



# **SERVICE LIFE DESIGN GUIDE**

## **For Corrosion Prevention of Concrete Structures**

**MAY 2023**

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**LOSSAN RAIL CORRIDOR**  
**San Diego, California**



Torrey Pines overhead bridge on LOSSAN Rail Corridor

## Approvals page for Service Life Design Guide for Corrosion Prevention of Concrete Structures in San Diego County

Prepared by: Tourney Consulting Group (TCG)

This document is published as a set of general guidelines to meet service life requirements of concrete structures in San Diego County. The guidelines are intended to prevent premature corrosion of concrete structures including rail bridges, stations, and other structures related to SANDAG Rail Projects located within the San Diego subdivision of the LOSSAN corridor. The intent of the guidelines is to establish general recommendations and criteria for project design. The Engineer is still responsible for project approach.

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## Table of Contents

<b>1 Corrosion Prevention .....</b>	<b>1</b>
<b>1.1 General.....</b>	<b>1</b>
<b>1.2 Standards, Codes and Guidelines .....</b>	<b>2</b>
<b>1.3 Design Life .....</b>	<b>2</b>
<b>1.4 Service Life Design Using Introduction to STADIUM.....</b>	<b>3</b>
<b>1.5 Degradation Mechanisms .....</b>	<b>4</b>
<b>1.5.1 Chloride Induced Corrosion .....</b>	<b>4</b>
<b>1.5.2 Carbonation Induced Corrosion.....</b>	<b>5</b>
<b>1.5.3 Sulfate Attack .....</b>	<b>5</b>
1.5.4 Alkali-Aggregate Reaction.....	5
1.5.5 Seawater Attack .....	6
1.5.6 Summary .....	7
<b>1.6 Service Life Design Process.....</b>	<b>8</b>
1.6.1 Service Life Engineering Flowchart .....	8
<b>1.7 Field Investigation and Exposure Zones .....</b>	<b>11</b>
1.7.1 Field Investigation .....	11
1.7.2 Environmental Exposure Condition Zones.....	14
1.7.3 Marine Severe – Exposure Zone .....	14
1.7.4 Marine Moderate – Exposure Zone .....	16
1.7.5 Inland Moderate – Exposure Zone.....	17
1.7.6 Inland Low – Exposure Zone .....	18
1.7.7 Condition Assessments .....	19
1.7.8 Exposure zone summary tables .....	19
1.7.9 Structure Exposure Explanation.....	22
Marine Structures .....	22
Inland Structures.....	23
<b>1.8 Concrete Mixtures .....</b>	<b>23</b>
1.8.1 Reinforced Concrete Materials.....	32

Portland Cement ..... 32

Blended Hydraulic Cement ..... 32

Expansive Cement..... 32

Aggregates ..... 33

1.8.2 Supplementary Cementitious Materials ..... 33

Fly Ash ..... 34

Silica Fume ..... 34

Ground-Granulated Blast Furnace Slag..... 35

High-reactivity Metakaolin ..... 35

1.8.3 Admixtures ..... 35

Air Entraining ..... 35

Water Reducing ..... 35

Set Controlling..... 36

Shrinkage Reducing..... 36

Corrosion Inhibiting ..... 36

**1.9 Corrosion Protection Systems ..... 37**

Reinforcement ..... 37

Black Steel ..... 37

Epoxy-Coated Steel (ECR) ..... 37

Other Corrosion Resistant Reinforcement..... 38

Corrosion Inhibiting Admixtures ..... 38

Surface Treatments..... 39

Surface Barriers..... 40

Concrete Cover ..... 41

Cover Tolerances..... 41

Placing and Finishing Concrete ..... 41

Curing Concrete ..... 42

Control of Cracking in Concrete ..... 42

Drying..... 42

Thermal ..... 42

Design-related..... 43

**1.10 LOSSAN Rail Corridor Design Methodology..... 43**

Design Methodology Steps ..... 43

Corrosion Protection Systems..... 45

Corrosion Protection Enhancements ..... 45

Design Example ..... 45

**1.11 Service Life Design Charts..... 47**

1.11.1 Inland Low Exposure ..... 47

1.11.2 Inland Moderate Exposure ..... 49

1.11.3 Marine Moderate (Airborne) Exposure ..... 57

1.11.4 Marine Severe Exposure ..... 62

**1.12 Specification Requirements..... 80**

Submittal and Acceptance ..... 80

Drying Shrinkage ..... 80

Alkali-Aggregate Reactivity ..... 80

Water Soluble Chloride Ion Content..... 81

Transport properties..... 81

Quality control/ Quality assurance ..... 82

**Appendix..... 83**

Appendix A - Terminology..... 83

Appendix B - STADIUM<sup>®</sup> MODEL..... 88

**Index of Figures**

Figure 1: Chloride Induced Corrosion ..... 4

Figure 2 - Carbonation depth being measured in a specimen..... 5

Figure 3 - Sulfate attack in concrete ..... 5

Figure 4 - Alkali aggregate reaction in Br 225.3 San Luis Rey River from 1916 ..... 6

Figure 5 - Evidence of ASR in Br 225.3 pier 2 ..... 6

Figure 6 - 100-year-old railway culvert outfall built in 1910 on the beach in Del Mar at MP244.3 ..... 7

Figure 7 – Flow Chart ..... 8

Figure 8 – Br 225.3 Sampling ..... 11

Figure 9 – Br 250.9 Sample location ..... 11

Figure 10 - Test samples ..... 12

Figure 11a - Chloride profile of the sample ..... 12

Figure 12 – San Elijo Lagoon Rail Trestle Br 240.4 substructure in Marine Severe tidal exposure (Photo Shoaff) ..... 15

Figure 13 – Torrey Pines Overhead Br 245.7 - Marine moderate (Airborne) exposure (Photo Shoaff) ..... 16

Figure 14 – Sorrento Valley - Inland moderate exposure (Photos Shoaff) ..... 17

Figure 15 - Inland moderate exposure at Br 248.7 ..... 17

Figure 16 - Inland low exposure at Las Pulgas Rd Br 217.32..... 18

Figure 17 - Inland low exposure between Las Pulgas Rd and CP San Onofre ..... 18

Figure 18 – Map of San Diego County..... 21

Figure 19 – Marine Structural Element Exposure Zoning ..... 22

Figure 20 – Inland Structural Element Exposure Zoning..... 23

Figure 21 - 6000 psi concrete mixture with 0% FA for inland moderate exposure ..... 46

Figure 22 - 6000 psi concrete mixture with 15% FA for inland moderate exposure ..... 46

Figure 23 – Inland Low Exposure – Service Life Chart for 4000 psi, 5000 psi, and 6000 psi Concrete Mixtures ..... 48

Figure 24 – Inland Moderate Exposure – Service Life Chart for 4000 psi Concrete Mixtures..... 53

Figure 25 – Inland Moderate Exposure – Service Life Chart for 5000 psi Concrete Mixtures..... 55

Figure 26 – Inland Moderate Exposure – Service Life Chart for 6000 psi Concrete Mixture ..... 56

Figure 27 – Marine Moderate Exposure – Service Life Chart for 4000 psi Concrete Mixtures ..... 59

Figure 28 – Marine Moderate Exposure – Service Life Chart for 5000 psi Concrete Mixtures ..... 60

Figure 29 – Marine Moderate Exposure – Service Life Chart for 6000 psi Concrete Mixtures ..... 61

Figure 30 – Marine Severe Exposure – Service Life Chart for 4000 psi Concrete Mixtures ..... 64

Figure 31 – Marine Severe Exposure – Service Life Chart for 5000 psi Concrete Mixtures ..... 66

Figure 32 – Marine Severe Exposure – Service Life Chart for 6000 psi Concrete Mixtures ..... 68

Figure 33 – Marine Severe Exposure – Service Life Chart for 5000 psi Concrete Mixture with 15% FA and 5% SF ..... 69

Figure 34 – Marine Severe Exposure – Service Life Chart Displaying the Chloride Ingress by Depth for 5000 psi Concrete Mixture with 15% FA and 5% SF ..... 69

Figure 35 – Marine Severe Exposure – Service Life Chart Displaying Chloride ingress by Time at 2 in. and 3 in. cover depth for 5000 psi Concrete Mixture with 15% FA and 5% SF ..... 70

Figure 36 – Marine Severe Exposure – Service Life Chart for 5000 psi Concrete Mixture with 20% Slag and 5% SF ..... 70

Figure 37 – Marine Severe Exposure – Service Life Chart Displaying the Chloride Ingress by Depth for 5000 psi Concrete Mixture with 20% Slag and 5% SF ..... 71

Figure 38 – Marine Severe Exposure – Service Life Chart Displaying Chloride ingress by Time at 2 in. and 3 in. cover depth for 5000 psi Concrete Mixture with 20% Slag and 5% SF ..... 71

Figure 39 – Marine Severe Exposure – Service Life Chart for 5000 psi Concrete Mixture with 20% FA and 5% SF ..... 72

Figure 40 – Marine Severe Exposure – Service Life Chart Displaying the Chloride Ingress by Depth for 5000 psi Concrete Mixture with 20% FA and 5% SF ..... 72

Figure 41 – Marine Severe Exposure – Service Life Chart Displaying Chloride ingress by Time at 2 in. and 3 in. cover depth for 5000 psi Concrete Mixture with 20% FA and 5% SF ..... 73

Figure 42 – Marine Severe Exposure – Service Life Chart for 5000 psi Concrete Mixture with 35% Slag and 5% SF ..... 73

Figure 43 – Marine Severe Exposure – Service Life Chart Displaying the Chloride Ingress by Depth for 5000 psi Concrete Mixture with 35% Slag and 5% SF ..... 74

Figure 44 – Marine Severe Exposure – Service Life Chart Displaying Chloride ingress by Time at 2 in. and 3 in. cover depth for 5000 psi Concrete Mixture with 35% Slag and 5% SF ..... 74

Figure 45 – Marine Severe Exposure – Service Life Chart for 6000 psi Concrete Mixture with 25% FA and 5% SF ..... 75

