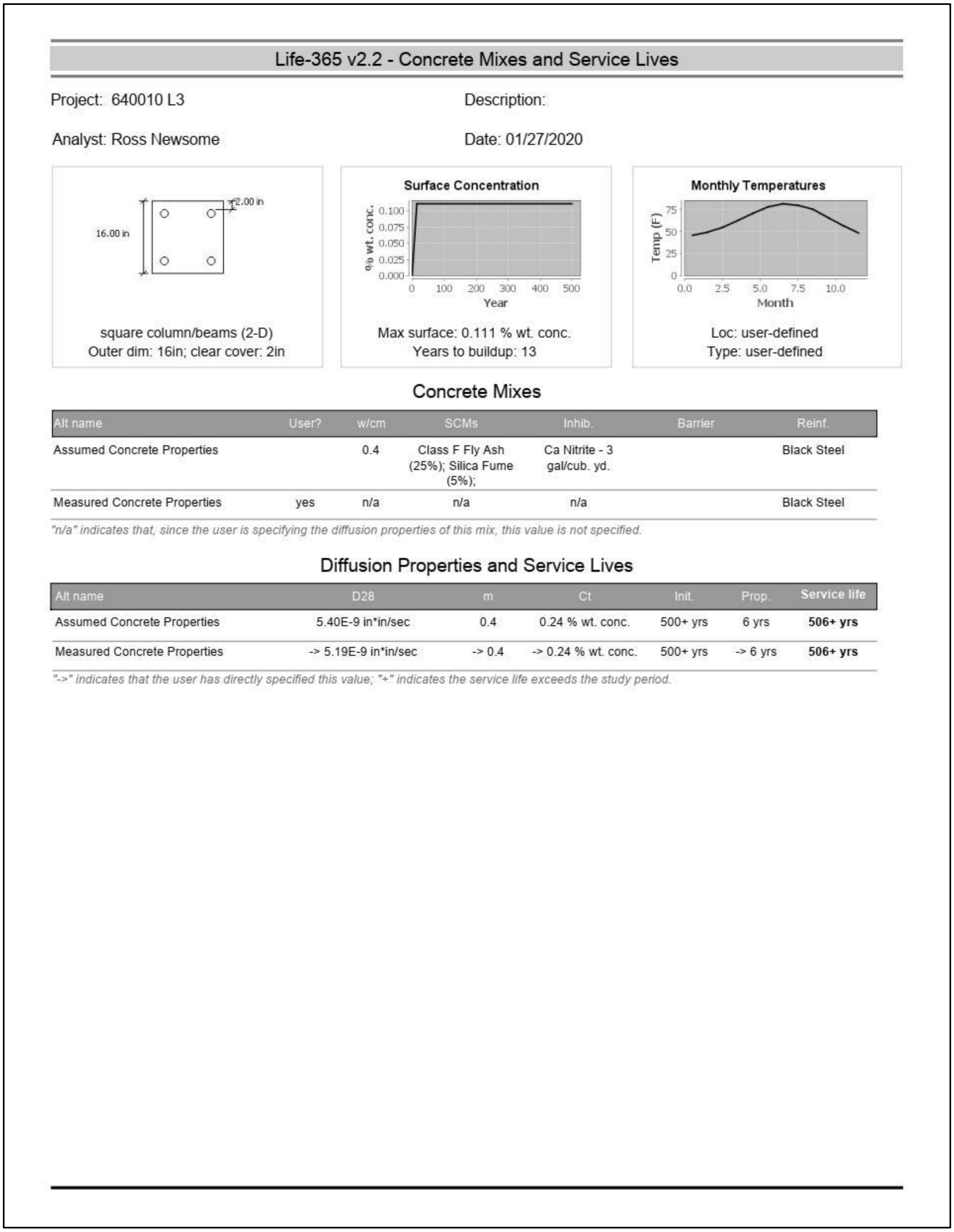
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4	Silica Fume (5%);	Ca Nitrite - 3.5 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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	5.40E-9 in*in/sec	0.2	0.28 % wt. conc.	94.5 yrs	6 yrs	**100.5 yrs**
Measured Concrete Properties	-> 1.78E-9 in*in/sec	-> 0.26	-> 0.28 % wt. conc.	385.7 yrs	-> 6 yrs	**391.7 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 L3

Description:

Analyst: Ross Newsome

Date: 01/27/2020

square column/beams (2-D)
Outer dim: 16in; clear cover: 2in

Max surface: 0.111 % wt. conc.
Years to buildup: 13

Loc: user-defined
Type: user-defined

Concrete Mixes

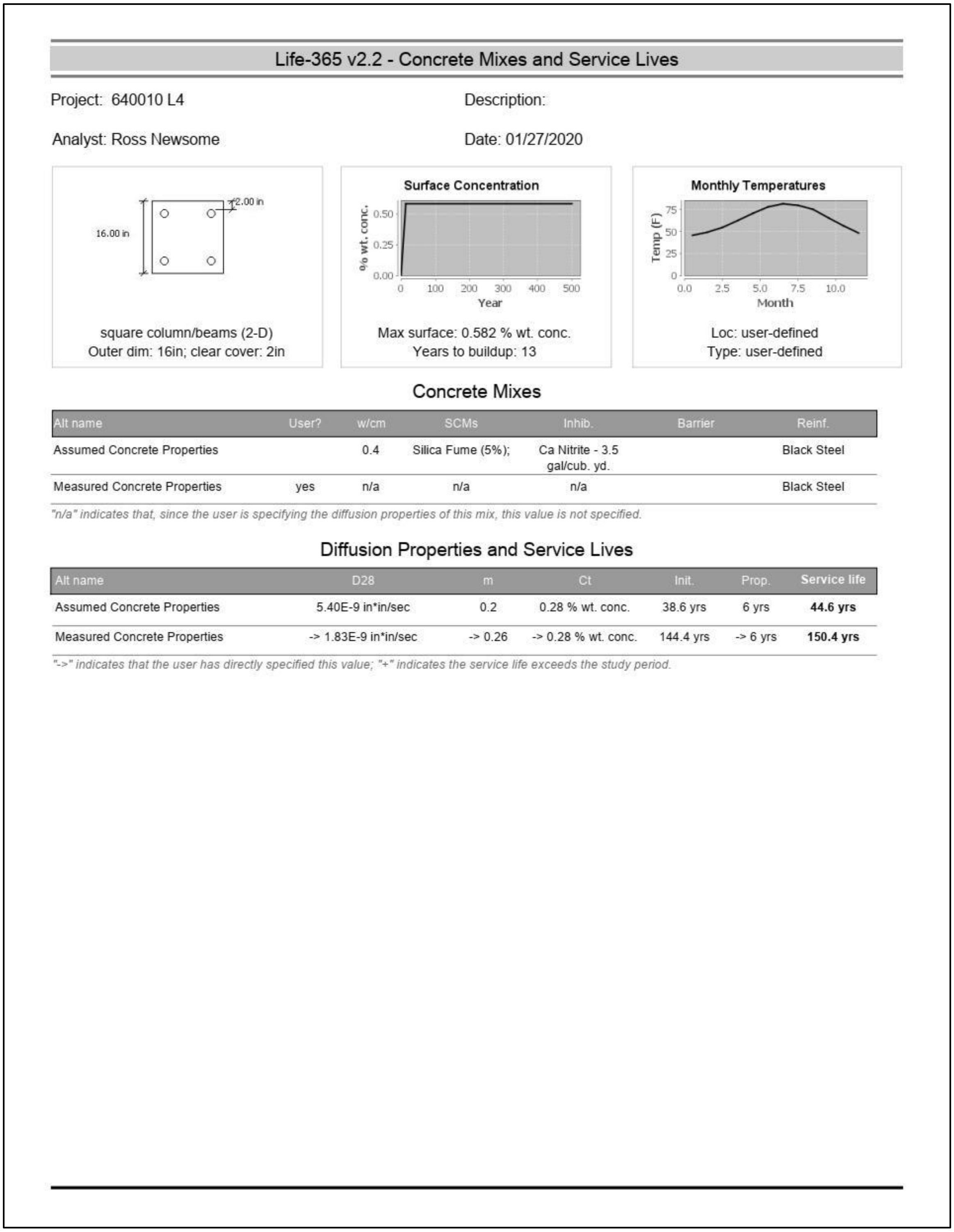
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4	Class F Fly Ash (25%); Silica Fume (5%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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 Concrete Properties	5.40E-9 in*in/sec	0.4	0.24 % wt. conc.	500+ yrs	6 yrs	**506+ yrs**
Measured Concrete Properties	-> 5.19E-9 in*in/sec	-> 0.4	-> 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: 640010 L4

Description:

Analyst: Ross Newsome

Date: 01/27/2020

square column/beams (2-D)
Outer dim: 16in; clear cover: 2in

Max surface: 0.582 % wt. conc.
Years to buildup: 13

Loc: user-defined
Type: user-defined

Concrete Mixes

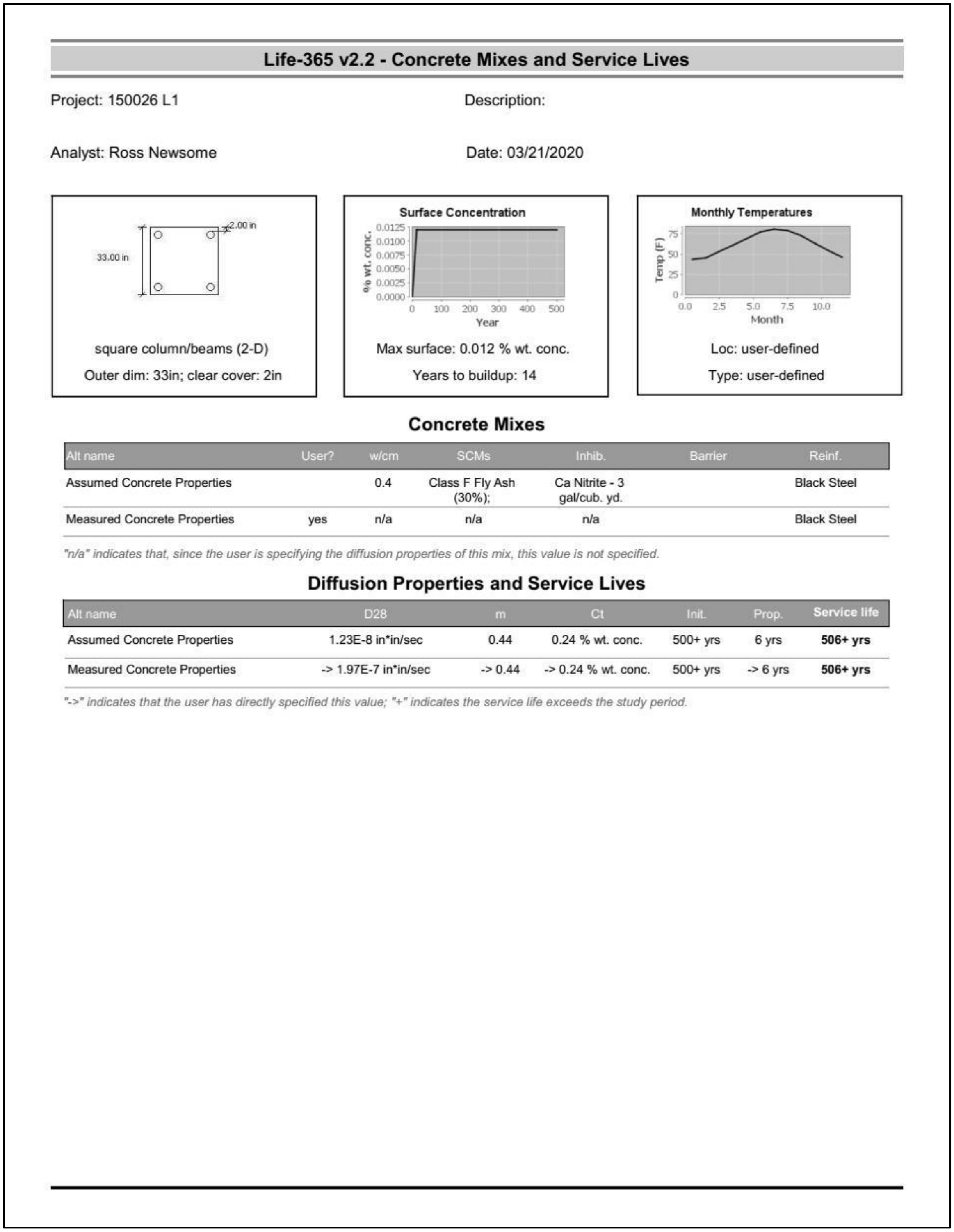
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4	Silica Fume (5%);	Ca Nitrite - 3.5 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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	5.40E-9 in*in/sec	0.2	0.28 % wt. conc.	38.6 yrs	6 yrs	**44.6 yrs**
Measured Concrete Properties	-> 1.83E-9 in*in/sec	-> 0.26	-> 0.28 % wt. conc.	144.4 yrs	-> 6 yrs	**150.4 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: 150026 L1

Description:

Analyst: Ross Newsome

Date: 03/21/2020

square column/beams (2-D)

Outer dim: 33in; clear cover: 2in

Max surface: 0.012 % wt. conc.

Years to buildup: 14

Loc: user-defined

Type: user-defined

Concrete Mixes

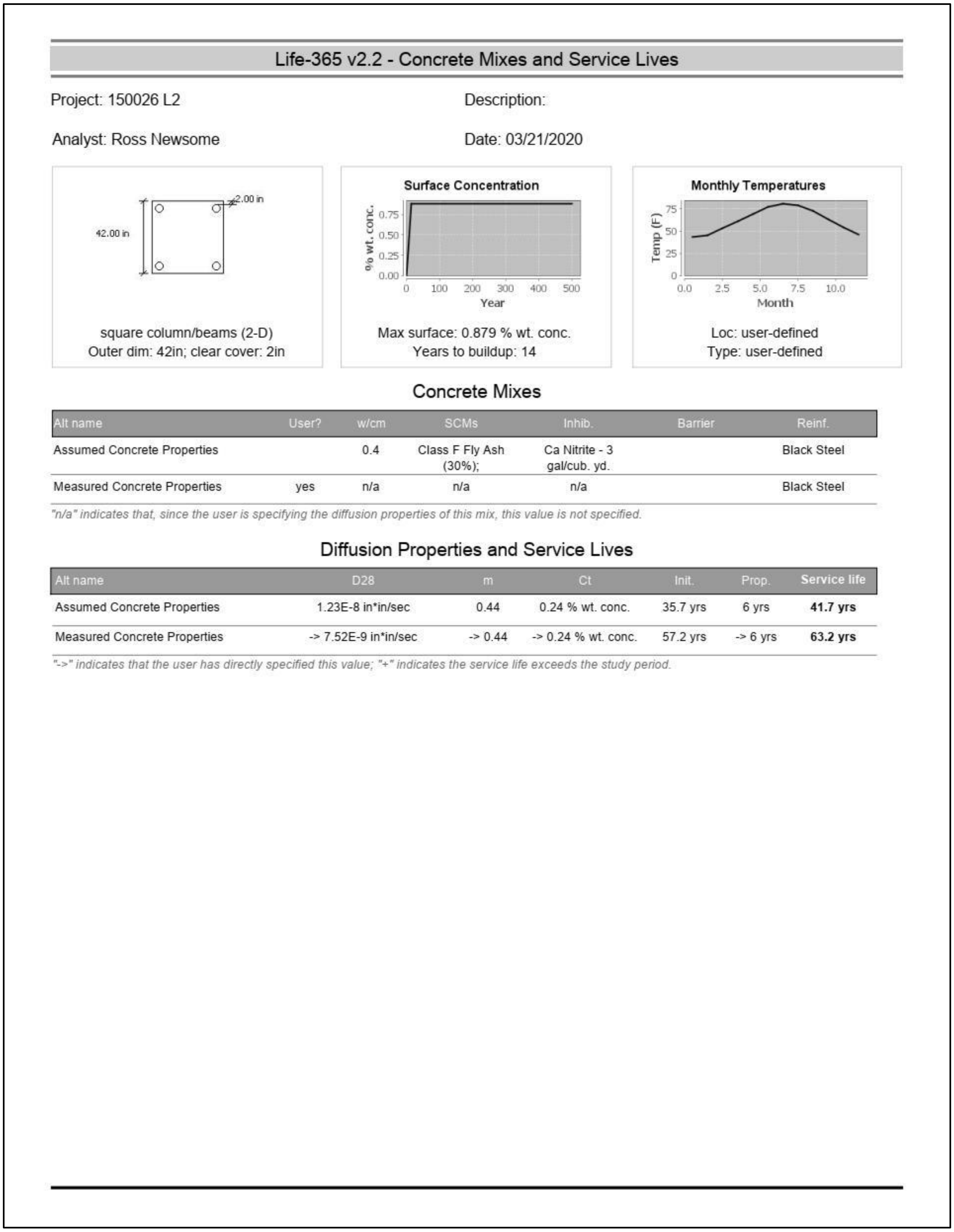
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4	Class F Fly Ash (30%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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	1.23E-8 in*in/sec	0.44	0.24 % wt. conc.	500+ yrs	6 yrs	**506+ yrs**
Measured Concrete Properties	-> 1.97E-7 in*in/sec	-> 0.44	-> 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: 150026 L2