Figure 46 – Marine Severe Exposure – Service Life Chart for 6000 psi Concrete Mixture with 50% Slag and 5% SF ..... 75

Figure 47 – Marine Severe Exposure – Service Life Chart for 5000 psi Concrete Mixture with 10% SF..... 76

Figure 48 – Marine Severe Exposure – Service Life Chart Displaying the Chloride Ingress by Depth for 5000 psi Concrete Mixture with 10% SF ..... 76

Figure 49 – Marine Severe Exposure – Service Life Chart Displaying Chloride ingress by Time at 2 in. and 3 in. cover depth for 5000 psi Concrete Mixture with 10% SF ..... 77

Figure 50 – Marine Severe Exposure – Service Life Chart for 4000, 5000, and 6000 psi Concrete Mixture ..... 78

Figure 51 – Marine Severe Exposure – Service Life Chart for 4000 and 6000 psi Concrete Mixture with 5%SF ..... 79

**Index of Tables**

Table 1 - Structural Design Life Recommendations in years. .... 3

Table 2 – Chloride Exposure Zones ..... 19

Table 3 – Sulfate Exposure in Zones ..... 20

Table 4 – Concrete Mixture Properties with Type II/V and Type I Portland Cement ..... 25

Table 5 – Concrete Mixture Properties with Type II/V Portland Cement and 15% Type F Fly Ash ..... 25

Table 6 – Concrete Mixture Properties with Type II/V Portland Cement and 20% Slag..... 26

Table 7 – Concrete Mixture Properties with Type II/V Portland Cement and 20% Type F Fly Ash ..... 26

Table 8 – Concrete Mixture Properties with Type II/V Portland Cement and 35% Slag..... 27

Table 9 – Concrete Mixture Type II/V Portland Cement and 25% Type F Fly Ash ..... 27

Table 10 – Concrete Mixture Properties with Type II/V Portland Cement and 50% Slag..... 28

Table 11 – Concrete Mixture Properties with Type II/V Portland Cement, 15% Type F Fly ash and 5% Silica Fume ..... 28

Table 12 – Concrete Mixture Properties with Type II/V Portland Cement, 20% Slag and 5% Silica Fume. 29

Table 13 – Concrete Mixture Properties with Type II/V Portland Cement, 20% Type F Fly ash and 5% Silica Fume ..... 29

Table 14 – Concrete Mixture Properties with Type II/V Portland Cement, 35% Slag and 5% Silica Fume. 30

Table 15 – Concrete Mixture Properties with Type II/V Portland Cement, 25% Type F Fly ash and 5% Silica Fume ..... 30

Table 16 – Concrete Mixture Properties with Type II/V Portland Cement, 50% Slag and 5% Silica Fume. 31

Table 17 – Concrete Mixture Properties with Type II/V Portland Cement and 10% Silica Fume ..... 31

Table 18 – Corrosion Threshold and Propagation Time for Reinforcement..... 38

Table 19 – Chloride Threshold for Cementitious Concrete Materials with CNI <sup>1,2</sup> ..... 39

Table 20 – Additional Corrosion Protection Components ..... 45

Table 21– Minimum Cover to Mitigate Sulfate Attack Degradation at 100-years with Relative Humidity 49

Table 22 – Minimum Cover to Mitigate Sulfate Attack Degradation at 100-years with Relative Humidity ..... 50

Table 23 – Minimum Cover to Mitigate Sulfate Attack Degradation at 100-years at a humidity of 100%.51

Table 24 – Minimum Cover to Mitigate Sulfate Attack Degradation at 100-years ..... 57

Table 25 – Minimum Cover to Mitigate Sulfate Attack Degradation at 100-years ..... 62

Table 26 – Maximum Chloride Ion Content for Corrosion Protection..... 81



## 1 Corrosion Prevention

### 1.1 General

The corrosion of embedded steel reinforcement in concrete due to the penetration of chlorides from deicing salts, groundwater or seawater is the most prevalent form of premature concrete deterioration worldwide and costs billions of dollars a year in terms of infrastructure repair and premature replacement. If we can extend the service life of structures along the LOSSAN Corridor from 75 to 100 years, then it has been estimated that future generations will save \$200 million in today's terms.

The Durability and Corrosion Prevention section of the LOSSAN Rail Corridor Design Criteria Manual shall apply to all new structures built in the Corridor, new stations and any new support facilities like parking garages. The LOSSAN Railway runs down the Southern Californian Coastline, passes over many coastal lagoons, rivers, creeks and through geological formations that were once under the sea. The climate is temperate and the structures are subject to varying exposure conditions. The goal of this Guide is to provide sufficient information at a program management level so that designers are able to consistently design new structures that cross streams, rivers and tidal lagoons that are serviceable and will require minimal repairs or maintenance for up to a 100-year service life. These Service Life Design Guidelines present a methodology for service life engineering of new structures.

Structures can be exposed to many degradation mechanisms that, over their service life, may require costly repairs if not adequately designed for. The degradation mechanisms specific to San Diego County will be discussed in this chapter, as will outline specification requirements, corrosion protection systems and durability enhancements to mitigate these degradation mechanisms.

Several strategies will need to be considered to achieve the design service life of reinforced and prestressed concrete structures exposed to chloride ions, and may include the use of the following:

- low-permeability or high-performance concrete with appropriate cover
- corrosion inhibiting admixtures for concrete with appropriate rebar cover
- protective coatings or treatment to steel reinforcement (e.g., epoxy-coated or galvanized steel)
- non-ferrous reinforcement (e.g., fiber-reinforced plastics)
- waterproofing membranes or sealers applied to the exposed surface of the concrete
- allowing for additional thickness of materials
- cathodic protection, and/or
- combinations of the above

The selection of the most appropriate strategy will be left up the Project Development Teams and shall be based on a cost benefit analysis prepared during the final design stage of the project.

#### Training

Consultants will be trained to use the design methodology presented in this Service Life Design Guide for Concrete Structures. SANDAG will provide training seminars presented by Tourney Consulting Group.

#### Note from the Authors

This chapter of the design criteria was completed under normal project constraints for service life assessment, using the latest modelling techniques and codes of practice and the information provided is believed to be representative of the concrete industry practices, technology, and standards as they exist

today. The design life predictions are based on un-cracked sections of concrete under normal operating conditions in the climate and environmental exposure conditions found along the San Diego coast.

This is a controlled document and information contained within this document is not to be altered except with the written approval of Tourney Consulting Group, LLC and SANDAG.

This chapter contains a Durability and Service Life Analysis that has been conducted using estimated material properties and test values, and based on exposure conditions specific to San Diego County, California. Actual results may vary depending on actual material properties.

## 1.2 Standards, Codes and Guidelines

The latest edition of the following standards, codes and guidelines shall also be used for corrosion prevention of structures. However, the STADIUM modelling techniques presented in this chapter are generally more detailed than the following references:

1. ACI 201.2R Guide for Durable Concrete.
2. ACI 212.3R-10 Report on Chemical Admixtures for Concrete, *Chapter 13 Corrosion Inhibiting Admixtures*.
3. ACI 304R-00 Guide for Measuring, Mixing, Transporting, and Placing Concrete.
4. ACI 308.1-11 Specification for Curing Concrete.
5. ACI 318 Building Code for Structural Concrete.
6. ACI 365.1R-00 Service Life Prediction.
7. ISO 8044-89 Corrosion of Metals and Alloys
8. ACI 515.2R-13 Guide to Selecting Protective Treatments for Concrete.
9. AREMA Manual of Railway Engineering.
10. ASTM Standards Volume 01.04, *Iron and Steel Products, Steel-Structural, Reinforcing, Pressure Vessel, Railway*, ASTM International, W. Conshohocken, PA.
11. CALTRANS Corrosion Guidelines, Division of Engineering Services, November 2012.
12. CALTRANS Bridge Design Specifications, September 2003, Section 8 Reinforced Concrete.
13. CALTRANS Protection of Reinforcement against Corrosion Due to Chlorides, Acids, and Sulfates, Memo to Designers, 10-5, January 2002.
14. CALTRANS Highway Design Manual, Section 850, 2011.
15. PCI Tolerance Manual for Precast and Prestressed Concrete Construction, MNL 135-00. Precast/Prestressed Concrete Institute.
16. A. Bentur, S. Diamond, and NS. Berke, *Steel Corrosion in Concrete—Fundamentals and Civil Engineering Practice*, E & FN SPON, 1997.

## 1.3 Design Life

The required design life of a structure is the utilization period specified by the owner or SANDAG, with respect to structural safety, serviceability and durability. That is the period for which the structure is to be used for its intended purpose with anticipated maintenance but without major repair being necessary. SANDAG requirements for design lives of transit and rail related structures are made by subdividing the structures according to their use and importance level as shown in Table 1;

**Table 1 - Structural Design Life Recommendations in years.**

Usage	Local or Regional Importance Level 1	Interstate or Intercity Importance Level 2
General Infrastructure like Bridges and Parking Structures	75	100
Buildings & Stations	75	100
Industrial Infrastructure & Rail Yards	50	75

**Usage** can be categorized as follows;

- General Infrastructure is classified as works used by the general public, including mainline railways and interstate highway works.
- Buildings are structures intended for public, residential and or commercial use.
- Industrial Infrastructure is classified as works in the service of a particular industrial installation or maintenance type facility.

**Importance** or security levels can be categorized as follows;

- Level 1 Importance is defined as works of local and regional interest, with a small risk of loss of human life or environmental damage in the case of failure. Including work in branch lines, urban trolley networks, local roads and road bridges.
- Level 2 Importance is defined as works of interstate or intercity interest, with an elevated risk of human loss or environmental damage in the case of failure. Including structures on interstate freeways and intercity railways like bridges, culverts and retaining walls.

### 1.4 Service Life Design Using Introduction to STADIUM

The corrosion protection systems presented in this chapter for concrete structures, have been evaluated through modelling developed for the U.S. Navy and others, and should provide the anticipated service life for new structures. This chapter presents the Design Methodology, which can be used for new concrete structures in the LOSSAN Rail corridor in San Diego County. The goal is to be able to design new concrete structures for up to an estimated 100-year service life, where service life is defined as the time to the initiation of corrosion of the reinforcement.