Description:

Analyst: Ross Newsome

Date: 03/21/2020

square column/beams (2-D)
Outer dim: 42in; clear cover: 2in

Max surface: 0.879 % wt. conc.
Years to buildup: 14

Loc: user-defined
Type: user-defined

Concrete Mixes

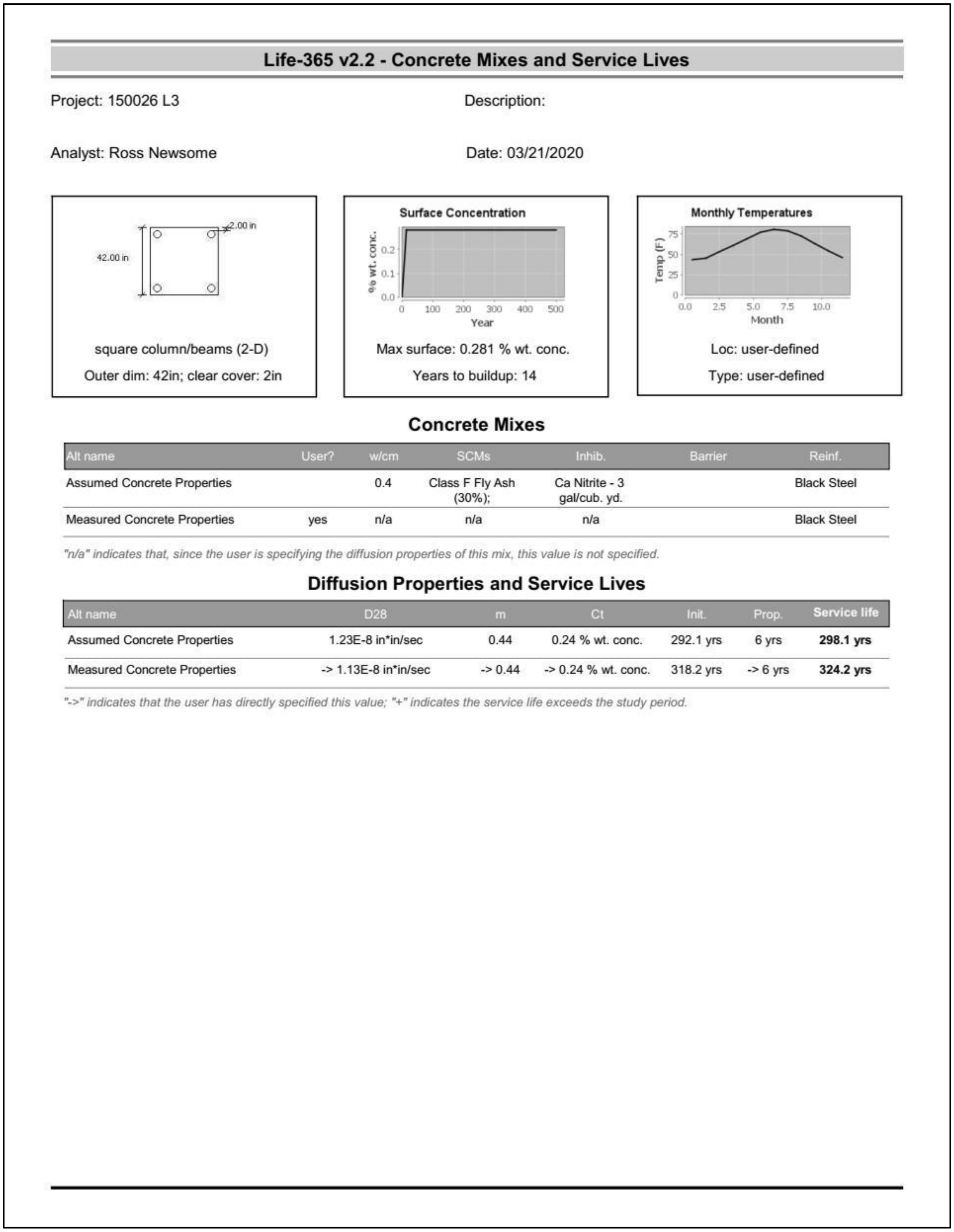
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4	Class F Fly Ash (30%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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	1.23E-8 in*in/sec	0.44	0.24 % wt. conc.	35.7 yrs	6 yrs	**41.7 yrs**
Measured Concrete Properties	-> 7.52E-9 in*in/sec	-> 0.44	-> 0.24 % wt. conc.	57.2 yrs	-> 6 yrs	**63.2 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: 150026 L3

Description:

Analyst: Ross Newsome

Date: 03/21/2020

square column/beams (2-D)

Outer dim: 42in; clear cover: 2in

Max surface: 0.281 % wt. conc.

Years to buildup: 14

Loc: user-defined

Type: user-defined

Concrete Mixes

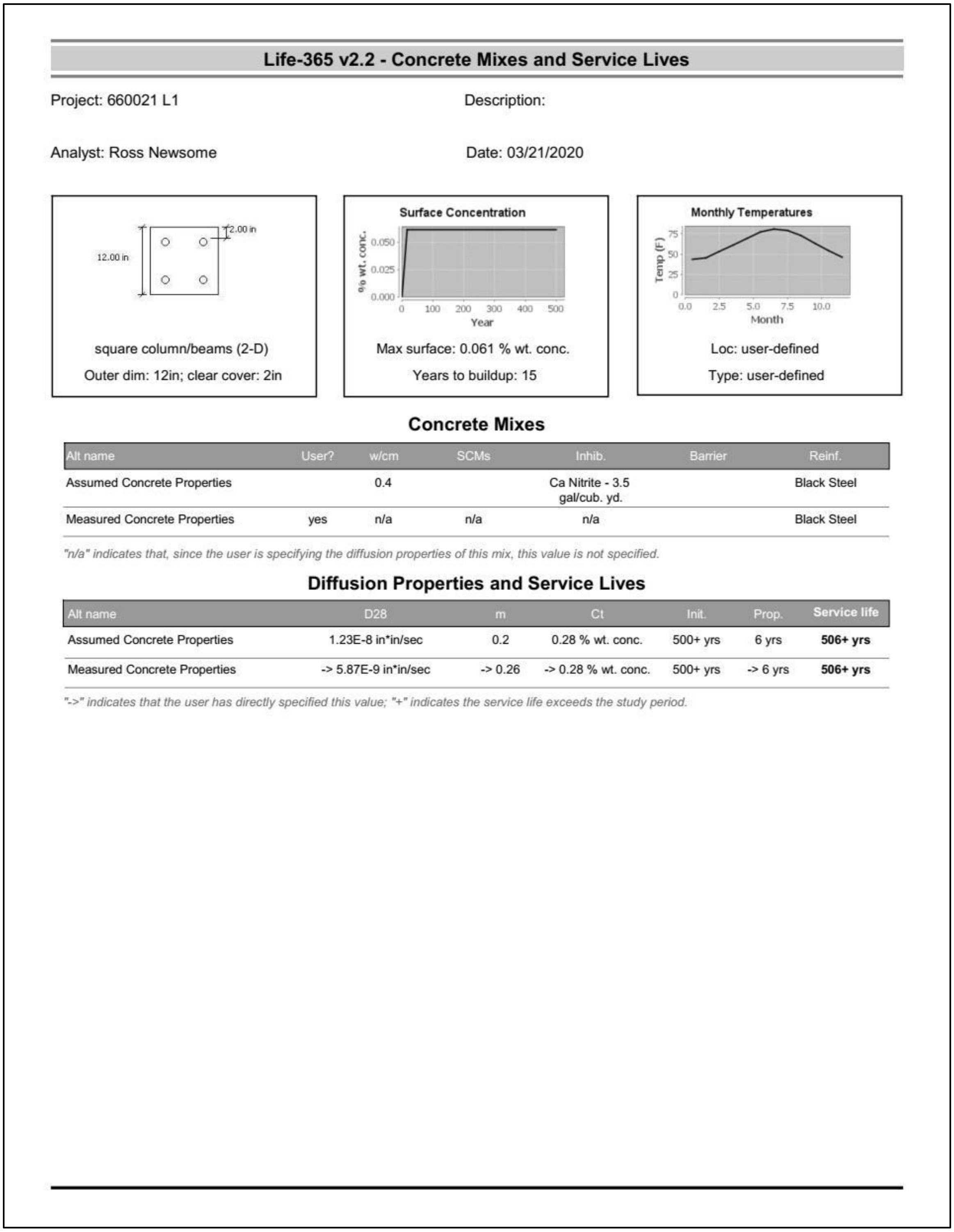
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4	Class F Fly Ash (30%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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	1.23E-8 in*in/sec	0.44	0.24 % wt. conc.	292.1 yrs	6 yrs	**298.1 yrs**
Measured Concrete Properties	-> 1.13E-8 in*in/sec	-> 0.44	-> 0.24 % wt. conc.	318.2 yrs	-> 6 yrs	**324.2 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: 660021 L1

Description:

Analyst: Ross Newsome

Date: 03/21/2020

square column/beams (2-D)

Outer dim: 12in; clear cover: 2in

Max surface: 0.061 % wt. conc.

Years to buildup: 15

Loc: user-defined

Type: user-defined

Concrete Mixes

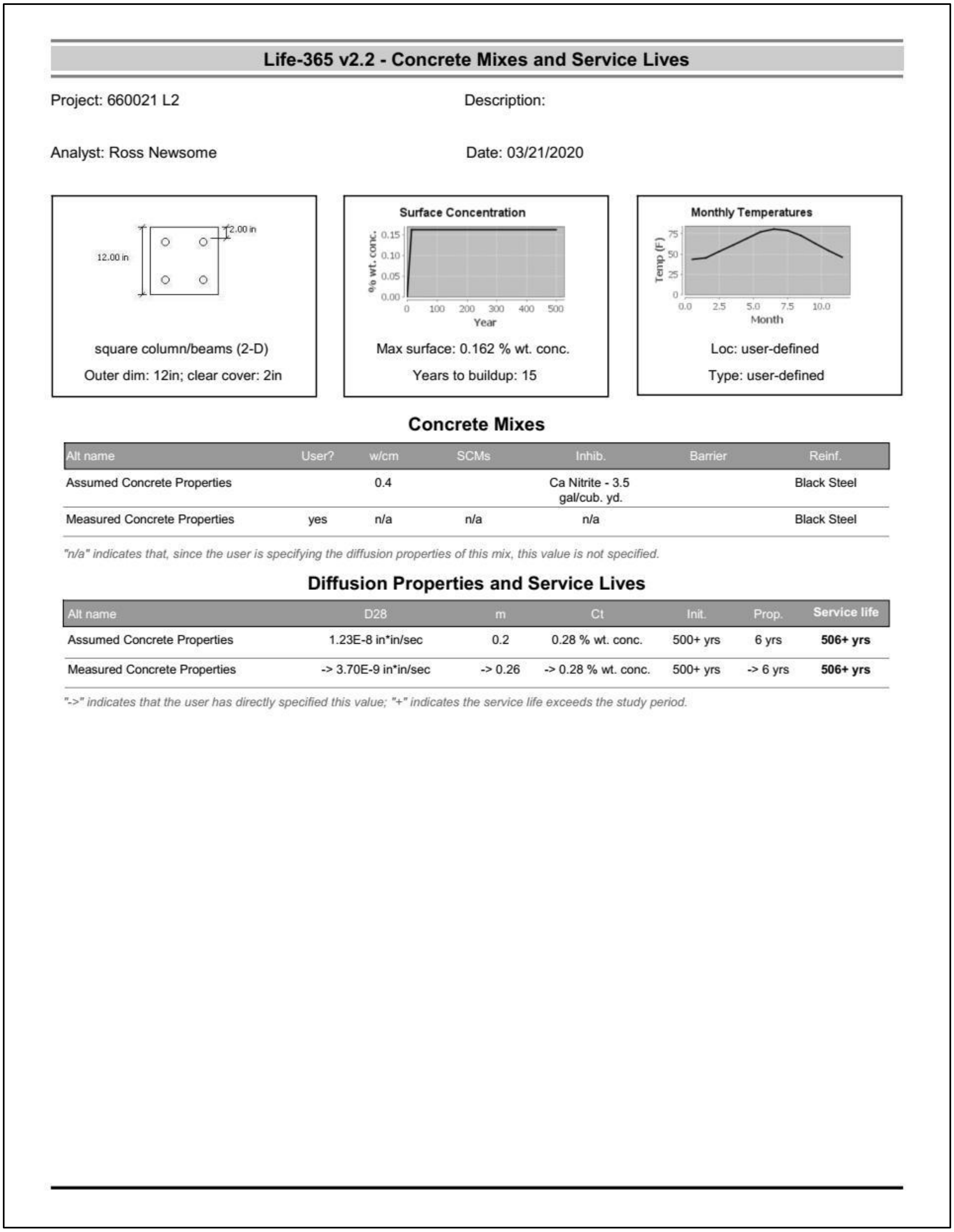
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4		Ca Nitrite - 3.5 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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	1.23E-8 in*in/sec	0.2	0.28 % wt. conc.	500+ yrs	6 yrs	**506+ yrs**
Measured Concrete Properties	-> 5.87E-9 in*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: 660021 L2

Description:

Analyst: Ross Newsome

Date: 03/21/2020

square column/beams (2-D)

Outer dim: 12in; clear cover: 2in

Max surface: 0.162 % wt. conc.

Years to buildup: 15

Loc: user-defined

Type: user-defined

Concrete Mixes

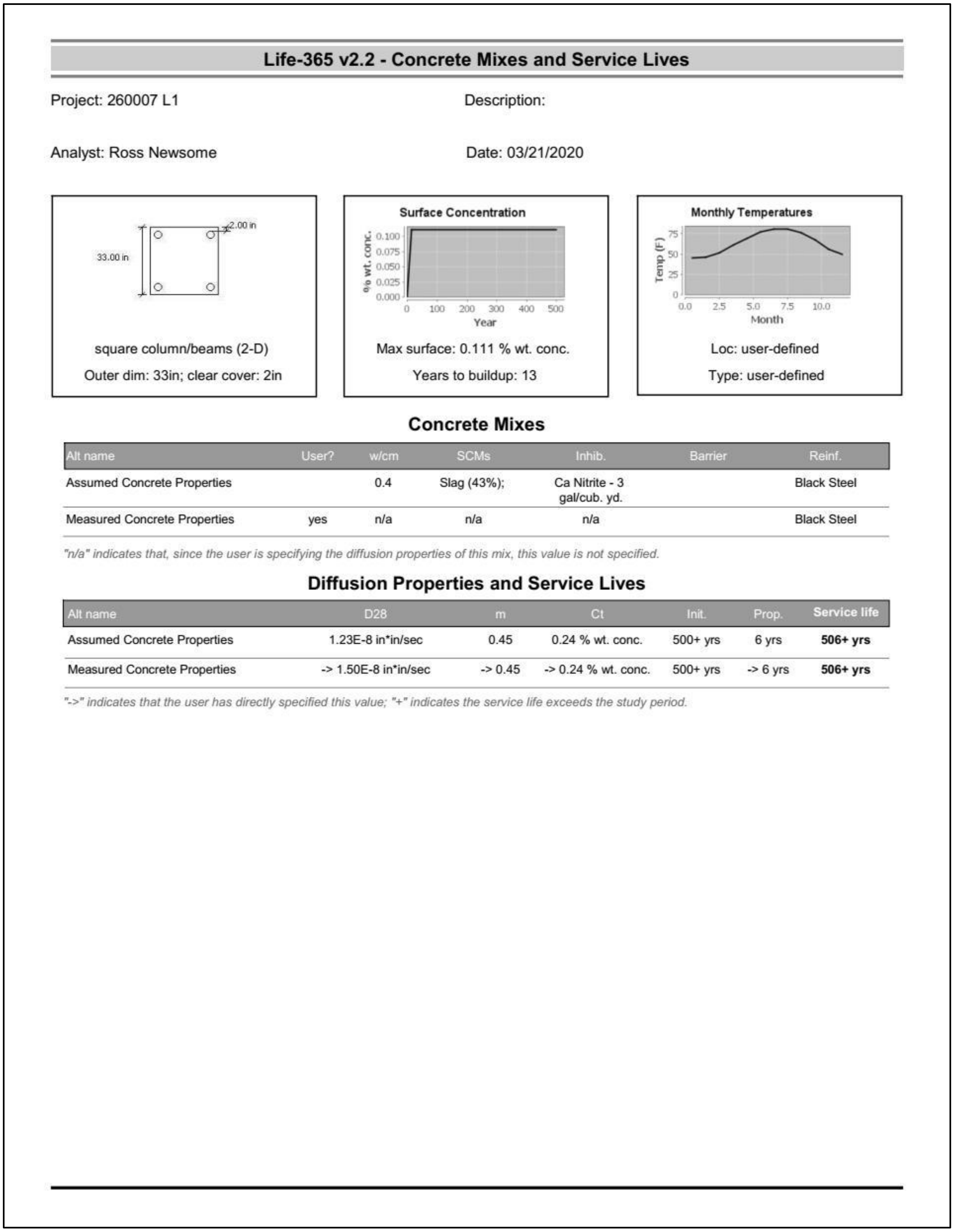
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4		Ca Nitrite - 3.5 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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	1.23E-8 in*in/sec	0.2	0.28 % wt. conc.	500+ yrs	6 yrs	**506+ yrs**
Measured Concrete Properties	-> 3.70E-9 in*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: 260007 L1

Description:

Analyst: Ross Newsome

Date: 03/21/2020

square column/beams (2-D)

Outer dim: 33in; clear cover: 2in

Max surface: 0.111 % wt. conc.

Years to buildup: 13

Loc: user-defined

Type: user-defined

Concrete Mixes

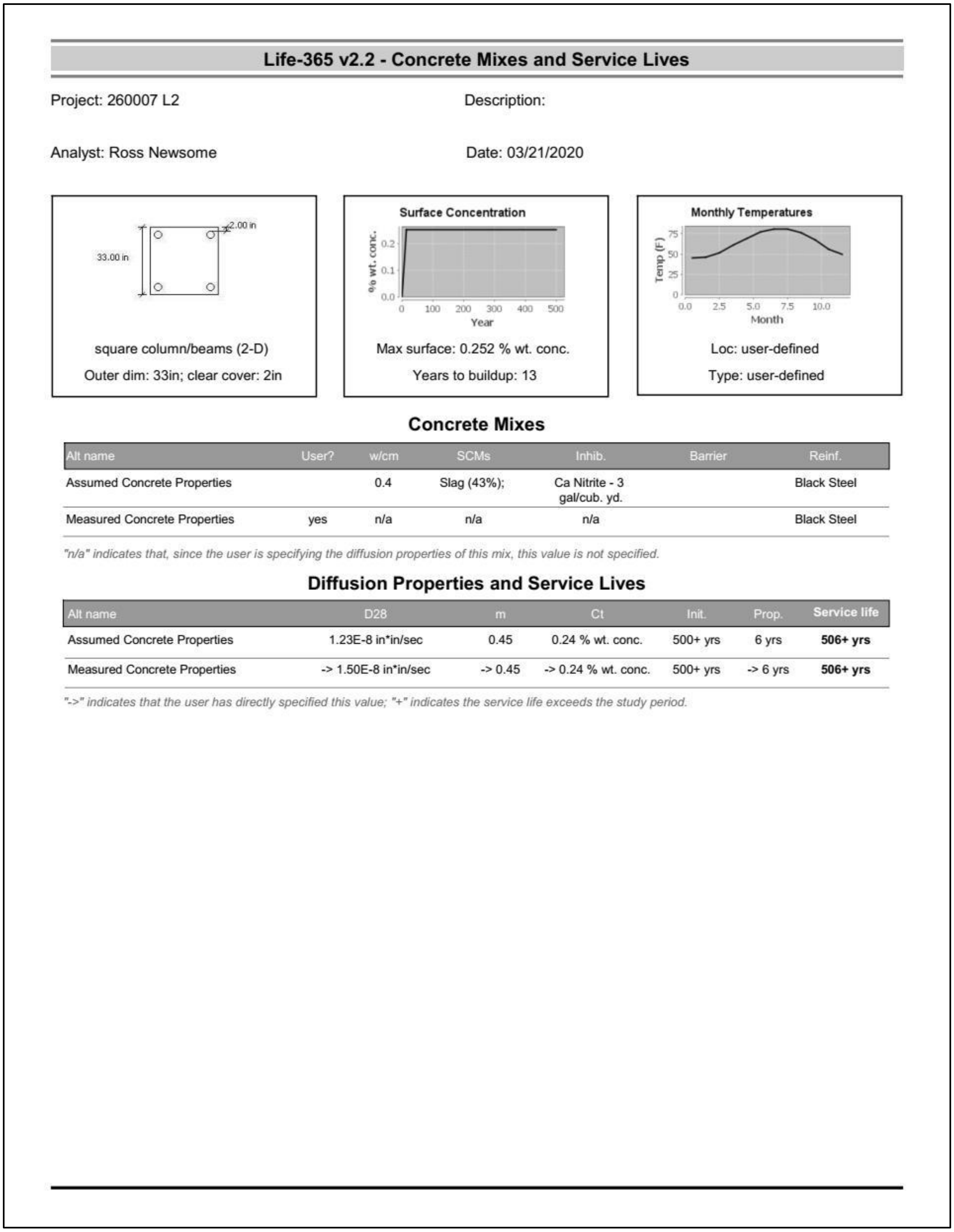
Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4	Slag (43%);	Ca Nitrite - 3 gal/cub. yd.		Black Steel
Measured Concrete Properties	yes	n/a	n/a	n/a		Black Steel

"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	1.23E-8 in*in/sec	0.45	0.24 % wt. conc.	500+ yrs	6 yrs	**506+ yrs**
Measured Concrete Properties	-> 1.50E-8 in*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:

Analyst: Ross Newsome

Date: 03/21/2020

square column/beams (2-D)

Outer dim: 33in; clear cover: 2in

Max surface: 0.252 % wt. conc.

Years to buildup: 13

Loc: user-defined

Type: user-defined

Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Assumed Concrete Properties		0.4	Slag (43%);	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	1.23E-8 in*in/sec	0.45	0.24 % wt. conc.	500+ yrs	6 yrs	**506+ yrs**
Measured Concrete Properties	-> 1.50E-8 in*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.

APPENDIX B: SUPPLIMENTAL INFORMATION FOR CHAPTER 4

Table B1: RCT results for bridges 660019 and 150020

Structure Number	**Location**	**Sample Number**	**Depth (in)**	**Voltage (mV)**	**% Cl by conc. wt.**	**Avg. % Cl by conc. wt.**	**% Cl (PCY)**	**Avg. % Cl (PCY)**
660019	L1	1	0	11.4	0.290	0.291	11.049	11.096
		2		11.2	0.292		11.143	
		1	1	52.1	0.052	0.051	2.000	1.963
		2		53.0	0.050		1.925	
		1	2	122.6	0.003	0.003	0.104	0.109
		2		120.1	0.003		0.115	
		1	3	125.0	0.002	0.003	0.094	0.097
		2		123.1	0.003		0.101	
	L2	1	0	6.5	0.356	0.341	13.574	13.001
		2		8.6	0.326		12.428	
		1	1	42.3	0.079	0.081	3.018	3.096
		2		41.1	0.083		3.174	
		1	2	111.3	0.004	0.004	0.166	0.167
		2		111.0	0.004		0.168	
		1	3	107.5	0.005	0.004	0.195	0.171
		2		114.2	0.004		0.147	
150020	L1	1	0	11.7	0.286	0.282	10.911	10.775
		2		12.3	0.279		10.639	
		1	1	55.2	0.046	0.046	1.756	1.767
		2		54.9	0.047		1.778	
		1	2	99.0	0.007	0.007	0.279	0.277
		2		99.3	0.007		0.275	
		1	3	102.7	0.006	0.006	0.239	0.237
		2		103.0	0.006		0.236	
		1	4	115.5	0.004	0.004	0.139	0.138
		2		116.0	0.004		0.137	
		1	5	120.1	0.003	0.003	0.115	0.119
		2		118.4	0.003		0.123	
	L2	1	0	9.9	0.308	0.310	11.768	11.817
		2		9.7	0.311		11.867	
		1	1	44.4	0.072	0.073	2.763	2.781
		2		44.1	0.073		2.798	
		1	2	90.5	0.010	0.010	0.399	0.393
		2		91.2	0.010		0.387	
		1	3	113.3	0.004	0.004	0.153	0.165
		2		109.7	0.005		0.178	
		1	4	110.5	0.005	0.004	0.172	0.169
		2		111.5	0.004		0.165	
		1	5	114.0	0.004	0.004	0.149	0.150
		2		113.4	0.004		0.152	

Table B2: RCT results for bridges 660091 and 090061

Structure Number	Location	Sample Number	Depth (in)	Voltage (mV)	% Cl by concrete weight	Avg. % Cl by concrete weight	% Cl (PCY)	Avg. % Cl (PCY)
660091	L1	1	1	96.5	0.008	0.008	0.315	0.316
		2		96.4	0.008		0.316	
		1	2	102.2	0.007	0.007	0.249	0.248
		2		102.4	0.006		0.247	
		1	3	102.7	0.006	0.007	0.244	0.252
		2		101.3	0.007		0.259	
	L2	1	1	90.2	0.011	0.011	0.408	0.436
		2		87.1	0.012		0.463	
		1	2	92.4	0.010	0.009	0.373	0.326
		2		99.5	0.007		0.279	
		1	3	90.3	0.011	0.011	0.406	0.408
		2		90.1	0.011		0.410	
090061	L1	1	0	2.2	0.426	0.417	16.261	15.894
		2		3.3	0.407		15.527	
		1	1	3.7	0.400	0.405	15.268	15.463
		2		3.1	0.410		15.658	
		1	2	14.4	0.255	0.256	9.741	9.762
		2		14.3	0.256		9.782	
		1	3	24.7	0.166	0.164	6.320	6.268
		2		25.1	0.163		6.215	
		1	4	36.0	0.103	0.102	3.932	3.907
		2		36.3	0.102		3.883	
		1	5	46.5	0.066	0.065	2.530	2.498
		2		47.1	0.065		2.467	
	L2	1	0	-11.3	0.751	0.759	28.668	28.972
		2		-11.8	0.767		29.276	
		1	1	-4.9	0.574	0.567	21.911	21.638
		2		-4.3	0.560		21.365	
		1	2	10.4	0.302	0.300	11.523	11.451
		2		10.7	0.298		11.379	
		1	3	22.1	0.185	0.181	7.050	6.905
		2		23.1	0.177		6.760	
		1	4	41.1	0.083	0.085	3.174	3.249
		2		40.0	0.087		3.324	

Table B3: RCT results for bridge 640010

Structure Number	**Location**	**Sample Number**	**Depth (in)**	**Voltage (mV)**	**% Cl by concrete weight**	**Avg. % Cl by concrete weight**	**% Cl (PCY)**	**Avg. % Cl (PCY)**
640010	L1	1	1	77.4	0.018	0.018	0.691	0.692
		2		77.3	0.018		0.694	
		1	2	116.5	0.004	0.004	0.134	0.139
		2		114.6	0.004		0.145	
		1	3	114.2	0.004	0.004	0.147	0.150
		2		113.3	0.004		0.153	
		1	4	114.5	0.004	0.004	0.145	0.145
		2		114.8	0.004		0.144	
		1	5	118.8	0.003	0.003	0.121	0.132
		2		115.1	0.004		0.142	
	L2	1	0	11.1	0.293	0.295	11.189	11.237
		2		10.9	0.296		11.284	
		1	1	36.2	0.102	0.104	3.899	3.957
		2		35.5	0.105		4.016	
		1	2	93.4	0.009	0.010	0.353	0.391
		2		88.7	0.011		0.430	
		1	3	89.6	0.011	0.011	0.414	0.408
		2		90.3	0.011		0.402	
		1	4	84.7	0.013	0.013	0.509	0.504
		2		85.1	0.013		0.500	
		1	5	87.0	0.012	0.012	0.462	0.468
		2		86.4	0.012		0.473	
	L3	1	0	33.5	0.114	0.115	4.367	4.395
		2		33.2	0.116		4.423	
		1	1	57.4	0.042	0.041	1.601	1.574
		2		58.2	0.041		1.548	
		1	2	98.2	0.008	0.007	0.288	0.281
		2		99.4	0.007		0.274	
		1	3	115.5	0.004	0.004	0.139	0.142
		2		114.8	0.004		0.144	
		1	4	118.5	0.003	0.003	0.123	0.125
		2		117.7	0.003		0.127	
		1	5	110.9	0.004	0.004	0.169	0.157
		2		114.5	0.004		0.145	
	L4	1	0	-5.7	0.594	0.588	22.659	22.424
		2		-5.2	0.582		22.188	
		1	1	24.4	0.168	0.166	6.400	6.347
		2		24.8	0.165		6.294	
		1	2	79.4	0.017	0.017	0.635	0.648
		2		78.5	0.017		0.660	
		1	3	76.2	0.019	0.019	0.727	0.738
		2		75.5	0.020		0.748	
		1	4	72.4	0.022	0.023	0.852	0.860
		2		72.0	0.023		0.867	
		1	5	73.3	0.022	0.021	0.821	0.793
		2		75.000	0.020		0.764	

Table B4: RCT results for bridge 150026

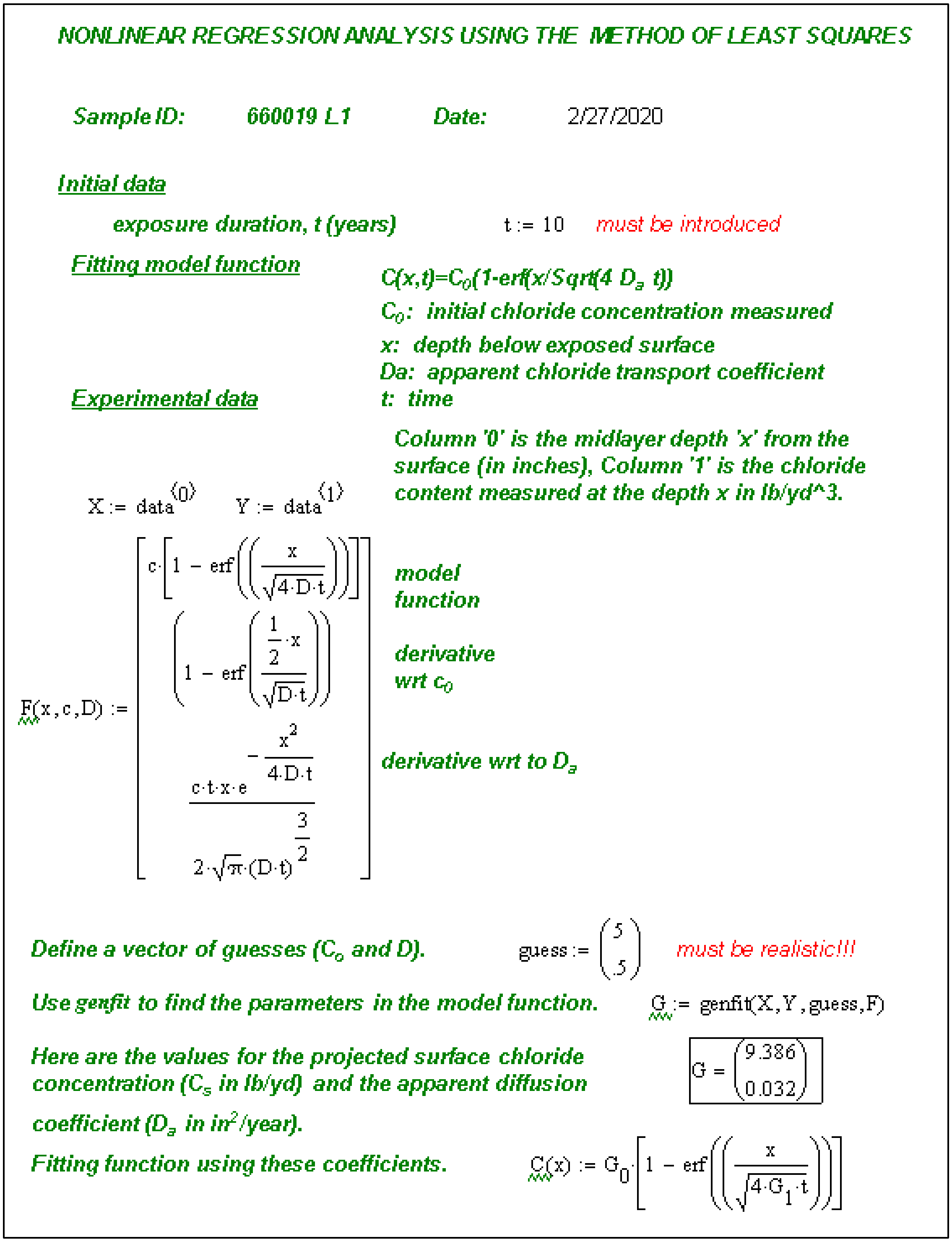
Structure Number	Location	Sample Number	Depth (in)	Voltage (mV)	% Cl by concrete weight	Avg. % Cl by concrete weight	% Cl (PCY)	Avg. % Cl (PCY)
150026	L1	1	0	61.1	0.036	0.037	1.370	1.406
		2		59.9	0.038		1.441	
		1	1	89.5	0.011	0.011	0.416	0.415
		2		89.6	0.011		0.414	
		1	2	104.4	0.006	0.006	0.222	0.222
		2		104.4	0.006		0.222	
		1	3	113.9	0.004	0.004	0.149	0.155
		2		112.2	0.004		0.160	
		1	4	111.8	0.004	0.004	0.163	0.161
		2		112.4	0.004		0.159	
		1	5	113.4	0.004	0.004	0.152	0.153
		2		113.3	0.004		0.153	
	L2	1	0	-0.5	0.477	0.473	18.214	18.062
		2		-0.1	0.469		17.910	
		1	1	22.2	0.184	0.183	7.020	6.976
		2		22.5	0.182		6.932	
		1	2	90.9	0.010	0.010	0.392	0.390
		2		91.1	0.010		0.389	
		1	3	108.7	0.005	0.005	0.186	0.181
		2		110.0	0.005		0.176	
		1	4	109.8	0.005	0.004	0.177	0.165
		2		113.4	0.004		0.152	
		1	5	115.5	0.004	0.004	0.139	0.142
		2		114.7	0.004		0.144	
	L3	1	0	6.6	0.354	0.357	13.517	13.603
		2		6.3	0.359		13.689	
		1	1	40.6	0.085	0.085	3.241	3.241
		2		40.6	0.085		3.241	
		1	2	90.5	0.010	0.011	0.399	0.401
		2		90.2	0.011		0.404	
		1	3	113.8	0.004	0.004	0.150	0.153
		2		112.8	0.004		0.156	
		1	4	110.0	0.005	0.004	0.176	0.168
		2		112.1	0.004		0.161	
		1	5	113.3	0.004	0.004	0.153	0.151
		2		114.0	0.004		0.149	