It should be noted that most degradation mechanisms, like chloride diffusion, are influenced by temperature and that these guidelines cannot be used for other geographic locations where different climatic conditions prevail. The service life of structures was modelled using a software called STADIUM® which was originally developed in the late 1990s and uses time-step finite element analysis to simulate the ingress of harmful ions (including chloride, sulphate, and hydroxide) through concrete, by considering the chemical and physical properties of the concrete being studied. The modelling program was developed using funding from the U.S. Navy, which needed a tool to accurately predict the service life of waterfront concrete structures at its many Naval bases around the world. For more information see Appendix B.

## 1.5 Degradation Mechanisms

Several concrete degradation mechanisms must be addressed in order to obtain a 100-year service life. Almost universally, concrete degradation mechanisms are related to the ingress of moisture and ions. All the degradation mechanisms common in the warm, humid, coastal regions produce cracks in the concrete which then accelerates moisture and chemical ingress and further distress. The durability challenge is to mitigate the multiple degradation mechanisms described below while optimizing cost and performance.

The most common degradations mechanisms that are seen in Southern California are;

- Chloride induced corrosion,
- Carbonation induced corrosion,
- Sulfate attack,
- Alkali-silica reaction, and
- Seawater scaling.

Chloride induced corrosion is the primary degradation mechanism that impacts the service life of concrete structures in the LOSSAN Rail Corridor. Chloride induced corrosion is mitigated by providing a corrosion protection system and a majority of the design methodology presented in this Design Guide deals with mitigating chloride induced corrosion.

### 1.5.1 Chloride Induced Corrosion

Corrosion of reinforcement is the most prevalent and costly degradation mechanism for concrete structures in coastal environments. Normally, reinforcing steel is protected by a protective film of passive iron hydroxide surface film that forms naturally in the alkaline environment inside Portland cement concrete. However, this protection can be destroyed when aggressive contaminants such as chloride ions penetrate into the concrete. The chloride ions (Cl<sup>-</sup>) ingress into the concrete and once a certain concentration threshold is reached, de-passivation of the protective oxide film occurs. This is initially a highly localized form of corrosion, known as pitting. Figure 1 shows the result of chloride induced corrosion in a bridge that was widened in 1952. The sixty-year-old structural beam needs to be replaced.



Figure 1: Chloride Induced Corrosion

Once this protective film is destroyed, the reinforcing steel begins to corrode. The pits coalesce and produce expansive corrosion by-products - "rust." The "rust" occupies a volume that is between 4 to 7 times the volume of the steel that has been consumed. Corrosion occurs over a period of time without any outward sign of damage to the concrete until the volume of "rust" is greater than the volume of the pore structure around the reinforcement. Eventually, the iron oxide expansion generates sufficient tensile stress to crack the concrete and cause spalling as shown in .

### 1.5.2 Carbonation Induced Corrosion

Carbon dioxide (CO<sub>2</sub>) diffusing through concrete reacts with Calcium hydroxide and with alkalis in the pore solution to reduce the pH level from a high alkalinity value of approximately 13, to a more neutral value of less than 9. This reduced pH value results in de-passivation of the concrete around the reinforcement, or loss of the naturally occurring protective oxide film.

The rate at which the carbonization front progresses depends on the diffusion rate of the CO<sub>2</sub> through the concrete pore structure. Reinforcement can corrode in the presence of moisture and oxygen, because the naturally occurring protective oxide film around the reinforcement has been destroyed.

One method of testing a structure for carbonation is to core drill a hole in the structure and then treat the freshly cut surface with phenolphthalein indicator solution. This solution will turn pink when in contact with alkaline concrete, making it possible to see the depth of carbonation.

Figure 2 shows ¾" of carbonation in pier 3 of the 95-year-old railway bridge over the San Luis Rey River in Oceanside California. The amount of carbonation ranged from 3/8" to 1.5" in the 95-year-old bridge piers.



Figure 2 - Carbonation depth being measured in a specimen

### 1.5.3 Sulfate Attack

Sulfate contamination from ground water, gypsum-contaminated soils, brackish water, and chemical processes causes expansion of the cement paste to occur. Sulfate attack occurs when external sulfate ion combines with aluminate phases to form ettringite in the presence of moisture. Ettringite crystal expansion produces progressive failure, commonly from the exposed surface inward. This distress relates to a loss of concrete cover over the reinforcing system, reference Figure 3. This type of deterioration has not been seen recently along LOSSAN in San Diego County.

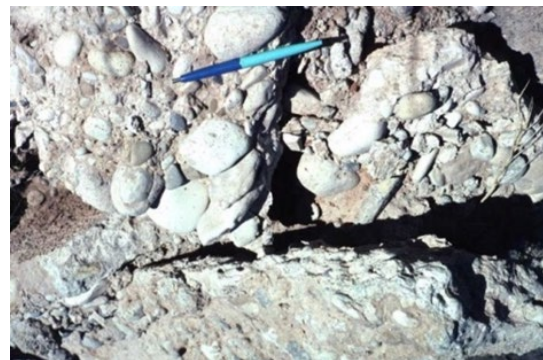


Figure 3 - Sulfate attack in concrete

### 1.5.4 Alkali-Aggregate Reaction

Certain minerals commonly found in concrete aggregates chemically react with the alkali hydroxides in cement. There are two primary classifications: alkali-silica reaction (ASR), and alkali-carbonate reaction (ACR). Alkali-silica reaction is the more prevalent form of distress.

The alkali hydroxides in cement paste, dissolve amorphous or poorly crystalline silica, in the ASR-susceptible aggregate and produces a water-soluble gel that expands when it comes into contact with water. This swelling gel first fills cracks and voids in the aggregate and cement paste until there is no further room to expand.

Further gel expansion develops internal stress in the concrete and causes cracking in both the paste and aggregate. As such, ASR distress is characterized by a period of time with little outward sign of distress, followed by progressive cracking with gel exudation, followed by spalling and deterioration of the concrete. ASR may reduce the expected service life of a structure. The ASR chemical reaction occurs at various rates depending on the combination of cementitious materials, aggregate minerals, temperature, and exposure to water.



Figure 4 - Alkali aggregate reaction in Br 225.3 San Luis Rey River from 1916

We can mitigate for Alkali-aggregate reaction by identifying and eliminating reactive aggregates during the mix design and materials selection stage of a project.



If an aggregate is found to be reactive, the proposed concrete mixture can be modified by replacing a portion of the cement with supplementary cementitious materials, and/or using a lithium nitrate admixture, both of which help mitigate ASR.

Once a structure is constructed and is found to be suffering from ASR, there is very little that can be done other than topical treatment or injection with lithium nitrate, which may slow the expansive reaction in the treated area.

Figure 5 - Evidence of ASR in Br 225.3 pier 2

### 1.5.5 Seawater Attack

Seawater attack is the phenomena associated with the gradual deterioration of the concrete in seawater. This distress is characterized by increased porosity and flaking of the surface in and below the tidal zone as shown in Figure 6 of a 100-year-old railway culvert outfall built in 1910 on the beach in Del Mar at MP 244.3. This distress is a complex process involving a combination of calcium leaching, sulfate attack, salt crystallization distress, and microbial attack. Sulfate attack mitigation techniques are also effective against seawater attack.



Figure 6 - 100-year-old railway culvert outfall built in 1910 on the beach in Del Mar at MP244.3

### 1.5.6 Summary

A Field Investigation Program was conducted to determine what degradation mechanisms concrete structures in the LOSSAN Rail Corridor are exposed. Based on this program the concrete structures in the LOSSAN Rail Corridor are primarily exposed to four forms of degradation:

- Chloride induced corrosion,
- Carbonation induced corrosion,
- Sulfate attack, and
- Alkali-aggregate reaction.

This Design Guide presents a Design Methodology to mitigate these degradation mechanisms.

Corrosion protection systems have been developed to mitigate these degradation mechanisms and predict up to a 100-year service life for four different zones of exposure. There are many corrosion protection system options and the following are considered in this Design Guide, such as:

- Black and or epoxy coated reinforcement,
- Depth of cover to reinforcement,
- Fly ash and or slag,
- Corrosion Inhibitors, and
- Silica fume.

Alkali-silica reaction (ASR) can be evaluated by testing the proposed aggregates and cement to determine the potential for reactivity, per Standard Test Method ASTM C1260. Supplemental cementitious materials can be used to mitigate alkali-silica reaction when aggregate found to have the potential for reactivity with the cement. The combination of cement, proposed SCM, and reactive aggregate should be evaluated by Standard Test Method ASTM C1567 to determine the required amount of SCM to mitigate the ASR. Refer to CALTRANS for current limits for expansion.