Table B5: RCT results for bridge 660021

Structure Number	**Location**	**Sample Number**	**Depth (in)**	**Voltage (mV)**	**% Cl by concrete weight**	**Avg. % Cl by concrete weight**	**% Cl (PCY)**	**Avg. % Cl (PCY)**
660021	L1	1	0	33.2	0.116	0.115	4.423	4.404
		2		33.4	0.115		4.386	
		1	1	72.1	0.023	0.023	0.863	0.887
		2		70.8	0.024		0.912	
		1	2	113.0	0.004	0.004	0.155	0.156
		2		112.8	0.004		0.156	
		1	3	121.4	0.003	0.003	0.109	0.109
		2		121.4	0.003		0.109	
		1	4	122.0	0.003	0.003	0.106	0.105
		2		122.3	0.003		0.105	
		1	5	123.1	0.003	0.003	0.101	0.102
		2		123.0	0.003		0.102	
	L2	1	0	21.0	0.194	0.183	7.383	6.987
		2		23.7	0.173		6.591	
		1	1	55.5	0.045	0.043	1.734	1.628
		2		58.6	0.040		1.522	
		1	2	114.9	0.004	0.004	0.143	0.145
		2		114.1	0.004		0.148	
		1	3	115.0	0.004	0.004	0.142	0.142
		2		115.0	0.004		0.142	
		1	4	121.2	0.003	0.003	0.110	0.112
		2		120.4	0.003		0.114	
		1	5	119.5	0.003	0.003	0.118	0.116
		2		120.1	0.003		0.115	

Table B6: RCT results for bridge 260007

Structure Number	Location	Sample Number	Depth (in)	Voltage (mV)	% Cl by concrete weight	Avg. % Cl by concrete weight	% Cl (PCY)	Avg. % Cl (PCY)
260007	L1	1	0	25.4	0.161	0.162	6.137	6.189
		2		25.0	0.164		6.241	
		1	1	55.5	0.045	0.045	1.734	1.726
		2		55.7	0.045		1.719	
		1	2	89.6	0.011	0.010	0.414	0.400
		2		91.2	0.010		0.387	
		1	3	130.0	0.002	0.002	0.076	0.077
		2		129.5	0.002		0.077	
		1	4	131.3	0.002	0.002	0.072	0.072
		2		131.0	0.002		0.073	
		1	5	132.5	0.002	0.002	0.068	0.069
		2		132.3	0.002		0.069	
	L2	1	0	38.0	0.095	0.095	3.615	3.608
		2		38.1	0.094		3.600	
		1	1	49.7	0.058	0.059	2.212	2.250
		2		48.9	0.060		2.287	
		1	2	110.6	0.004	0.004	0.171	0.163
		2		113.0	0.004		0.155	
		1	3	134.4	0.002	0.002	0.063	0.064
		2		133.9	0.002		0.064	
		1	4	129.8	0.002	0.002	0.077	0.076
		2		130.1	0.002		0.076	
		1	5	133.2	0.002	0.002	0.066	0.065
		2		133.9	0.002		0.064	



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 660019 L1 **Date:** 2/27/2020

Initial data

exposure duration, t (years) $t := 10$ must be introduced

Fitting model function

$C(x,t)=C_0(1-\mathrm{erf}(x/\mathrm{Sqrt}(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := \mathrm{data}^{\langle 0\rangle}$ $Y := \mathrm{data}^{\langle 1\rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\mathrm{erf}\left(\left(\frac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\mathrm{erf}\left(\frac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \frac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

derivative wrt to D_a

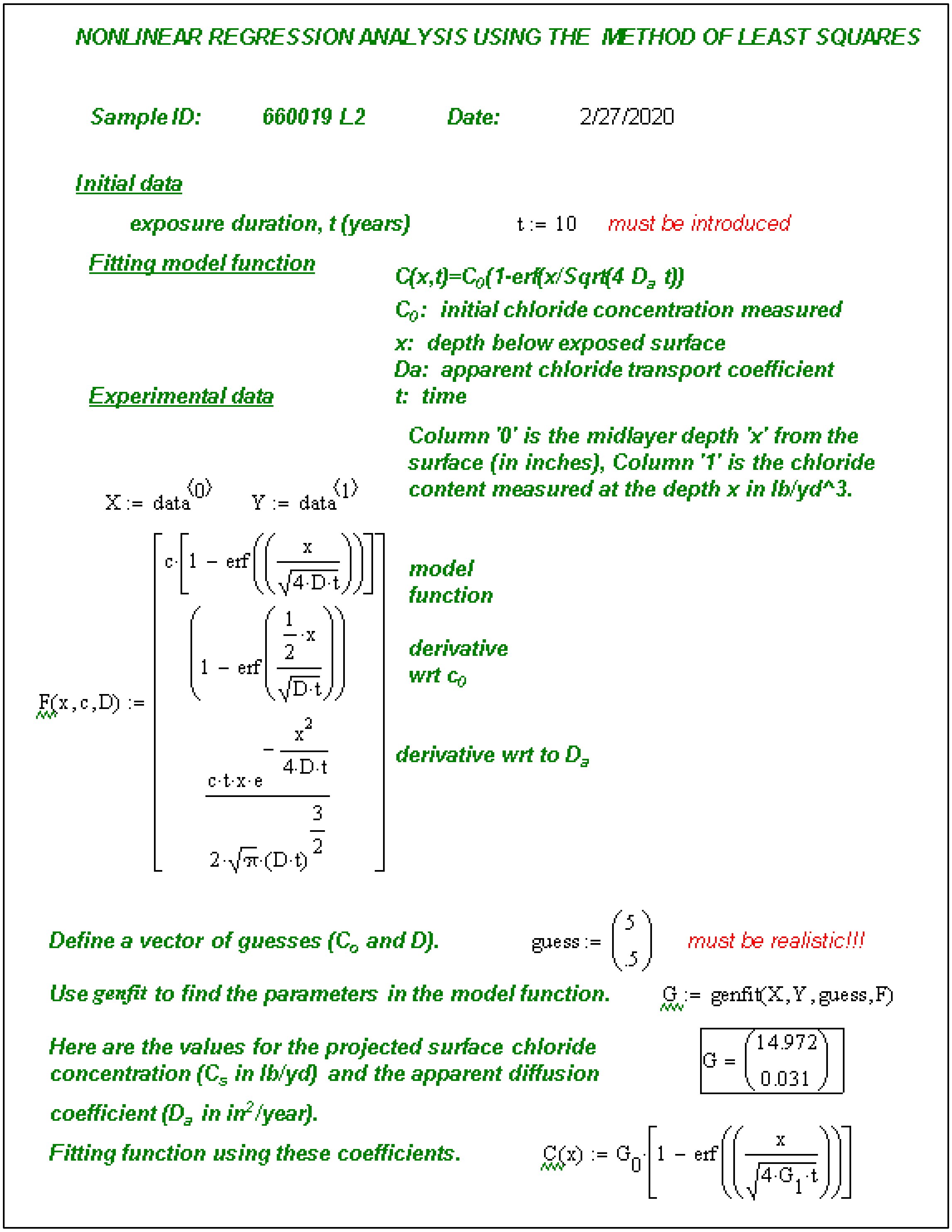
Define a vector of guesses (C_o and D). $\mathrm{guess} := \begin{pmatrix} 5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := \mathrm{genfit}(X,Y,\mathrm{guess},F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year).

$$\boxed{G = \begin{pmatrix} 9.386 \\ 0.032 \end{pmatrix}}$$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\mathrm{erf}\left(\left(\frac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 660019 L2 Date: 2/27/2020

Initial data

exposure duration, t (years) $t := 10$ must be introduced

Fitting model function

$C(x,t)=C_0(1\text{-}erf(x/Sqrt(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := data^{\langle 0 \rangle} \qquad Y := data^{\langle 1 \rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\mathrm{erf}\left(\dfrac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \dfrac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

derivative wrt to D_a

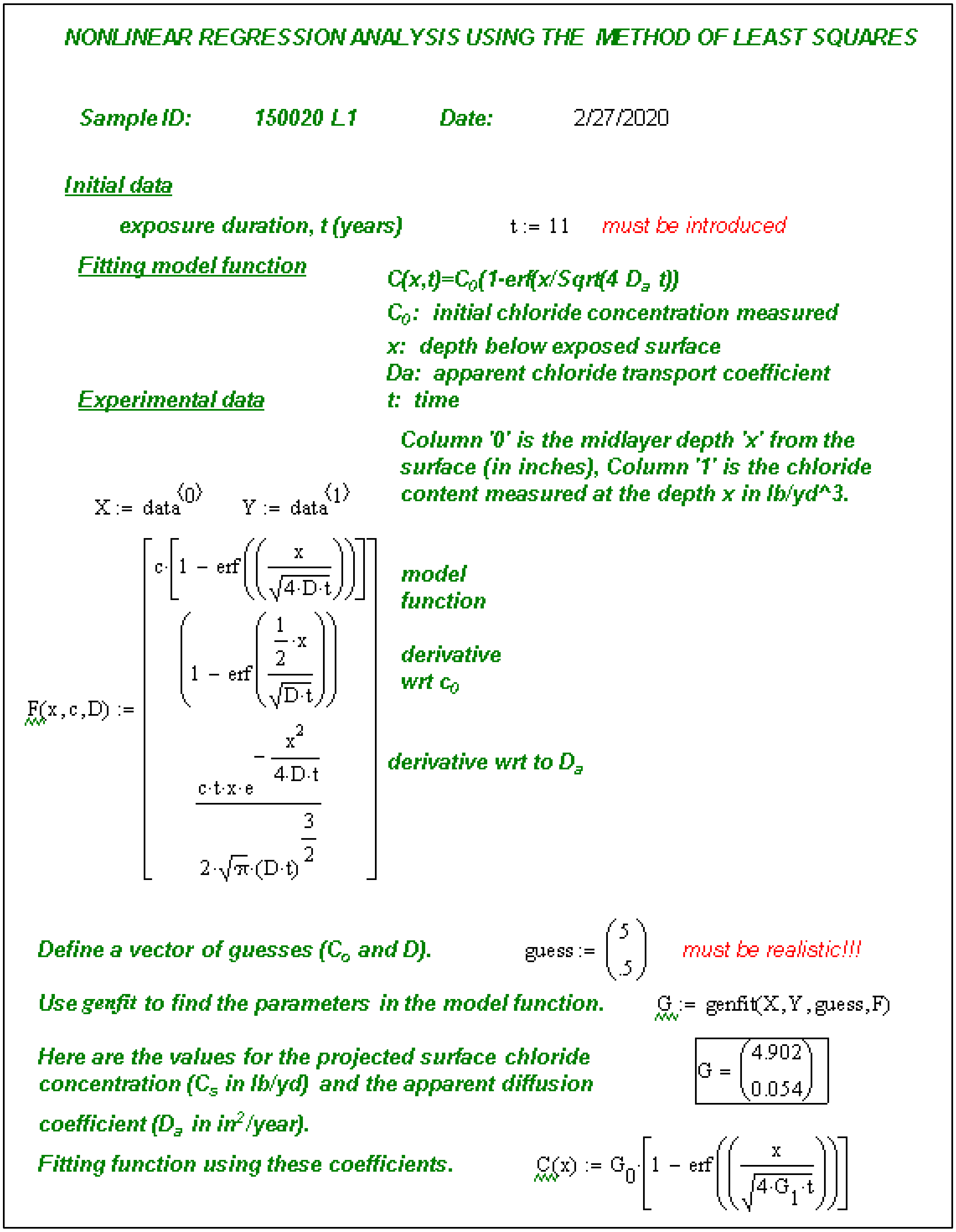
Define a vector of guesses (C_o and D). $guess := \begin{pmatrix} 5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := \mathrm{genfit}(X, Y, guess, F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year).

$$\boxed{G = \begin{pmatrix} 14.972 \\ 0.031 \end{pmatrix}}$$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 150020 L1 **Date:** 2/27/2020

Initial data

exposure duration, t (years) $t := 11$ must be introduced

Fitting model function

$C(x,t)=C_0(1\text{-}erf(x/Sqrt(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := data^{\langle 0\rangle}$ $Y := data^{\langle 1\rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\mathrm{erf}\left(\dfrac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \dfrac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

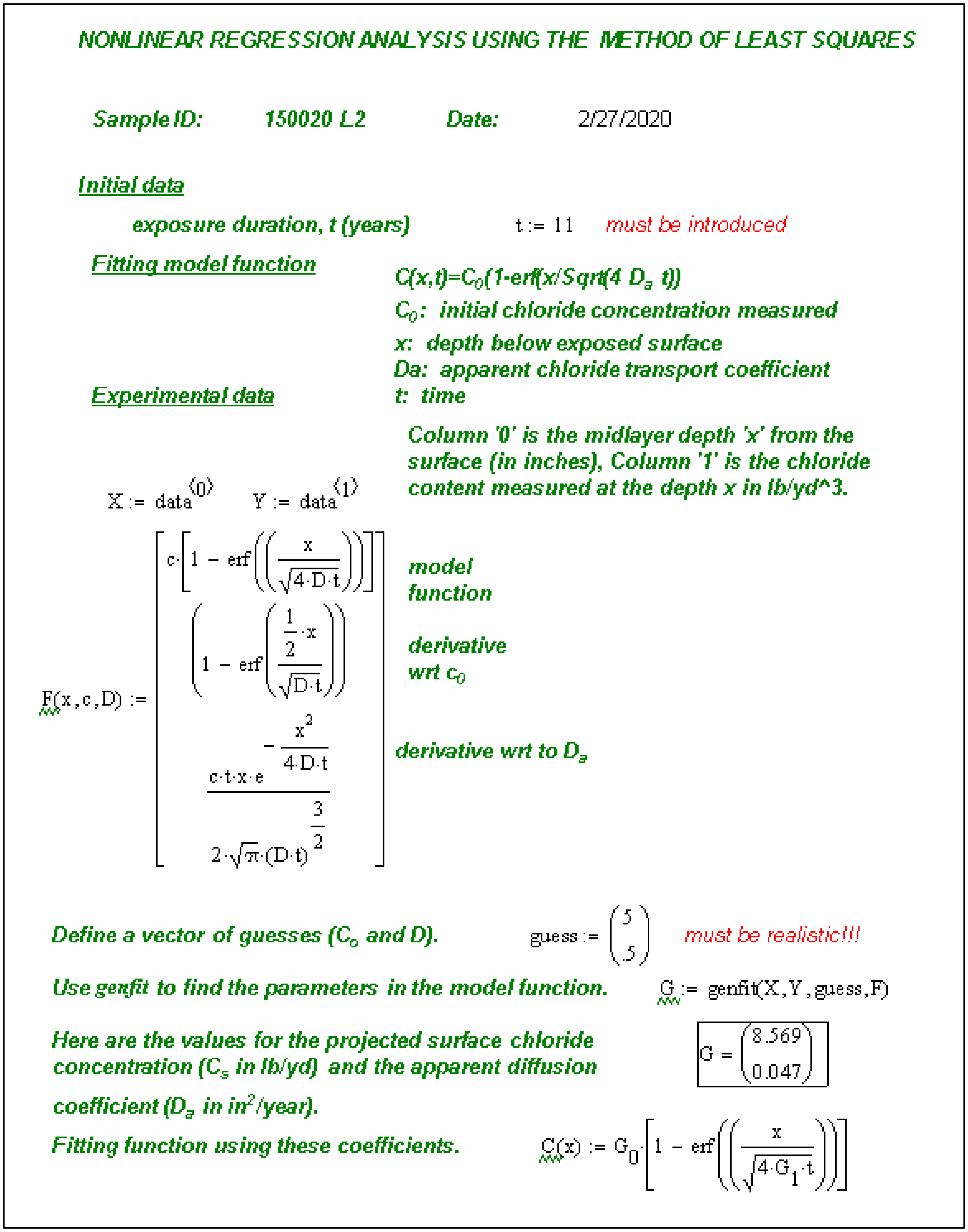
derivative wrt to D_a

Define a vector of guesses (C_o and D). $guess := \begin{pmatrix} 5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := \mathrm{genfit}(X,Y,guess,F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year). $\boxed{G = \begin{pmatrix} 4.902 \\ 0.054 \end{pmatrix}}$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 150020 L2 **Date:** 2/27/2020

Initial data

exposure duration, t (years) $t := 11$ must be introduced

Fitting model function

$C(x,t)=C_0(1\text{-}erf(x/Sqrt(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := data^{\langle 0\rangle}$ $Y := data^{\langle 1\rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\mathrm{erf}\left(\dfrac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \dfrac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

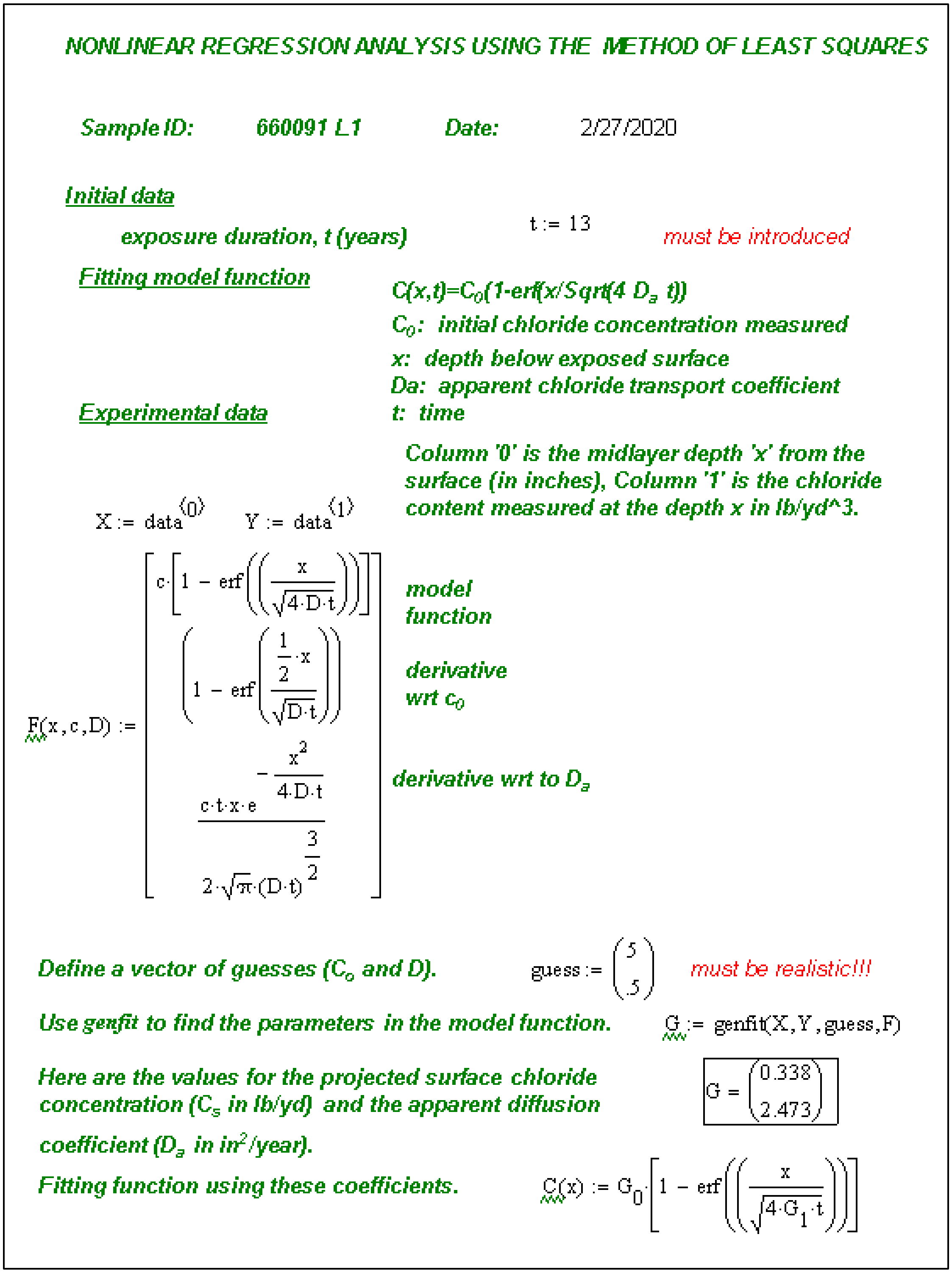
derivative wrt to D_a

Define a vector of guesses (C_o and D). $guess := \begin{pmatrix} 5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := genfit(X, Y, guess, F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year). $G = \begin{pmatrix} 8.569 \\ 0.047 \end{pmatrix}$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 660091 L1 Date: 2/27/2020

Initial data

exposure duration, t (years) $t := 13$ must be introduced

Fitting model function

$C(x,t)=C_0(1\text{-}erf(x/Sqrt(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := data^{\langle 0\rangle} \qquad Y := data^{\langle 1\rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\operatorname{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\operatorname{erf}\left(\dfrac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \dfrac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

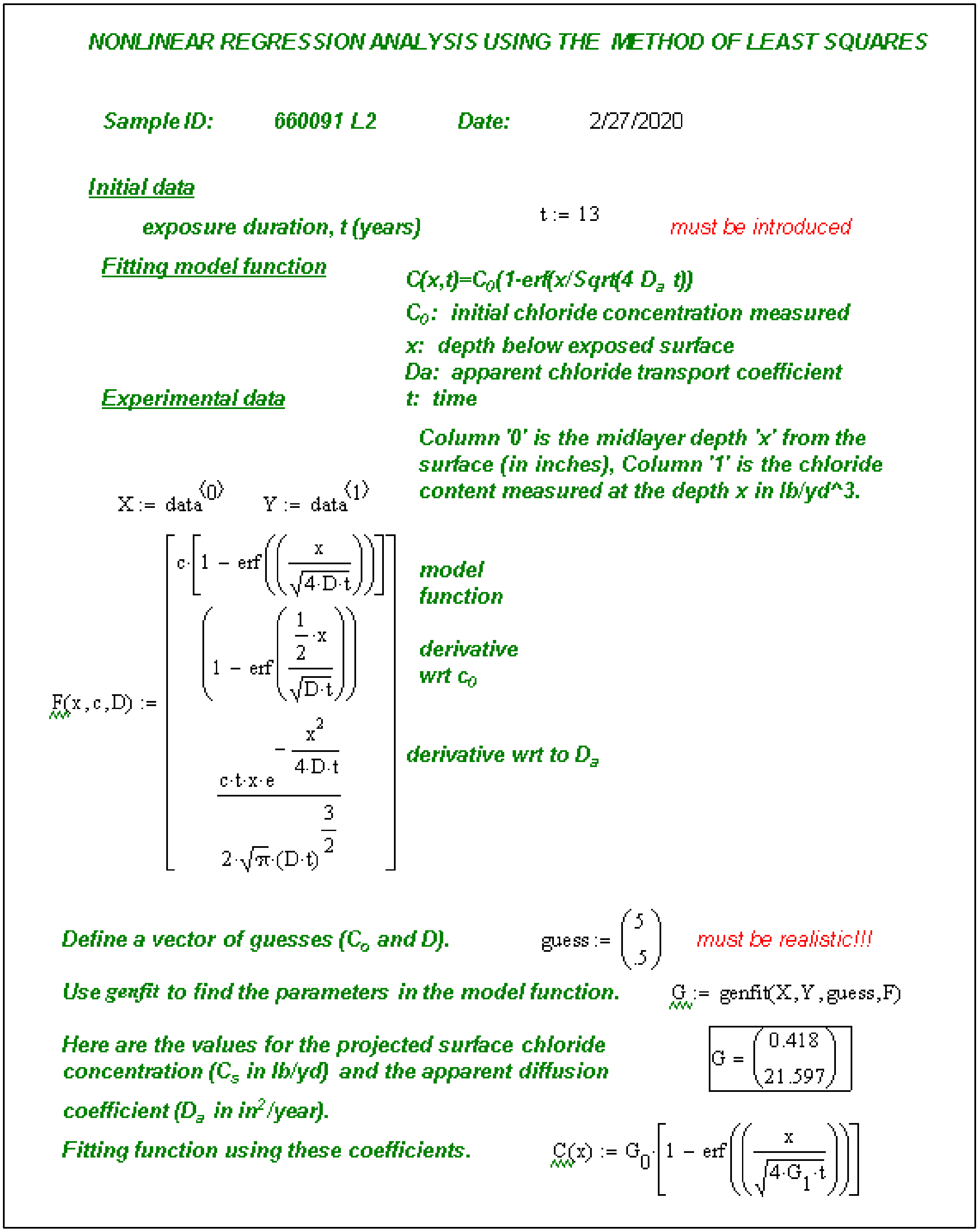
derivative wrt to D_a

Define a vector of guesses (C_o and D). $guess := \begin{pmatrix} .5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := genfit(X, Y, guess, F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year). $\boxed{G = \begin{pmatrix} 0.338 \\ 2.473 \end{pmatrix}}$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\operatorname{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: **660091 L2** **Date:** 2/27/2020

Initial data

exposure duration, t (years) $t := 13$ must be introduced

Fitting model function

$C(x,t)=C_0(1\text{-}erf(x/Sqrt(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := data^{\langle 0\rangle}$ $Y := data^{\langle 1\rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\mathrm{erf}\left(\dfrac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \dfrac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

derivative wrt to D_a

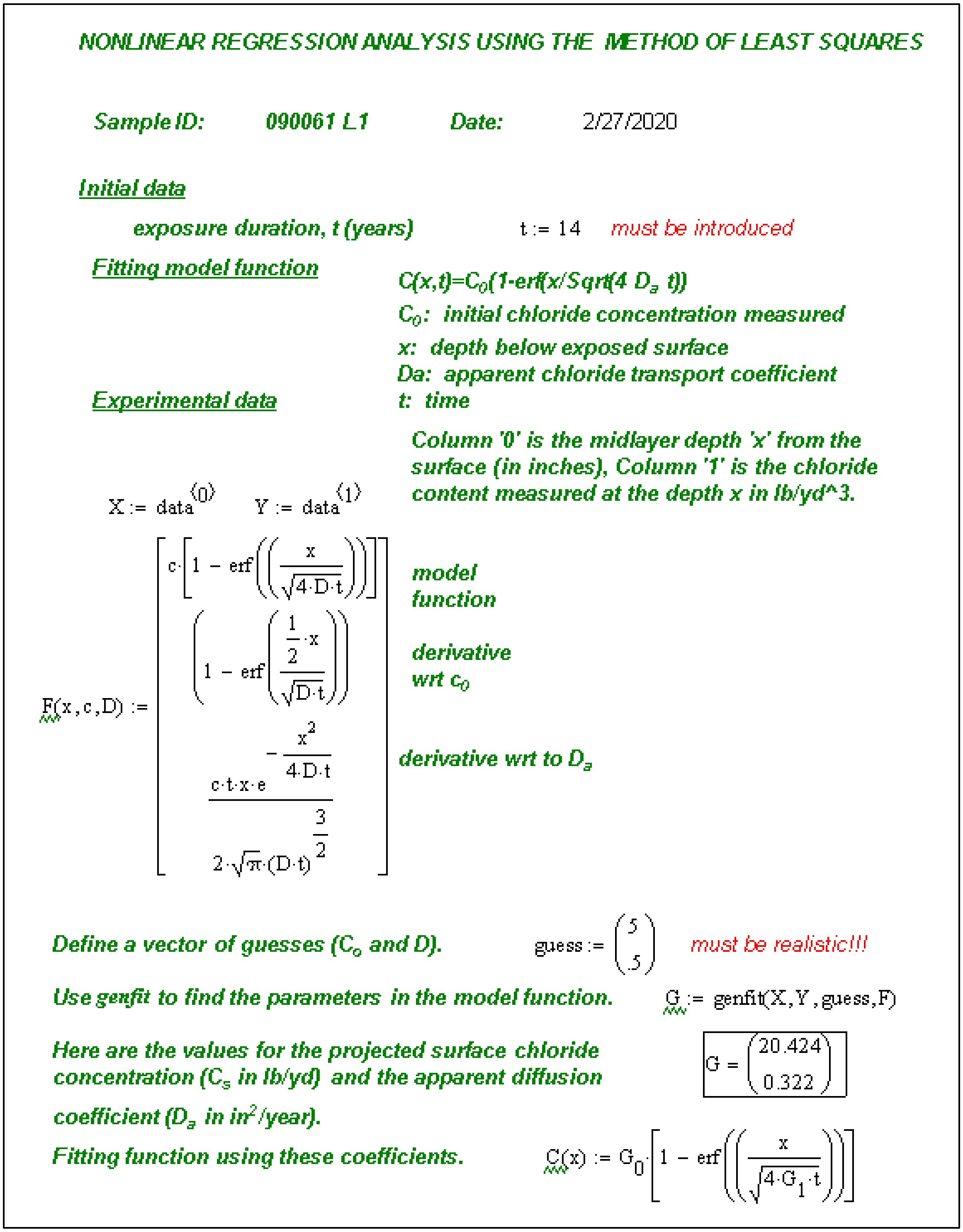
Define a vector of guesses (C_o and D). $guess := \begin{pmatrix} 5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := genfit(X, Y, guess, F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year).

$$\boxed{G = \begin{pmatrix} 0.418 \\ 21.597 \end{pmatrix}}$$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 090061 L1 Date: 2/27/2020

Initial data

exposure duration, t (years) $t := 14$ must be introduced

Fitting model function

$C(x,t)=C_0(1\text{-erf}(x/\text{Sqrt}(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := data^{\langle 0\rangle}$ $Y := data^{\langle 1\rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\operatorname{erf}\left(\left(\frac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\operatorname{erf}\left(\frac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \frac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