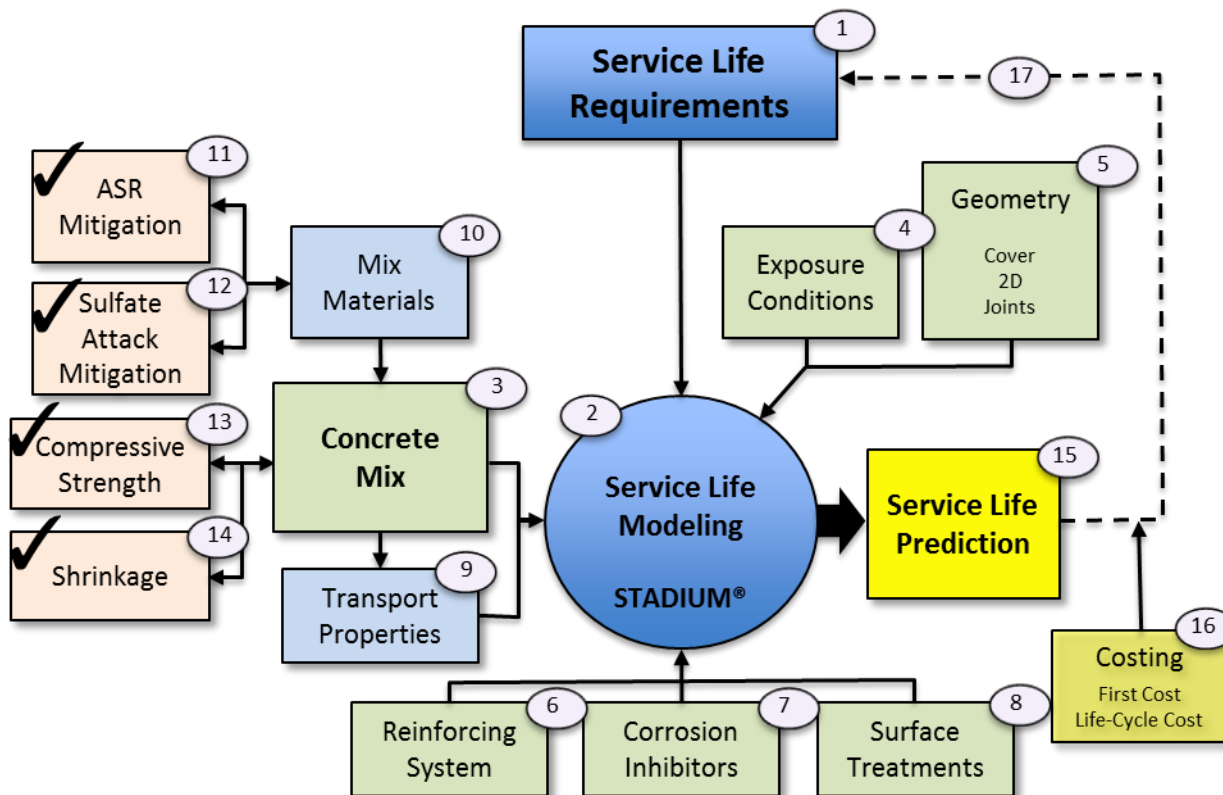
The depth of degradation of the concrete due to sulfate ingress is addressed by using sulfate resistant cements, adding supplementary cementitious materials (SCM's), such as fly ash, slag or silica fume, and by increasing concrete cover over the reinforcing bars or by using protective coatings. Even with sulfate resisting cements and SCM's, there still is potential for degradation which most service life software packages do not address.

**1.6 Service Life Design Process**

**1.6.1 Service Life Engineering Flowchart**

Service life engineering involves many levels of interacting exposures, conditions, and inputs within a decision-making process. Each part of the service life engineering process has an interacting or dependent role in predicting any future performance of the entire concrete system (concrete, reinforcing, and surface treatments).

Figure 7 presents a flowchart of the typical service life engineering and decision process.



Notes:

1. Freeze-thaw does not need to be considered in San Diego County.

**Figure 7 – Flow Chart**

Each block in the flowchart has importance and this will be explained below. The flowchart blocks are numbered to enhance identification in the description below and are not directly the order of the process steps.

1. **Service Life Requirements** – The first step of the design process is the statement of the service life expectations. This step should also define service life performance, for example: What level of deterioration is acceptable vs. what level is not acceptable? Will maintenance and repair be expected during service, and if so, at what intervals or under what circumstances? These questions should be well understood before any other activities.



2. **Service Life Modeling** – A service life modeling approach can be applied once all the exposure conditions and modeling inputs are identified. The service life modeling software used for SANDAG is the STADIUM® software. It is important that the operator of the software is properly trained and understands the sensitivity of changes of inputs to the software. STADIUM software will be used in an iterative process as various modeling inputs are adjusted to meet economic and performance objectives.
3. **Concrete Mix** – The concrete mix is the cornerstone of the reinforced concrete structural element. The concrete mix proposed for a given project or element will be modeled for service life through STADIUM using determined or estimated transport properties (9). In addition to the transport properties, the proportions of the concrete mix design (10) and the chemistry of all cementitious materials are critical inputs to STADIUM. The concrete mix will require at least four additional durability and structural performance checks (11, 12, 13, and 14).
4. **Exposure Conditions** – The exposure conditions are determined from existing structures, soil chemistry, water chemistry (i.e., fresh, brackish, seawater, etc.), climatic conditions (including seasonal differences and airborne salts), applied conditions (deicing, cleaning, floods, etc.), and duration of the various environments. It is also critical to apply varying exposure conditions as necessary to each element or from one element to another.
5. **Geometry** – The geometry inputs to the modeling is primarily to characterize the depth of the steel reinforcing from the exposed concrete surface (concrete cover). The two-dimensional exposures are more aggressive and more complex to model. Service life engineering may lead to design modifications on construction details or in final concrete cover requirements to address durability expectations.
6. **Reinforcing System** – The reinforcing system / type is important to many infrastructure projects. The risk of corrosion of the reinforcing can be addressed by geometry, protection against harmful chloride exposures (admixtures and surface treatments), and the selection of the type of reinforcing bars (black, epoxy-coated, galvanized, micro composite, non-metallic, or stainless steel).
7. **Corrosion Inhibitors** – The use of concrete corrosion inhibiting admixtures can increase the corrosion threshold of the various reinforcing systems. Calcium nitrite inhibitor is available from multiple sources in the industry and the technology has a well-documented successful performance.
8. **Surface Treatments** – One effective way to extend service life is by placing a partial or complete barrier between some of the exposure conditions and the reinforced concrete. Penetrating sealers are examples of a partial barrier. Epoxy, polyurethane, and other paints / coatings are examples of complete barriers.
9. **Transport Properties** – The primary service life modeling characteristics for a concrete mix are its transport properties. Three laboratory tests are conducted to obtain two key STADIUM modeling inputs (ionic diffusion coefficient (IDC) and moisture transport coefficient (MTC)). The MTC and IDC values are numerical parameters used by the modeling software to quantify moisture and ion ingress and movement through the concrete paste structure.

10. **Mix Materials** - The ingredients used in the actual concrete mix have different roles in the overall concrete durability. These materials should meet all applicable ASTM, CALTRANS and ACI material requirements for quality and consistency.
11. **ASR Mitigation** – Aggregates potentially susceptible to alkali-silica reaction are present in the SANDAG region. Therefore, it is important to understand the influence of ASR distress on service life. The use of aggregates with low susceptibility to ASR are preferred, however; if some reactivity is expected, other mitigation techniques will need to be employed, such as Class F fly ash, slag, silica fume, or lithium-based admixtures.
12. **Sulfate Attack Mitigation** – The sulfate content of soil conditions in the Sand Diego region are relatively high as compared to the average soils across the country. Therefore, the concrete mix should meet all ASTM, ACI, AASHTO and CALTRANS standards to mitigate against sulfate attack.
13. **Compressive Strength** – The strength of the concrete is important for structural requirements. It is often the case, that when long service life performance is required, the designed concrete strength is exceeded in order to achieve the durability performance. Strength is an important requisite property to durable concrete, but strength alone is not a surrogate to durability.
14. **Shrinkage** – Most durability modeling processes consider uncracked concrete. Unfortunately, cracking occurs in most concrete. To minimize the occurrence of cracks, designing lower shrinkage concrete will reduce cracking. The keys to lower shrinkage are to reduce total paste volume. Reducing water additions (beyond necessary hydration and workability issues), use proper design details, and curing techniques.
15. **Service Life Prediction** – Once all the inputs are entered into the STADIUM® service life modeling program and other non-modeling checks have been performed, then a service life simulation is conducted. The predicted service life should be compared to the service life expectations (1). If the prediction exceeds expectations, then this concrete system can proceed as a variable option for SANDAG and/or the contractor to consider. If the predicted service life dramatically exceeds expectations, then refinement to the concrete system can be studied. If the predicted service life falls short of expectations, the concrete system is not acceptable as currently configured and is dropped or further modified to meet project requirements.
16. **Costing** – The cost of the various concrete systems that meet the service life expectations can be studied further for first costs and by life cycle costing measures. Often there is interest in both first and life-cycle costs.
17. **Looping Process** – The overall service life engineering process involves iterations to optimize the various concrete systems for cost and performance.

## 1.7 Field Investigation and Exposure Zones

### 1.7.1 Field Investigation

The existing concrete structures in the LOSSAN Rail Corridor are exposed to varying magnitudes of chloride and sulfate, depending on their environmental exposure location. A testing program was undertaken to sample concrete from existing structures in and along the LOSSAN Rail Corridor to measure actual chloride content and ingress rates. Soil and water samples were also collected to measure chloride and sulfate content. Concrete structures were selected based on their environmental exposure. Concrete core samples were collected at various locations in the LOSSAN Rail Corridor; including San Onofre-Pulgas, Santa Margarita River Bridge, San Luis Rey River Bridge (shown in Figure 8), San Elijo, Sorrento Valley, and Miramar Hill).



Figure 8 – Br 225.3 Sampling



Figure 9 – Br 250.9 Sample location

Concrete samples were extracted from concrete structures in locations where the concrete is continuously exposed to seawater, brackish water, groundwater, and/or soil. The acid-soluble chloride content was measured at incremental depths (0.25, 0.75, 1.25, 1.75, 2.25, 3.75, and 4.5 inches) and reported in parts per million (ppm) chloride by mass of concrete. A chloride content profile was

developed for each location using the incremental acid-soluble chloride content depths, reference Figure 11a.