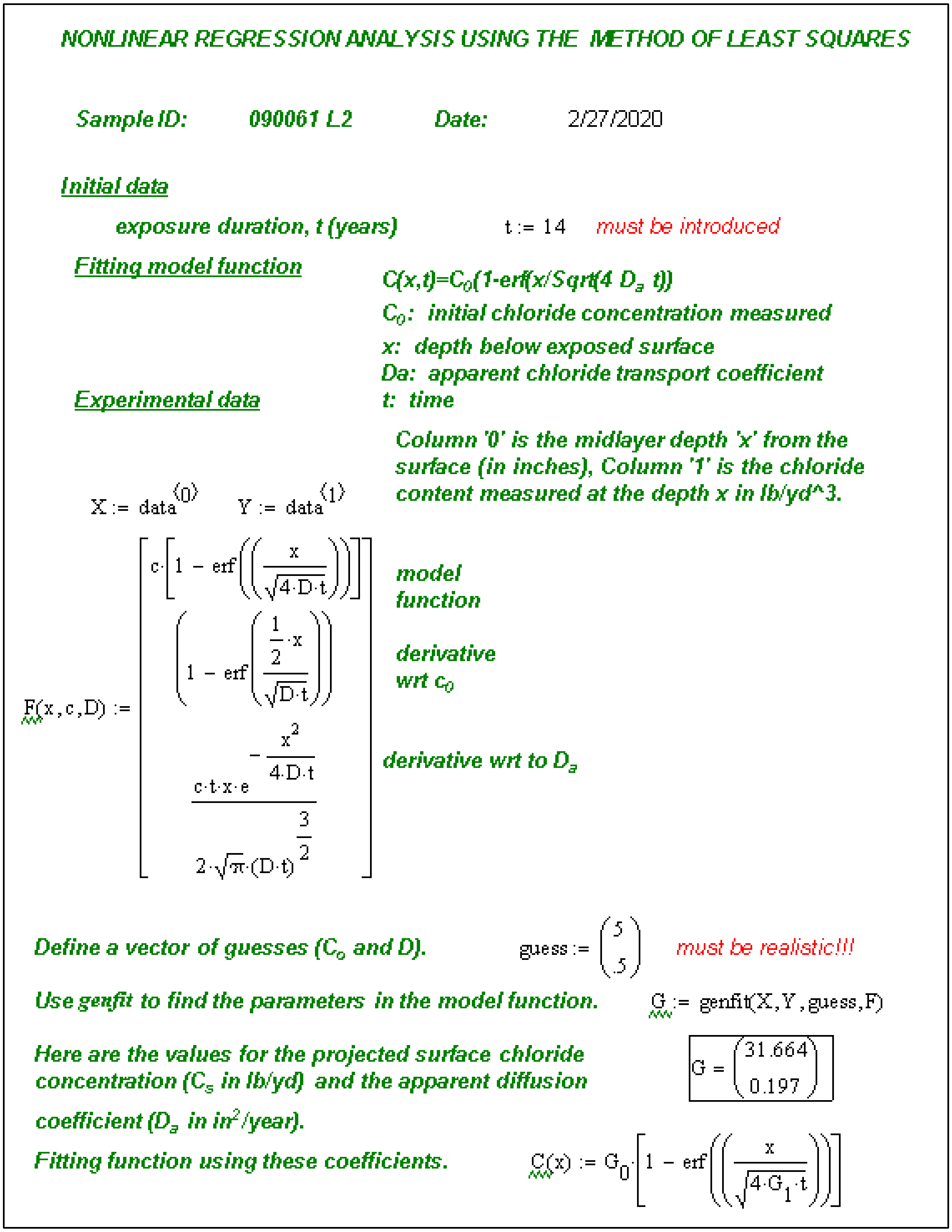
derivative wrt to D_a

Define a vector of guesses (C_o and D). $guess := \begin{pmatrix} 5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := \text{genfit}(X,Y,guess,F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year). $\boxed{G = \begin{pmatrix} 20.424 \\ 0.322 \end{pmatrix}}$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\operatorname{erf}\left(\left(\frac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 090061 L2 Date: 2/27/2020

Initial data

exposure duration, t (years) $t := 14$ must be introduced

Fitting model function

$C(x,t)=C_0(1\text{-}erf(x/Sqrt(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := data^{\langle 0 \rangle} \qquad Y := data^{\langle 1 \rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\mathrm{erf}\left(\dfrac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \dfrac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

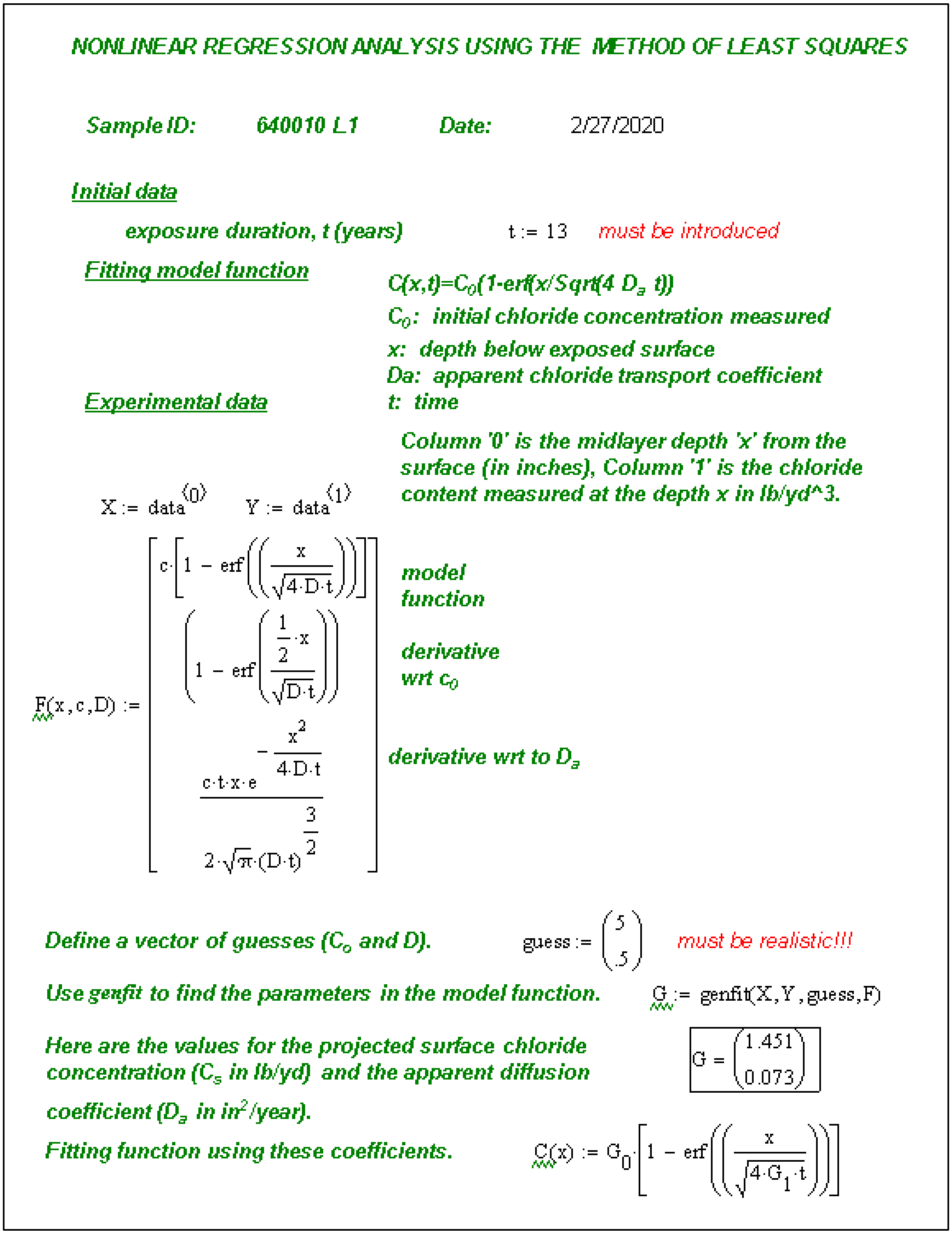
derivative wrt to D_a

Define a vector of guesses (C_o and D). $guess := \begin{pmatrix} 5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := genfit(X,Y,guess,F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year). $G = \begin{pmatrix} 31.664 \\ 0.197 \end{pmatrix}$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 640010 L1 **Date:** 2/27/2020

Initial data

exposure duration, t (years) $t := 13$ must be introduced

Fitting model function

C(x,t)=C_0(1-erf(x/Sqrt(4 D_a t))

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := data^{\langle 0 \rangle} \qquad Y := data^{\langle 1 \rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\operatorname{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\operatorname{erf}\left(\dfrac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \dfrac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

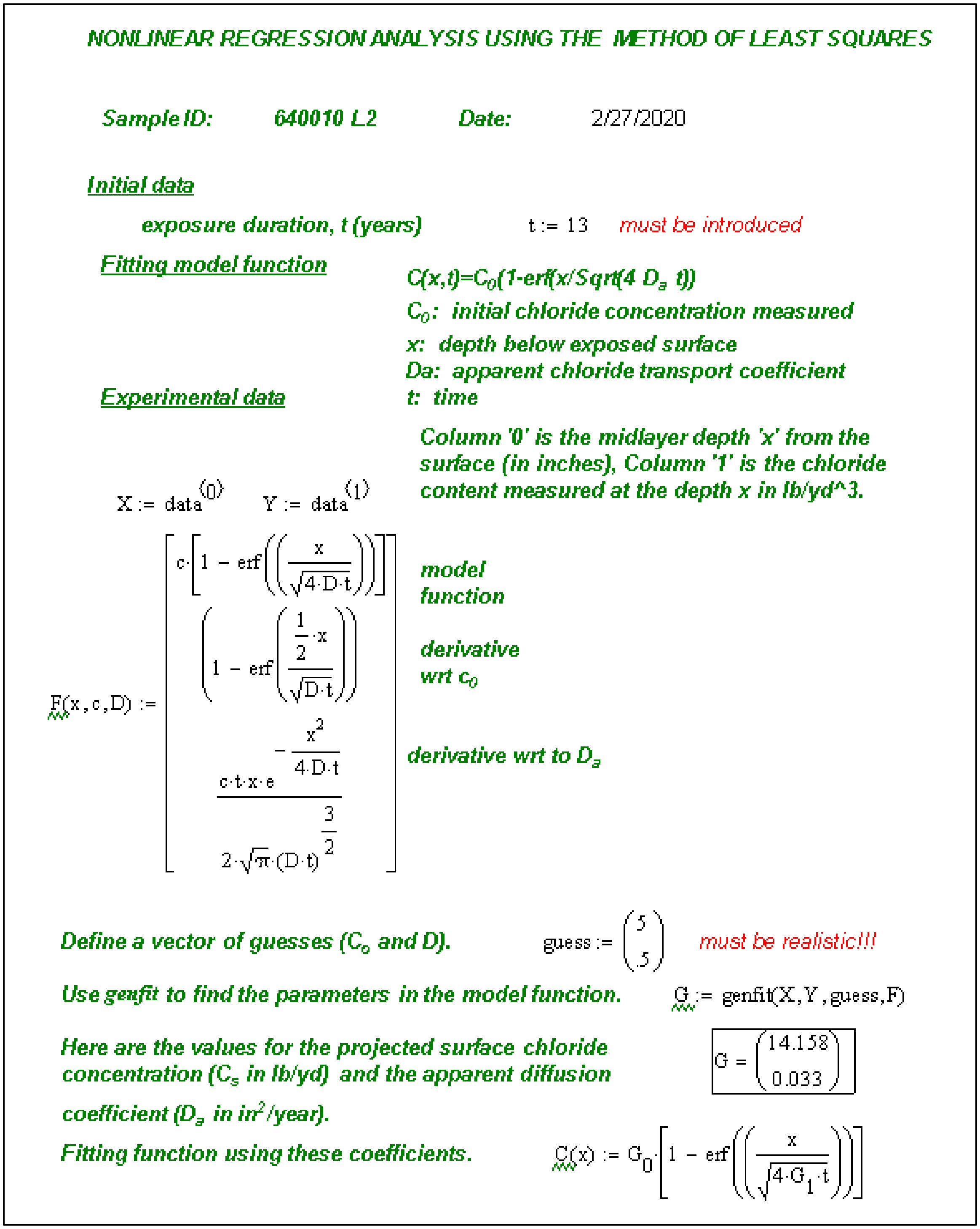
derivative wrt to D_a

Define a vector of guesses (C_o and D). $guess := \begin{pmatrix} 5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := genfit(X, Y, guess, F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year). $\boxed{G = \begin{pmatrix} 1.451 \\ 0.073 \end{pmatrix}}$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\operatorname{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 640010 L2 **Date:** 2/27/2020

Initial data

exposure duration, t (years) $t := 13$ must be introduced

Fitting model function

$C(x,t)=C_0(1\text{-erf}(x/\text{Sqrt}(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := \text{data}^{\langle 0\rangle}$ $Y := \text{data}^{\langle 1\rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\text{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\text{erf}\left(\dfrac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \dfrac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

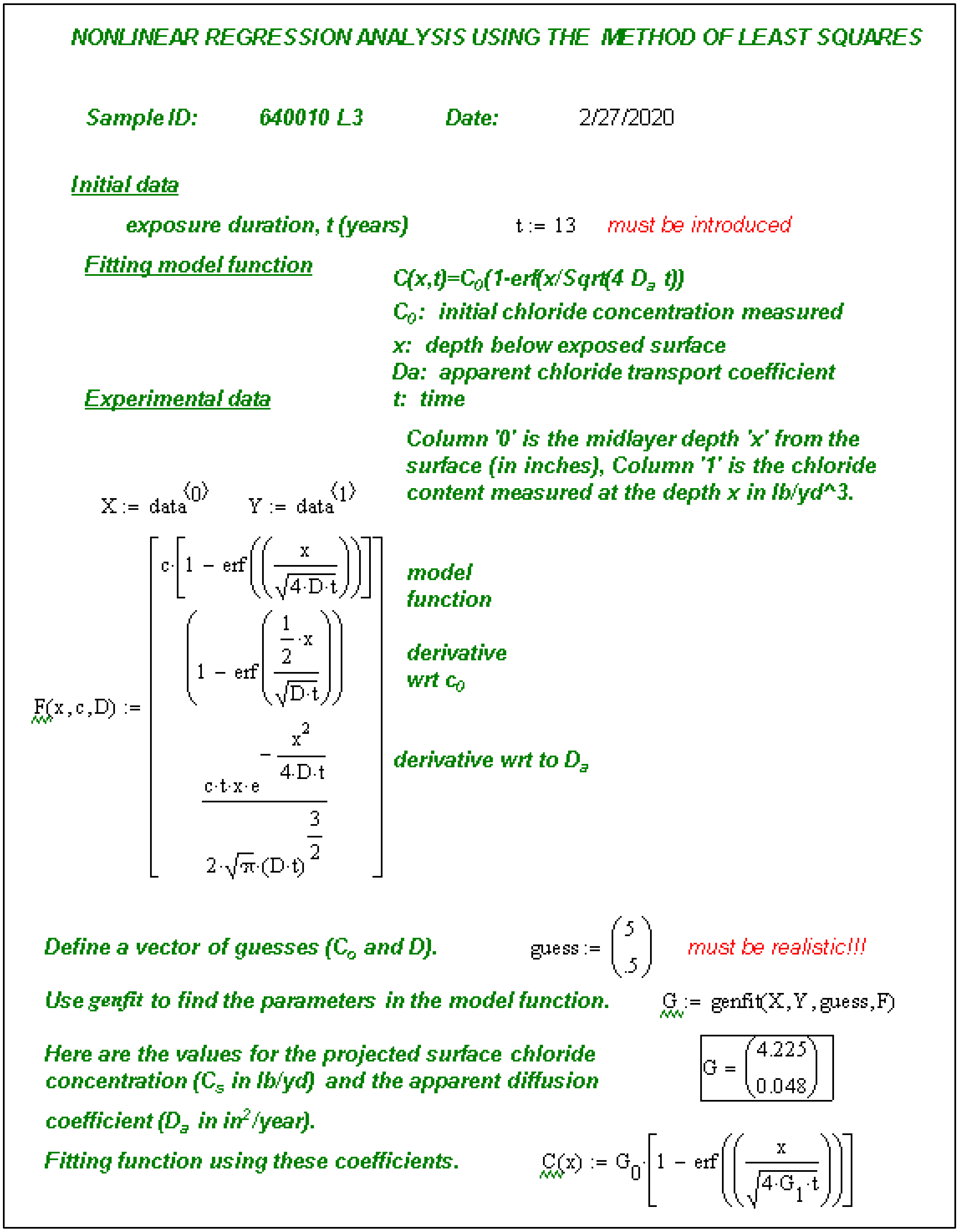
derivative wrt to D_a

Define a vector of guesses (C_o and D). $guess := \begin{pmatrix} 5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := \text{genfit}(X,Y,guess,F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year). $\boxed{G = \begin{pmatrix} 14.158 \\ 0.033 \end{pmatrix}}$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\text{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 640010 L3 Date: 2/27/2020

Initial data

exposure duration, t (years) $t := 13$ must be introduced

Fitting model function

$C(x,t)=C_0(1\text{-}erf(x/Sqrt(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := data^{\langle 0 \rangle}$ $Y := data^{\langle 1 \rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1 - \mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1 - \mathrm{erf}\left(\dfrac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \dfrac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