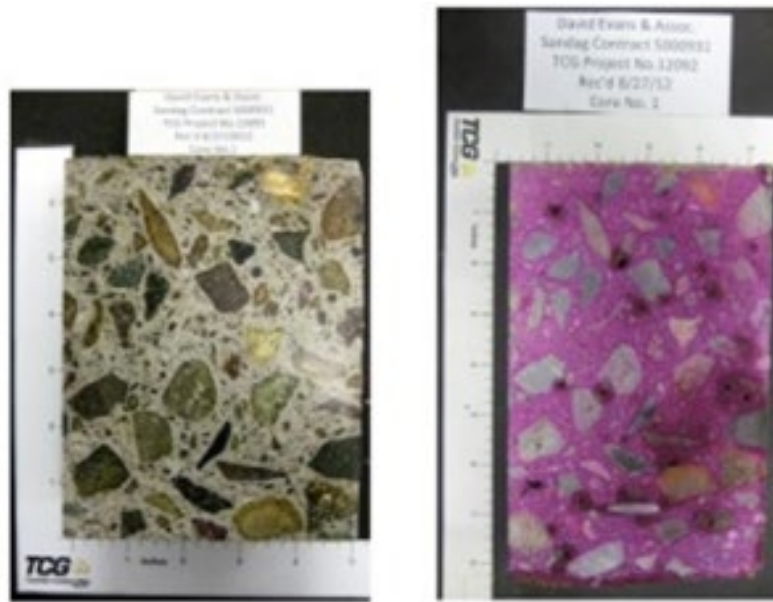


Figure 10 - Test samples

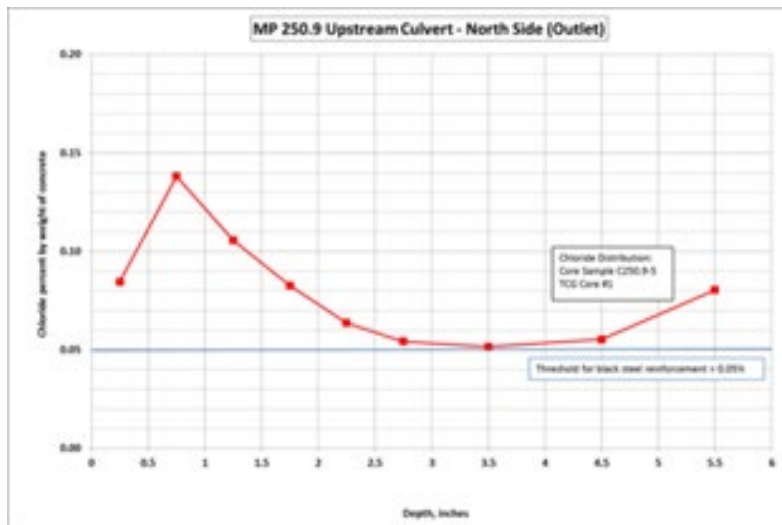


Figure 11a - Chloride profile of the sample

These chloride profiles were used to define the exposure conditions that would generate the same profile used in the Design Methodology for new concrete structures. Four exposure conditions were developed based on the collected field data, for the design of new concrete structures, reference Table 2.

Soil and ground water samples were collected, and the chloride and sulfate content were measured. The measured sulfate content, along with the chloride content was used to further define exposure conditions.

The following Figures 11b and 11c show the measured vertical and horizontal distribution of chlorides from the tidal zone to 80 feet above the seawater and for up to 5 miles inland from the coastline for select locations along the LOSSAN corridor in San Diego.

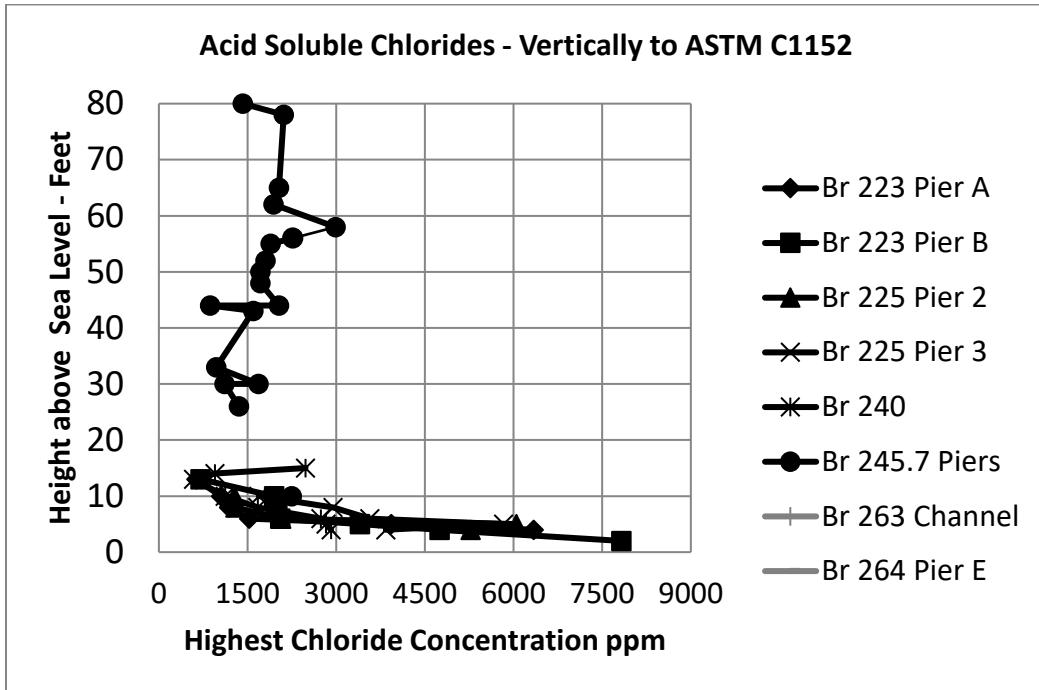


Figure 11b. Vertical Limits of Chloride Contamination – From tidal to 80 feet above mean low water level (MLWL)

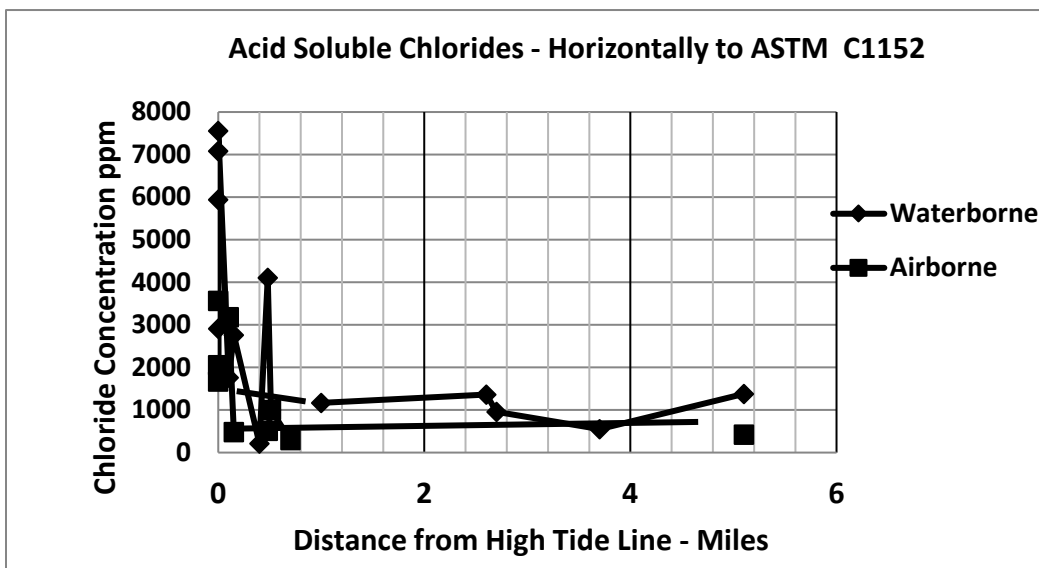


Figure 11c. Horizontal Limits of Chloride Contamination - up to 5 miles from the Coastline

### 1.7.2 Environmental Exposure Condition Zones

The active tectonics of southern California resulted in uplift of coastal land. The southern California coast is known as emergent, meaning that land is being lifted by tectonic forces relative to sea level. The marine deposits in these coastal lands contain chloride and sulfate ions, and the structures in the LOSSAN Rail Corridor can be exposed directly to these marine deposits. Chloride and sulfate ions can leach out of marine deposits and into the groundwater, and the groundwater can expose the structures of the LOSSAN Rail Corridor to these ions.

The chloride and sulfate exposure can vary from an inland exposure to a tidal exposure along the coast of the Pacific Ocean, reference Figure 12 and Figure 13. Chloride exposure is of prime concern in this Design Guide, but design guidelines are also given to address sulfate exposures. The classes of chloride exposure used in the design methodology is based on a pseudo-chloride loading on the surface of the concrete in units of parts per million (ppm). The maximum measured chloride contents are placed in the following four (4) categories in this Guide:

- Marine Severe,
- Marine (Airborne) Moderate,
- Inland Moderate, and
- Inland Low.

With a few exceptions the field data falls into these four categories, reference Figure 18 for map of San Diego County.

### 1.7.3 Marine Severe – Exposure Zone

Structural elements within 10 feet of MLWL, or in the water are in a Marine Severe Exposure Zone. A seawater salinity value of 33.5 ppt defines a Marine Severe Zone. The 10-foot distance should be measured from the mean low water level elevation (MLWL), reference Figure 19. Note that MHHW is at elevation +3.04 feet measured on the NGVD29 datum. All Bridges in Rivers or Lagoons along the LOSSAN Corridor are in this exposure condition, including Telecote Creek and San Diego River bridges because of their proximity to tidal saltwater.

Figure 11d shows the actual measured chloride results in LOSSAN railway bridges. From this data, it is clear that at approximately 10 feet above water level, chloride concentrations decrease dramatically. Hence, structures located in the marine exposure zone can be exposed to both severe and moderate marine exposure. Exposure to waves, wind and proximity to the coastline will all effect the precise height of the splash zone which in turn may move the exposure zone boundary. **This guide is for structures largely in lagoons and river mouths and is not intended for designing piers in the ocean along the exposed coastline.**

Structural elements that are exposed to two different exposure zones should be designed for the worst-case exposure. For example, the bottom portion of a bridge pier may be exposed to a Marine Severe Exposure while the top to a Marine Moderate.