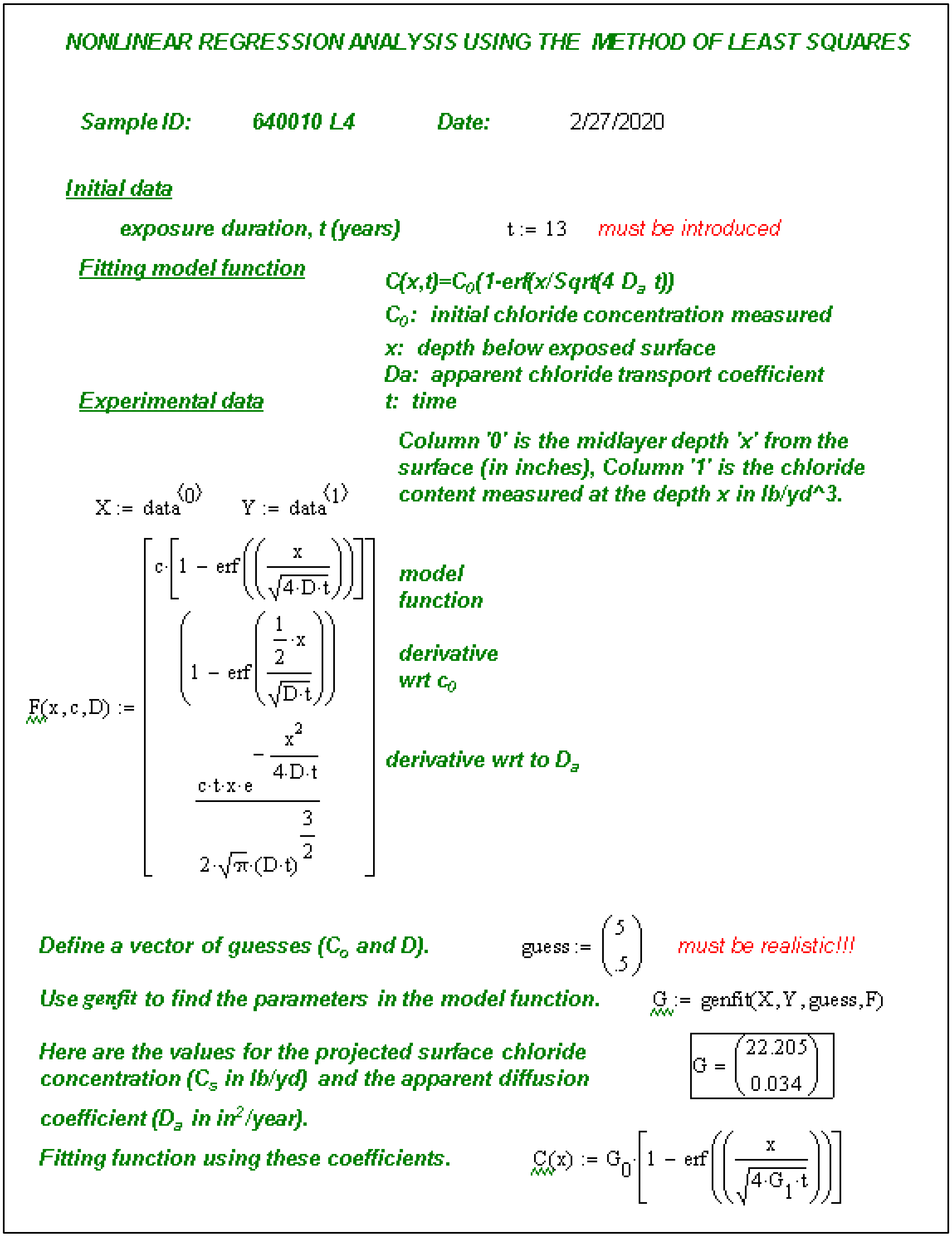
derivative wrt to D_a

Define a vector of guesses (C_o and D). $guess := \begin{pmatrix} .5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := genfit(X,Y,guess,F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year). $\boxed{G = \begin{pmatrix} 4.225 \\ 0.048 \end{pmatrix}}$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1 - \mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 640010 L4 Date: 2/27/2020

Initial data

exposure duration, t (years) $t := 13$ must be introduced

Fitting model function

$C(x,t)=C_0(1\text{-}erf(x/Sqrt(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := data^{\langle 0 \rangle}$ $Y := data^{\langle 1 \rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\mathrm{erf}\left(\dfrac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \dfrac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

derivative wrt to D_a

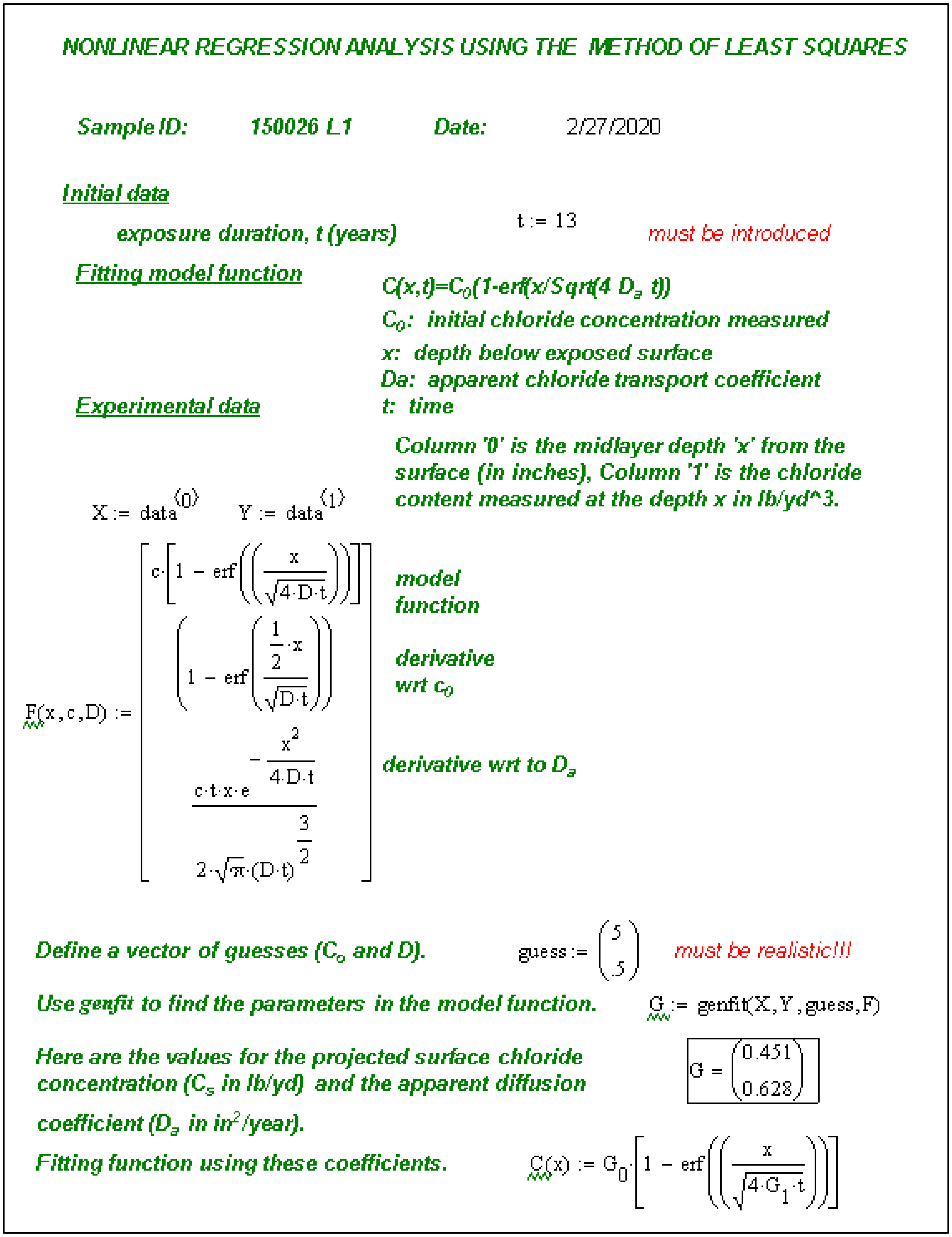
Define a vector of guesses (C_o and D). $guess := \begin{pmatrix} .5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := genfit(X,Y,guess,F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year).

$$\boxed{G = \begin{pmatrix} 22.205 \\ 0.034 \end{pmatrix}}$$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 150026 L1 Date: 2/27/2020

Initial data

exposure duration, t (years) $t := 13$ must be introduced

Fitting model function

$C(x,t)=C_0(1\text{-}erf(x/Sqrt(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := data^{\langle 0\rangle}$ $Y := data^{\langle 1\rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\mathrm{erf}\left(\dfrac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \dfrac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

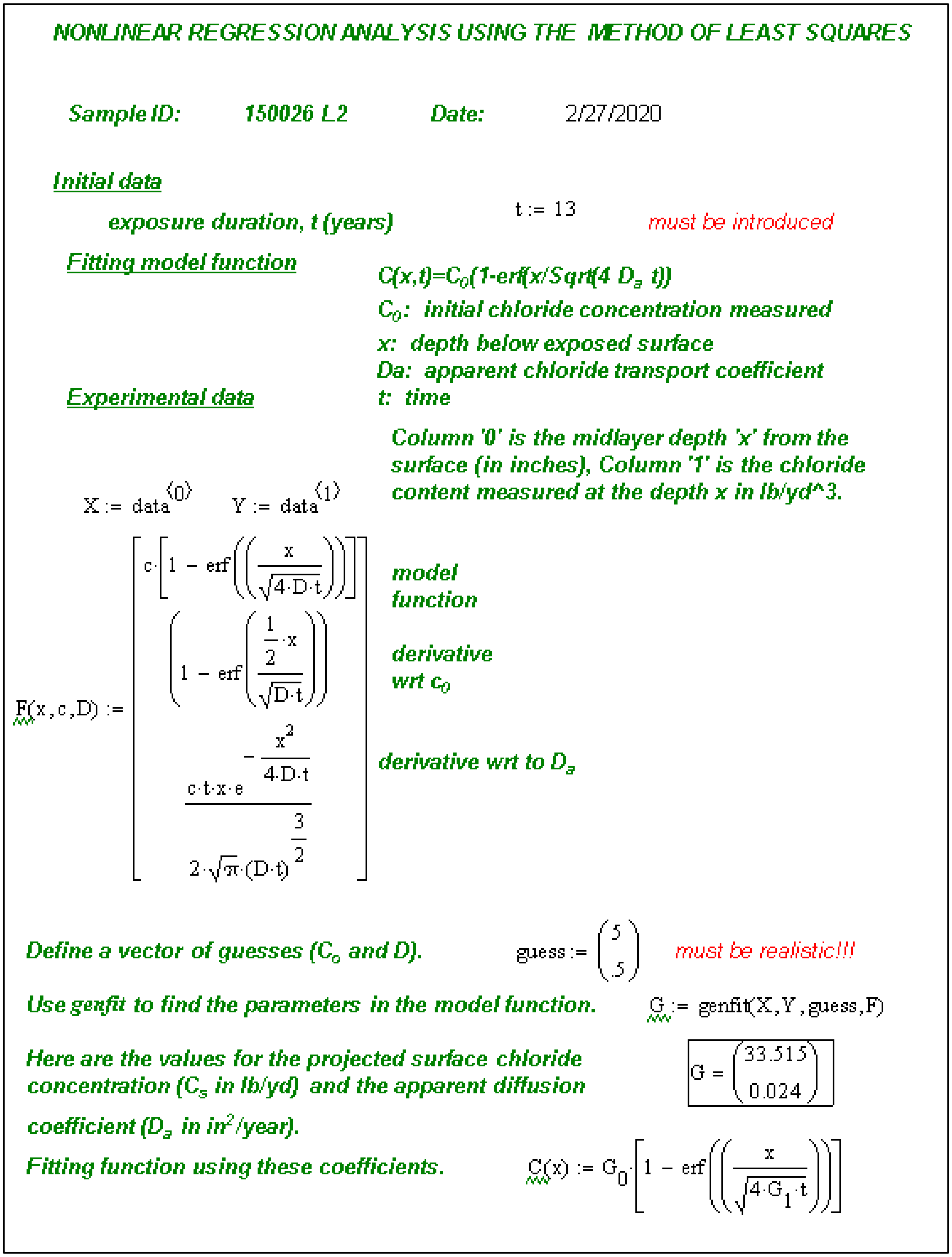
derivative wrt to D_a

Define a vector of guesses (C_o and D). $guess := \begin{pmatrix} 5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := \mathrm{genfit}(X, Y, guess, F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year). $\boxed{G = \begin{pmatrix} 0.451 \\ 0.628 \end{pmatrix}}$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: *150026 L2* *Date:* 2/27/2020

Initial data

exposure duration, t (years) $t := 13$ *must be introduced*

Fitting model function *C(x,t)=C$_0$(1-erf(x/Sqrt(4 D$_a$ t))*

C$_0$: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

Experimental data *t: time*

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := \mathrm{data}^{\langle 0\rangle}$ $Y := \mathrm{data}^{\langle 1\rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\mathrm{erf}\left(\left(\frac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\mathrm{erf}\left(\frac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \frac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c$_0$

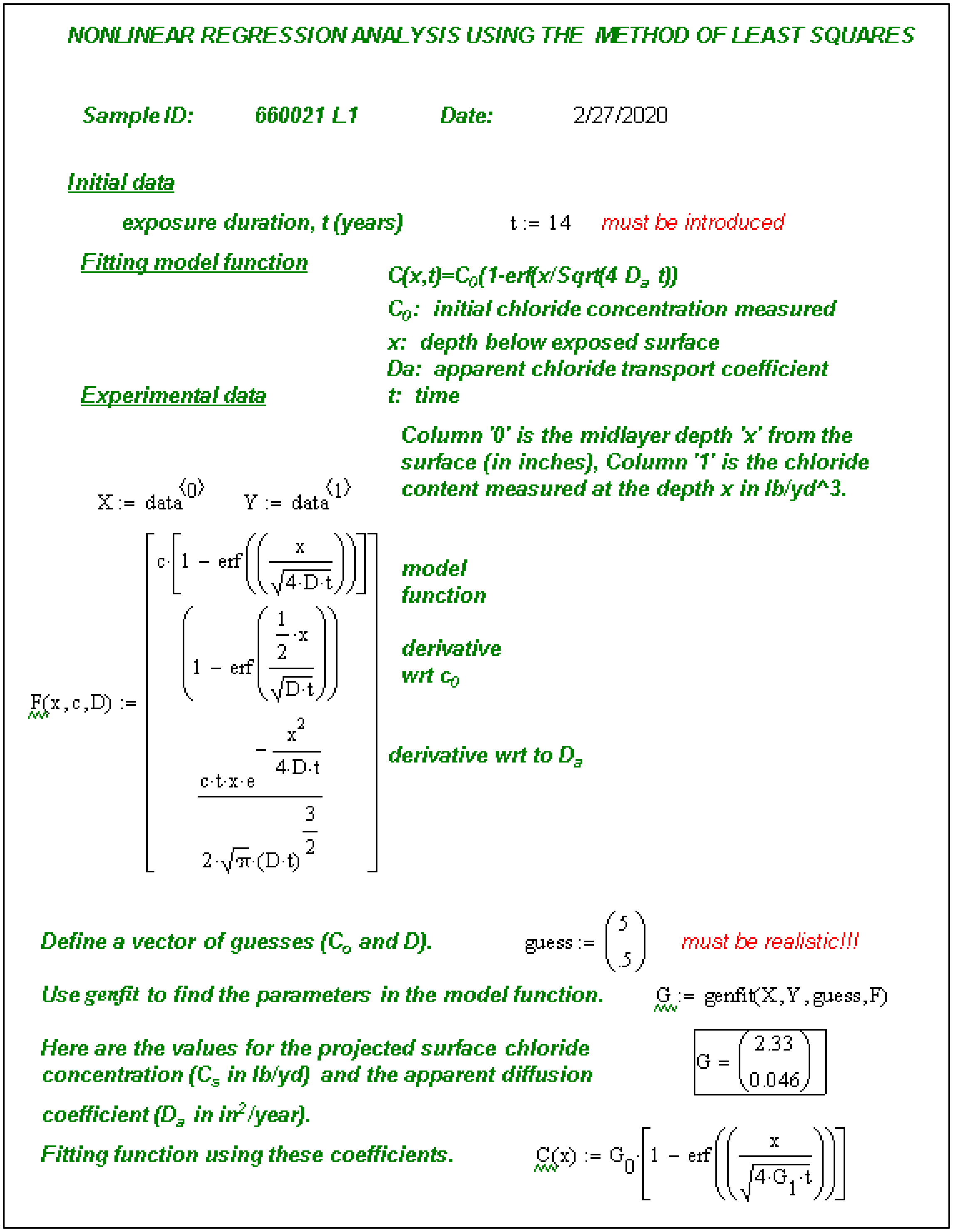
derivative wrt to D$_a$

Define a vector of guesses (C$_o$ and D). $\mathrm{guess} := \begin{pmatrix} 5 \\ .5 \end{pmatrix}$ *must be realistic!!!*

Use genfit to find the parameters in the model function. $G := \mathrm{genfit}(X, Y, \mathrm{guess}, F)$