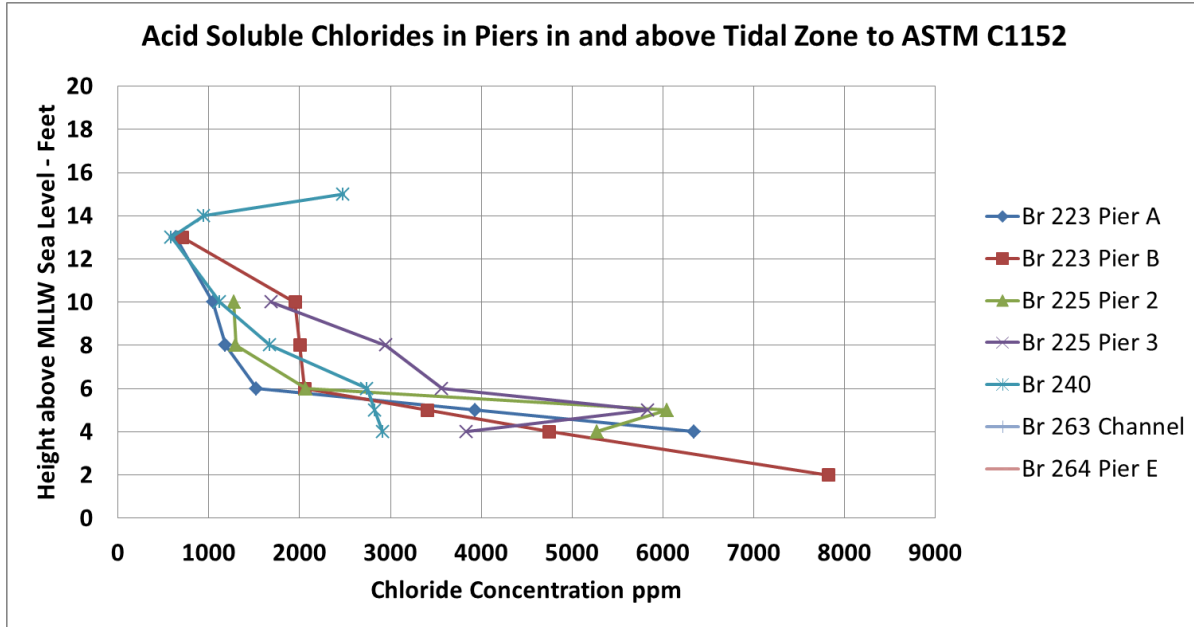


Figure 11d - Measured chloride result in splash zone with height above NGVD DATUM



Figure 12 – San Elijo Lagoon Rail Trestle Br 240.4 substructure in Marine Severe tidal exposure (Photo Shoaff)

### 1.7.4 Marine Moderate – Exposure Zone

The coastal sea breeze coming from the Pacific Ocean transports salt spray inland exposing structures in the rail corridor to chlorides. The corridor varies in distance from on coastal bluffs in Del Mar to 7/10 of a mile from the Pacific Ocean shoreline. Structural elements adjacent to seawater or within a distance of 10 feet and greater above the water are in a Marine Moderate (Airborne) Exposure Zone. A surface chloride ion loading of 4,500 ppm and SO<sub>4</sub> sulfate content of 625 ppm is used to define this exposure, reference Table 2 and Table 3, unless testing determines otherwise. The 10-foot distance should be measured from at mean low water level, MLWL elevation, reference Figure 19. Structures located in the rail corridor from MP 207.4 (San Diego – Orange County Line) to MP 209 near San Onofre Creek and MP 218 and from MP 222 to MP 247.0 (West of the Sorrento Valley Station) and MP 260.5.0 to 264.0 (Balboa Avenue to San Diego River) should be considered in this exposure condition.



Figure 13 – Torrey Pines Overhead Br 245.7 - Marine moderate (Airborne) exposure (Photo Shoaff)

Note: Bridge 218 foundations on Las Flores creek in Camp Pendleton is also in this exposure zone due to the presence of saline creek water.



**1.7.5 Inland Moderate – Exposure Zone**

Structural elements that are exposed to seasonal water conditions and soil are in an Inland Moderate Exposure Zone, such as, foundations and piers for bridges and drainage structures that are exposed to soil, reference Figure 14 and Figure 15. A moderate exposure is defined with a surface chloride ion loading of 3000 ppm and an SO<sub>4</sub> sulfate content in soil of 2.0% by mass and SO<sub>4</sub> dissolved in groundwater of less than 10,000 ppm per Chapter 4 “Durability Requirements” of ACI 318, reference Table 2 and Table 3. SO<sub>4</sub> has been measured at a value as great as 3850 ppm in the ground water and 0.157% (1572 ppm) by mass in the soil. Structures located in the rail corridor from approximately MP 247.0 (West of the Sorrento Valley Station) to MP 263.0, and 264.0 to 267.5 should be considered in this exposure condition, unless testing determines otherwise.



Figure 14 – Sorrento Valley - Inland moderate exposure (Photos Shoaff)



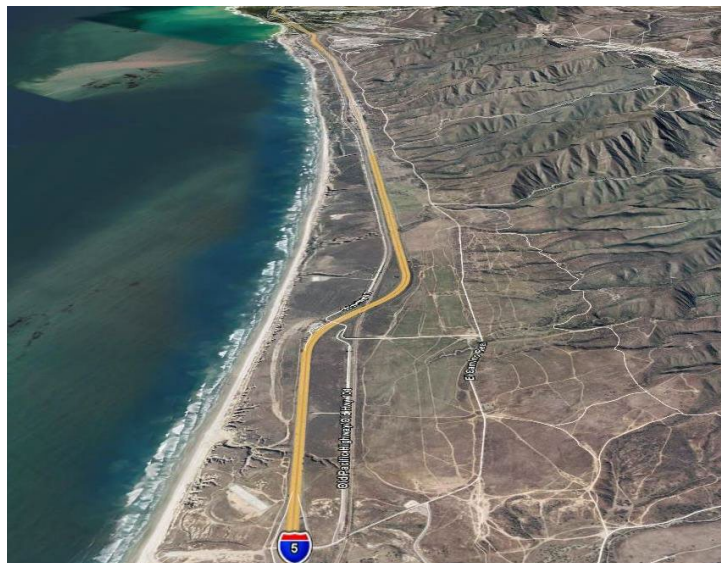
Figure 15 - Inland moderate exposure at Br 248.7

**1.7.6 Inland Low – Exposure Zone**

Structural elements that are not constantly exposed to ground water or soil are in an Inland Low Exposure Zone, such as, the superstructure of bridges, reference Figure 14. A low exposure is defined with a surface chloride ion loading of 1500 ppm and negligible sulfate, unless testing determines otherwise.



**Figure 16 - Inland low exposure at Las Pulgas Rd Br 217.32**



**Figure 17 - Inland low exposure between Las Pulgas Rd and CP San Onofre**

The low exposure will cover an airborne chloride exposure that may occur further inland, but that has a much smaller impact than a marine airborne exposure along the coast. Structures located in the rail corridor from approximately MP 212 to MP 222 and from MP 250 (West of the Sorrento Valley) to MP 260.0, and from MP 264.0 to MP 267.5 should be considered in this exposure condition unless testing determines otherwise.

**1.7.7 Condition Assessments**

Four exposure zones (as noted in the previous sections) have been identified in the LOSSAN Rail Corridor that can be used for service life design. A methodology is presented for assessing field conditions to determine exposure conditions for new construction. A field Condition assessment should be performed to confirm the exposure condition that the new concrete structure belongs in.

An existing structure in the Rail Corridor or an adjacent structure should be selected so that concrete core samples can be extracted. The existing structure, with similar exposure, should be within one quarter mile of the new concrete structure. This will also improve the statistical significance of the data used to determine the boundaries of the exposure zones.

For bridge piers, between three and six specimens should be extracted from the structural concrete at intervals of 2 ft above Low tide level in the most exposed pier, beam or deck up to the top surface of the structure. For drainage channels or culverts, test specimens should be taken adjacent to the edge of water in a zone of seasonal wetting and drying. Include locations exposed to seawater or ground water and airborne.

For example, cores should be extracted from a bridge at the locations of the portion of the pier in-the water and the superstructure above the water and deck to identify chloride loading with height.

Acid-soluble chloride content at increments of 0.25, 0.75, 1.25, 1.75, 2.25, 3.75, and 4.5 inches should be measured. A chloride content profile can be developed, to confirm which exposure zone that the new concrete structure belongs to. Depth of carbonation should also be measured.

One soil and one water sample should also be collected and the chloride and sulfate content measured, to determine the exposure from the soil.

**1.7. 8 Exposure zone summary tables**

The following tables present parameters for each of the exposure zones;

Table 2 – Chloride Exposure Zones

Exposure Zone	Salinity ppt	Maximum Measured Chloride ppm	Stadium Cl loading rate Mmol/L	Temperature (°F)	Humidity (%)
Marine Severe	33.5	8000	522.8	67	100
Marine Moderate	NA	4500	127	67	68
Inland Moderate (Wet)	NA	3000	84.6	67	68 to 100%
Inland Low (Dry)	NA	1500	42.3	67	68

The following table presents sulfate exposure parameters for each of the exposure zones.

Table 3 – Sulfate Exposure in Zones

Exposure Zone	Salinity	Sulfate (SO <sub>4</sub> )	Sodium (Na)	Potassium (K)	Magnesium (Mg)	Temperature	Humidity
	ppt	ppm	ppm	Ppm	ppm	°F	%
Marine Severe	33.5	2574	10254	367	1235	67	100
Marine Moderate	NA	625	4500	89	300	67	68
Inland Moderate (Dry)	NA	3850	1850	20	0	67	68%
Inland Moderate (Wet)	NA	3850	1850	20	0	67	68 to 100%
Inland Low (Dry)	NA	0	0	0	0	67	68

A map of San Diego County is presented in Figure 18 with the approximate locations of the marine and inland exposure zones. It should be noted that Tecolote creek channel at Bridge 263 had the highest Marine Exposure condition measured along the corridor.

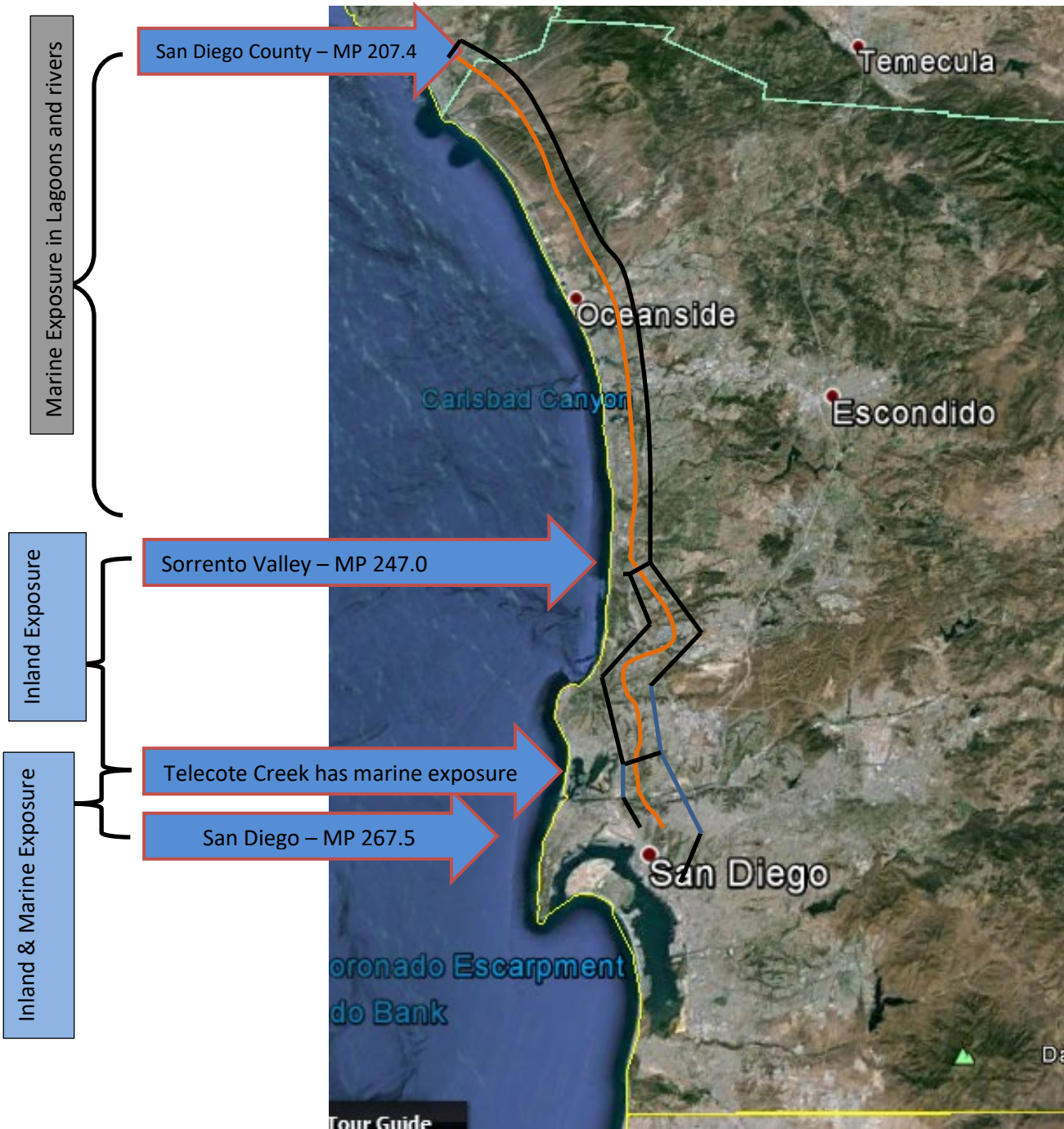


Figure 18 – Map of San Diego County

**1.7.9 Structure Exposure Explanation**

**Marine Structures**

Structures located in the marine exposure zone can be exposed to both severe and moderate marine exposure. Severe exposure means immersed in seawater or in the tidal splash zone. Moderate exposure would be in the airborne zone more than 10 ft above MLWL, reference Figure 19. (Note that the boundary between Marine Severe and Marine Moderate may be adjusted if there is sufficient field coring data from adjacent structures to support moving the boundary. Exposure to waves, wind and proximity to the coastline will all effect the height of the splash zone which in turn may move the boundary. This guide is for structures largely in lagoons and river mouths and is not intended for designing piers in the ocean along the exposed coastline.)

Structural elements that are exposed to two different exposure zones should be designed for the worst-case exposure. For example, the bottom portion of a bridge pier may be exposed to a Marine Severe Exposure while the top to a Marine Moderate, reference Figure 19, so the pier should be designed for the severe exposure. Piers need to be high enough to ensure superstructures are in the Marine Moderate exposure zone.

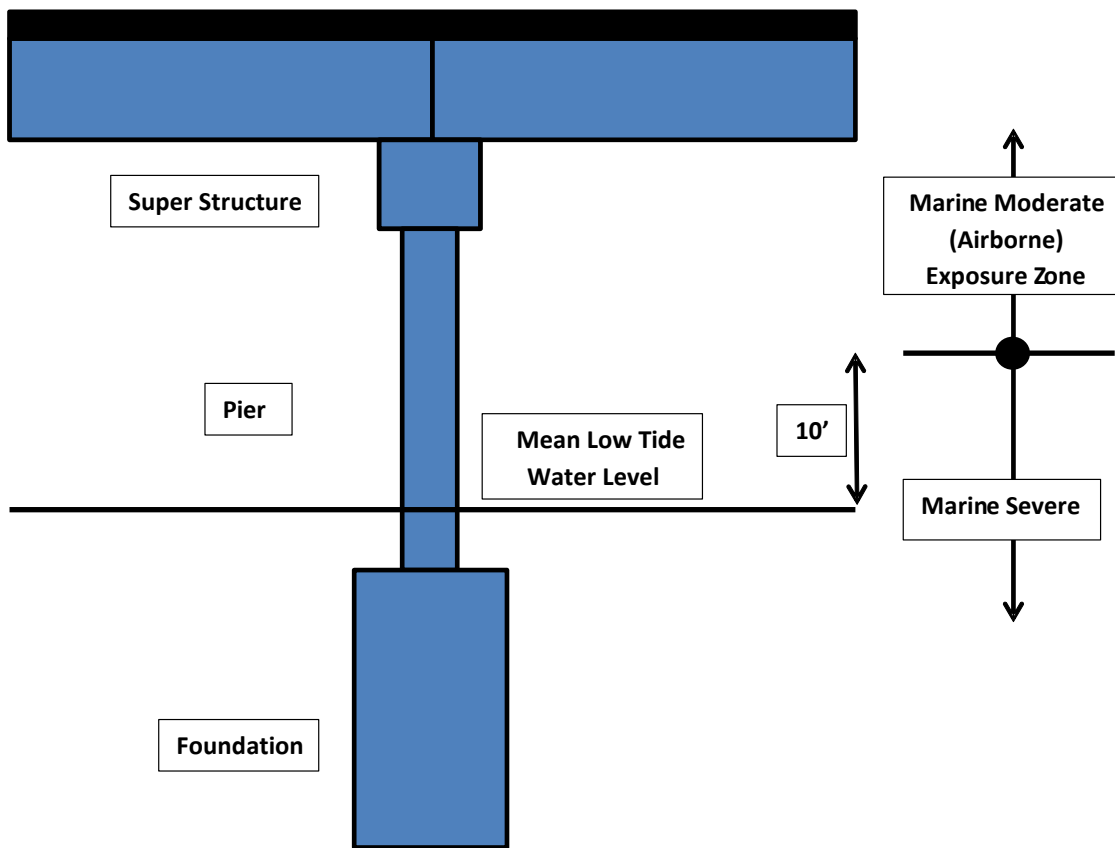


Figure 19 – Marine Structural Element Exposure Zoning

**Inland Structures**

Structures located in the inland exposure zone are divided into structural elements continually exposed to moisture and to structural elements not exposed to moisture, reference Figure 20.

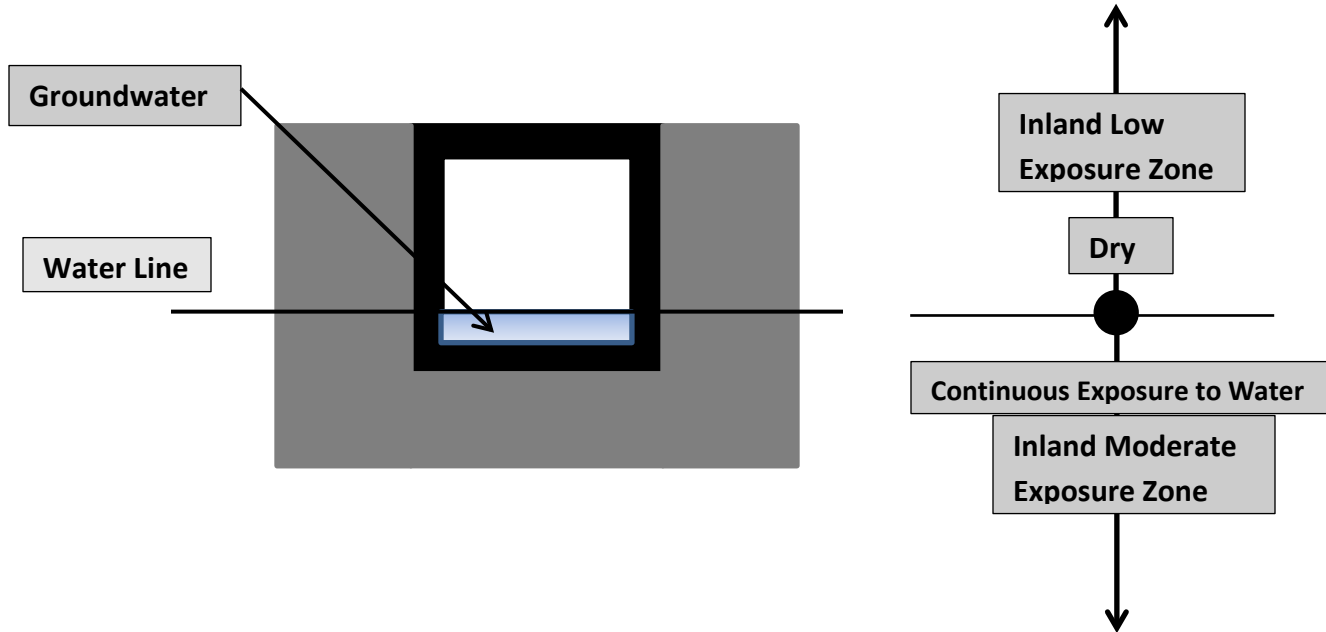


Figure 20 – Inland Structural Element Exposure Zoning

**1.8 Concrete Mixtures**

A durable concrete mixture is just one component of a durable structural concrete element. Durable concrete mixtures may be defined as concrete mixtures capable of resisting weathering action, chemical attack, and abrasion or other environmental exposures while maintaining its intended engineering properties without significant deterioration.