Here are the values for the projected surface chloride concentration (C$_s$ in lb/yd) and the apparent diffusion coefficient (D$_a$ in in^2/year). $G = \begin{pmatrix} 33.515 \\ 0.024 \end{pmatrix}$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\mathrm{erf}\left(\left(\frac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 660021 L1 Date: 2/27/2020

Initial data

exposure duration, t (years) $t := 14$ must be introduced

Fitting model function

$C(x,t)=C_0(1\text{-}erf(x/Sqrt(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := data^{\langle 0\rangle}$ $Y := data^{\langle 1\rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\operatorname{erf}\left(\left(\frac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\operatorname{erf}\left(\frac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \frac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

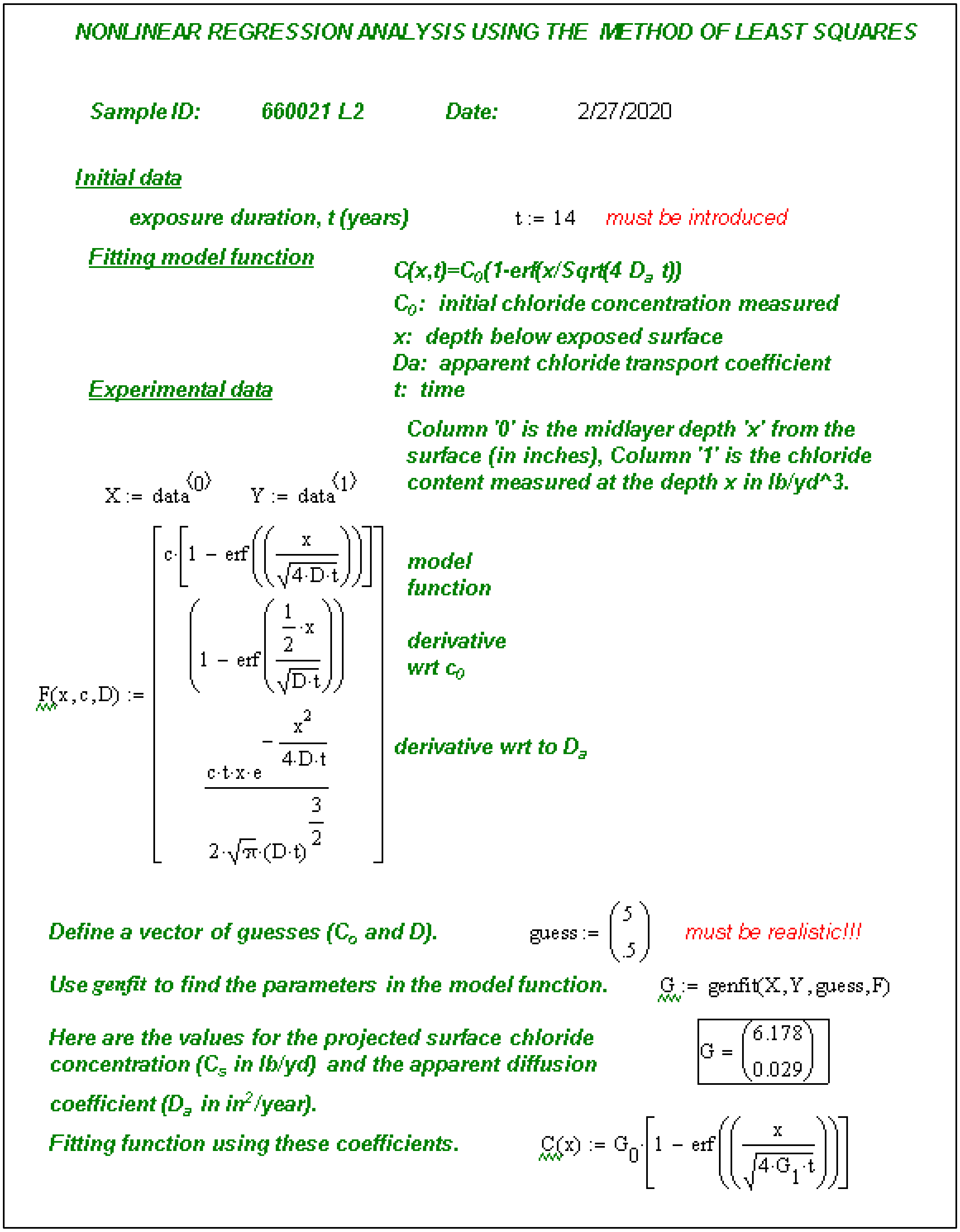
derivative wrt to D_a

Define a vector of guesses (C_o and D). $guess := \begin{pmatrix} .5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := genfit(X,Y,guess,F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year). $\boxed{G = \begin{pmatrix} 2.33 \\ 0.046 \end{pmatrix}}$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\operatorname{erf}\left(\left(\frac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: **660021 L2** **Date:** 2/27/2020

Initial data

exposure duration, t (years) $t := 14$ *must be introduced*

Fitting model function

$C(x,t)=C_0(1\text{-}erf(x/Sqrt(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := \text{data}^{\langle 0\rangle} \qquad Y := \text{data}^{\langle 1\rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\text{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\text{erf}\left(\dfrac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \dfrac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

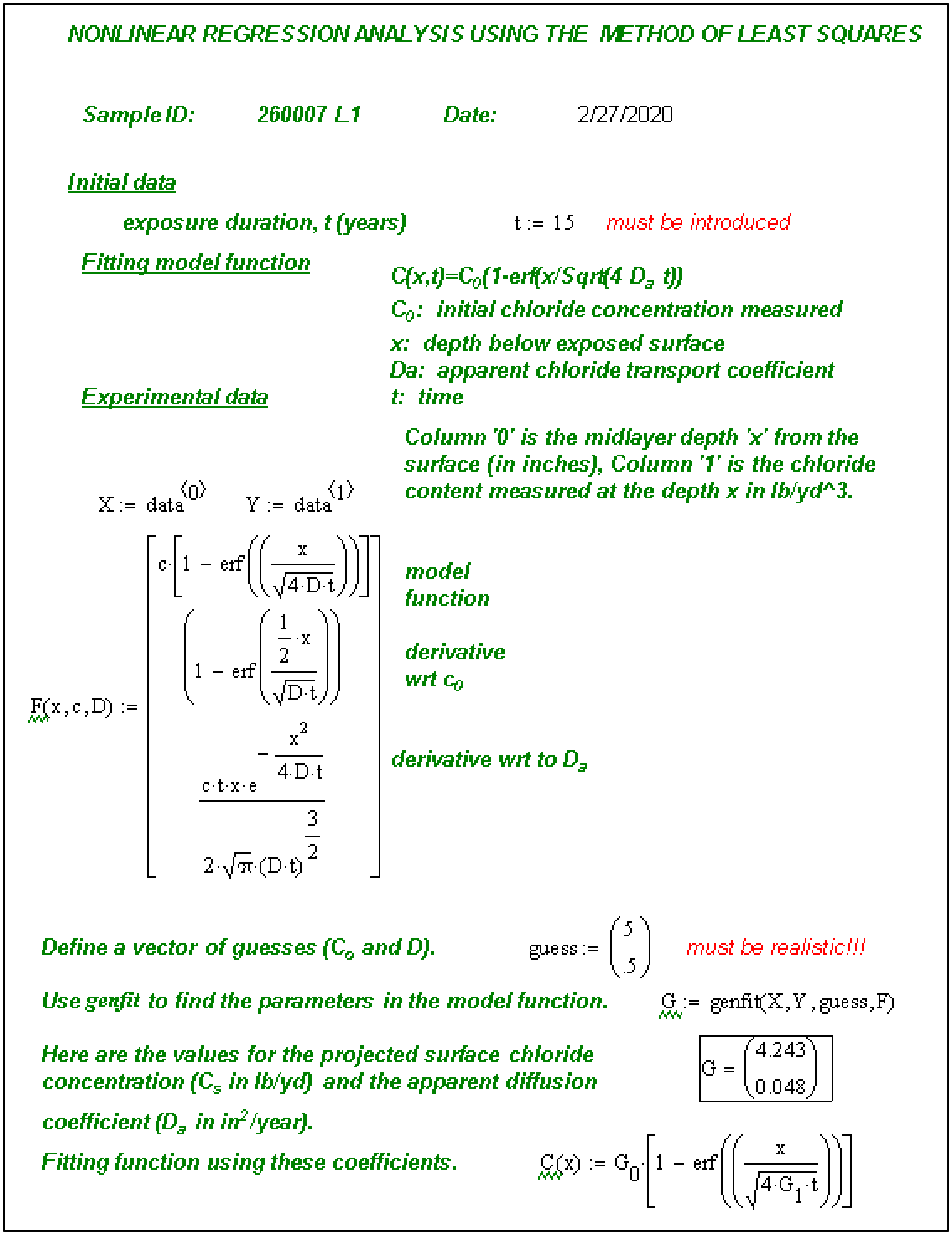
derivative wrt to D_a

Define a vector of guesses (C_o and D). $\text{guess} := \begin{pmatrix} 5 \\ .5 \end{pmatrix}$ *must be realistic!!!*

Use genfit to find the parameters in the model function. $G := \text{genfit}(X, Y, \text{guess}, F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year). $\boxed{G = \begin{pmatrix} 6.178 \\ 0.029 \end{pmatrix}}$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\text{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 260007 L1 Date: 2/27/2020

Initial data

exposure duration, t (years) $t := 15$ must be introduced

Fitting model function

$C(x,t)=C_0(1\text{-}erf(x/Sqrt(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := data^{\langle 0\rangle}$ $Y := data^{\langle 1\rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\mathrm{erf}\left(\left(\frac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\mathrm{erf}\left(\frac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \frac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

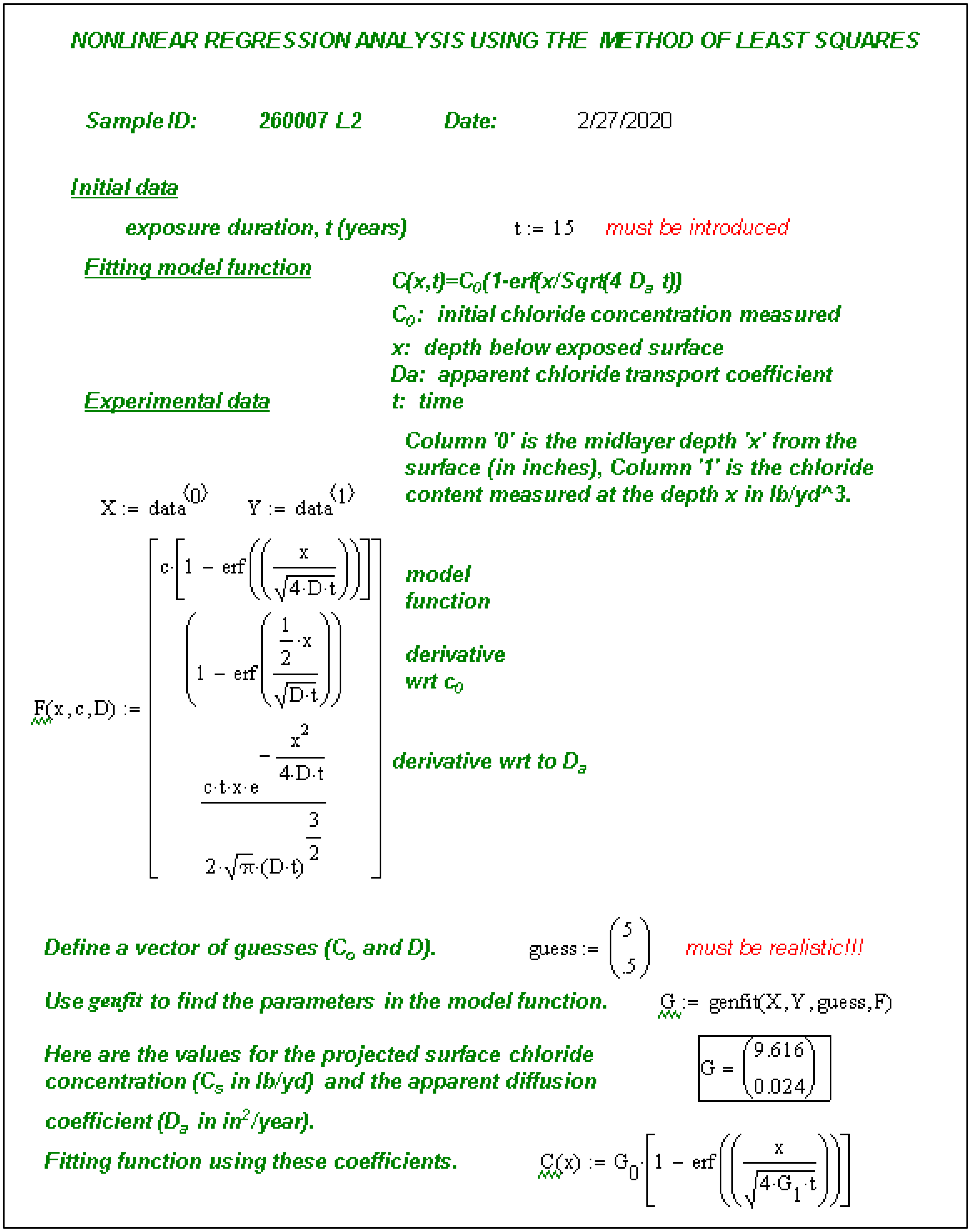
derivative wrt to D_a

Define a vector of guesses (C_o and D). $guess := \begin{pmatrix} .5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := genfit(X,Y,guess,F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year). $G = \begin{pmatrix} 4.243 \\ 0.048 \end{pmatrix}$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\mathrm{erf}\left(\left(\frac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 260007 L2 Date: 2/27/2020

Initial data

exposure duration, t (years) $t := 15$ must be introduced

Fitting model function

$C(x,t)=C_0(1\text{-}erf(x/Sqrt(4\ D_a\ t))$

C_0: initial chloride concentration measured

x: depth below exposed surface

Da: apparent chloride transport coefficient

t: time

Experimental data

Column '0' is the midlayer depth 'x' from the surface (in inches), Column '1' is the chloride content measured at the depth x in lb/yd^3.

$X := data^{\langle 0\rangle} \qquad Y := data^{\langle 1\rangle}$

$$F(x,c,D) := \begin{bmatrix} c\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot D\cdot t}}\right)\right)\right] \\ \left(1-\mathrm{erf}\left(\dfrac{\frac{1}{2}\cdot x}{\sqrt{D\cdot t}}\right)\right) \\ \dfrac{c\cdot t\cdot x\cdot e^{-\frac{x^2}{4\cdot D\cdot t}}}{2\cdot\sqrt{\pi}\cdot(D\cdot t)^{\frac{3}{2}}} \end{bmatrix}$$

model function

derivative wrt c_0

derivative wrt to D_a

Define a vector of guesses (C_o and D). $guess := \begin{pmatrix} 5 \\ .5 \end{pmatrix}$ must be realistic!!!

Use genfit to find the parameters in the model function. $G := genfit(X, Y, guess, F)$

Here are the values for the projected surface chloride concentration (C_s in lb/yd) and the apparent diffusion coefficient (D_a in in^2/year). $\boxed{G = \begin{pmatrix} 9.616 \\ 0.024 \end{pmatrix}}$

Fitting function using these coefficients. $C(x) := G_0\cdot\left[1-\mathrm{erf}\left(\left(\dfrac{x}{\sqrt{4\cdot G_1\cdot t}}\right)\right)\right]$

Concentration Analysis Report

Report time 3/17/2020 1:19:23 PM
Batch name C:\Users\Admin\Desktop\Ross UV\ACTUAL TEST 2.BCN
Application Concentration 5.0.0.999
Operator Ross Newsome
Instrument Settings

Instrument Cary 60
Instrument version no. 2.00
Wavelength (nm) 540.0
Ordinate Mode Abs
Ave Time (sec) 0.1000
Replicates 1
Standard/Sample averaging OFF
Weight and volume corrections OFF
Fit type Linear
Min R² 0.95000
Concentration units mg/mL

Analysis

Collection time 3/17/2020 1:19:23 PM
Sample Concentration mg/mL F Readings
270007-L1 5in 0.090 0.1628
270007-L1 2in 0.223 0.2636
150026-L2 2in 0.198 0.2448
150026-L2 5in 0.222 0.2628
150020-L2 2in 0.212 0.2555
150020-L2 5in 0.215 0.2572
660021-L1 2in 0.275 0.3028
660021-L1 5in 0.227 0.2669
090061-L3 2in 0.139 0.1998
090061-L3 5in 0.217 0.2587
660019-L1 2in 0.257 0.2897
660019-L1 3in 0.304 0.3252
640010-L1 2in 0.169 0.2229
640010-L1 5in 0.202 0.2475
Results Flags Legend

U = Uncalibrated O = Overrange
N = Not used in calibration R = Repeat reading