This Guide is based on concrete mixtures with materials and proportions intended to satisfy 28-day specified compressive strengths of 4000, 5000, and 6000 psi. Note that it is critically important to recognize that strength alone is generally not a good indicator of durability, and it is equally important to understand that the requirements for durability may result in concrete strengths well in excess of its structural requirements. Exposure conditions frequently govern over minimum strength requirements in elements designed to resist corrosion deterioration or sulfate attack throughout a defined service life. The use of a low water to cementitious materials (w/cm) ratio and low permeability are critical to the performance of concrete elements exposed to chlorides or sulfates. Concrete meeting an intended mechanical property, such as compressive strength should not summarily be considered durable concrete in and of itself. Most durable concretes will require a minimum level of strength in order to attain requisite durability properties; therefore, this Guide does not recommend designing structural elements with concretes having specified compressive strength below 4000 psi at 28 days. Conversely, concretes proportioned to exceed a specified compressive strength of 6000 psi may be needed depending on the design of the structural element.

Standard concrete mixtures in this Guide contain (i) Class F fly ash at portland cement replacement levels of 15, 20, and 25% by weight, or (ii) slag at replacement levels of 20, 35, and 50%, with and without the inclusion of calcium nitrite corrosion inhibitor. This Guide also addresses concrete mixtures containing silica fume, which may be beneficial under certain conditions of severe exposure.

The materials and proportions used in the manufacture of durable concrete should be selected considering the potential degradation mechanisms. At a minimum, the following material-related degradation mechanisms should be considered for LOSSAN corridor structural concrete elements:

- Reinforcement corrosion,
- External sulfate attack,
- Shrinkage cracking,
- Alkali-silica reaction (ASR).

Different concretes require different degrees of durability depending on the exposure environment and properties desired. For example, an element exposed to tidal seawater will have different requirements than an element exposed to airborne chlorides.

The water-cementitious ratio and cementitious material content and composition are primary factors in the service life that a concrete mixture can provide. Concrete mixtures having water-cementitious ratios of 0.38, 0.44, and 0.50 are the basis of the service life charts. The following tables present the concrete characterization parameters used in the service life analysis to generate the sample service life charts contained in this Design Guide. These concrete mixtures are not recommendations or proposed mixture designs, they simply are illustrative of typical designs that may fall into certain broad classes. The proposed concrete mixture designs will need to have properties that match or outperform one of these concrete mixtures and the generated service life charts can then be used to predict the performance of the proposed concrete mixtures. The concrete mixtures are given the following designations includes the assumed compressive strength, water-cementitious ratio, amount of fly ash, amount of slag, amount of silica fume, and calcium nitrite corrosion inhibitor:

Concrete Mixture Designation: 4000 psi, 5000 psi, and 6000 psi.

Where:

The concrete mixtures have water-cementitious ratio of 0.50 for 4000 psi, 0.44 for 5000 psi, and 0.38 for 6000 psi concrete,

When fly ash or slag is used in the concrete mixture the water-cementitious ratio of is 0.48 for 4000 psi, 0.42 for 5000 psi, and 0.36 for 6000 psi concrete,

FAXX is equal to the amount of fly ash used in the concrete mixture of 15%, 20%, and 25%,

GGBFSXX is equal to the amount of slag used in the concrete mixture of 20%, 35%, and 50%,

SFX is equal to the amount of silica fume used in a concrete mixture of 5%,

CNIX is equal to the amount of corrosion inhibitor in gallons per cubic yard of 2, 3, or 4.



Concrete mixtures presented in Table 4 to Table 17 are the basis of the analysis performed, for the service life charts presented in this Design Guide. These concrete mixtures are not recommendations or proposed mixture designs, they simply are illustrative of typical designs that may fall into certain broad classes.

Table 4 – Concrete Mixture Properties with Type II/V and Type I Portland Cement

Mix Design Designation	4000 psi	5000 psi	6000 psi
Water-to-Binder Ratio	0.50	0.44	0.38
Total Cementitious Content, lbs/cy	560	670	790
Fine Aggregate, lbs/cy	1415	1282	1166
Coarse Aggregate, lbs/cy	1650	1650	1650
OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	17.0	13.5	10.0
Permeability, e-22 m <sup>2</sup> /s	20.0	16.5	13.0
Porosity, %	15.0	14.0	13.0

Table 5 – Concrete Mixture Properties with Type II/V Portland Cement and 15% Type F Fly Ash

Mix Design Description	4000 psi w/15% Fly ash	5000 psi w/15% Fly ash	6000 psi w/15% Fly ash
Water-to Binder-Ratio	0.48	0.42	0.36
Portland Cement Content, lbs/cy	476	569	672
Fly Ash Content, lbs/cy	84	101	119
Total Cementitious Content, lbs/cy	560	670	790
Fine Aggregate, lbs/cy	1408	1271	1148
Coarse Aggregate, lbs/cy	1650	1650	1650
28 Day - OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	14.6	11.7	8.8
Hydration Parameter - a	.30	.29	.28
Ult. - OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	4.4	3.4	2.5
Permeability, e-22 m <sup>2</sup> /s	20.0	16.5	13.0
Porosity, %	15.0	14.0	13.0

Table 6 – Concrete Mixture Properties with Type II/V Portland Cement and 20% Slag

Mix Design Description	4000 psi w/20% Slag	5000 psi w/20% Slag	6000 psi w/20% Slag
Water-to-Binder Ratio	0.48	0.42	0.36
Portland Cement Content, lbs/cy	448	536	632
Slag Content, lbs/cy	112	134	158
Total Cementitious Content, lbs/cy	560	670	790
Fine Aggregate, lbs/cy	1408	1271	1148
Coarse Aggregate, lbs/cy	1650	1650	1650
28 Day OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	11.0	10.0	9.0
Hydration Parameter - a	0.56	0.60	0.60
Ult. OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	6.2	6.0	5.4
Permeability, e-22 m <sup>2</sup> /s	20.0	16.5	13.0
Porosity, %	15.0	14.0	13.0

Table 7 – Concrete Mixture Properties with Type II/V Portland Cement and 20% Type F Fly Ash

Mix Design	4000 psi w/20% Fly ash	5000 psi w/20% Fly ash	6000 psi w/20% Fly ash
Water to Binder Ratio	0.48	0.42	0.36
Cement Content, lbs/cy	448	536	632
Cementitious Replacement, Lbs/cy	112	134	158
Total Cementitious, lbs/cy	560	670	790
Fine Aggregate, lbs/cy	1405	1264	1140
Coarse Aggregate, lbs/cy	1650	1650	1650
28 Day - OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	14.3	11.4	8.5
Hydration Parameter - a	.25	.24	.23
Ult. - OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	3.6	2.7	2.0
Permeability, e-22 m <sup>2</sup> /s	20.0	16.5	13.0
Porosity, %	15.0	14.0	13.0

Table 8 – Concrete Mixture Properties with Type II/V Portland Cement and 35% Slag

Mix Design Description	4000 psi w/35% Slag	5000 psi w/35% Slag	6000 psi w/35% Slag
Water-to-Binder Ratio	0.48	0.42	0.36
Portland Cement Content, lbs/cy	364	435	513
Slag Content, lbs/cy	196	235	277
Total Cementitious Content, lbs/cy	560	670	790
Fine Aggregate, lbs/cy	1405	1264	1140
Coarse Aggregate, lbs/cy	1650	1650	1650
28 Day OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	9.0	8.0	7.0
Hydration Parameter - a	0.58	0.60	0.58
Ult. OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	5.2	4.8	4.1
Permeability, e-22 m <sup>2</sup> /s	20.0	16.5	13.0
Porosity, %	15.0	14.0	13.0

Table 9 – Concrete Mixture Type II/V Portland Cement and 25% Type F Fly Ash

Mix Design Description	4000 psi w/25% Fly ash	5000 psi w/25% Fly ash	6000 psi w/25% Fly ash
Water-to-Binder Ratio	0.48	0.42	0.36
Portland Cement Content, lbs/cy	420	502	593
Fly Ash Content, lbs/cy	140	168	198
Total Cementitious Content, lbs/cy	560	670	790
Fine Aggregate, lbs/cy	1400	1258	1130
Coarse Aggregate, lbs/cy	1650	1650	1650
28 Day - OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	14.0	11.1	8.2
Hydration Parameter - a	.20	.19	.18
Ult. - OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	2.8	2.1	1.5
Permeability, e-22 m <sup>2</sup> /s	20.0	16.5	13.0
Porosity, %	15.0	14.0	13.0

Table 10 – Concrete Mixture Properties with Type II/V Portland Cement and 50% Slag

Mix Design Description	4000 psi w/50% Slag	5000 psi w/50% Slag	6000 psi w/50% Slag
Water-to-Binder Ratio	0.48	0.42	0.36
Portland Cement Content, lbs/cy	280	335	395
Slag Content, lbs/cy	280	335	395
Total Cementitious Content, lbs/cy	560	670	790
Fine Aggregate, lbs/cy	1400	1258	1130
Coarse Aggregate, lbs/cy	1650	1650	1650
28 Day OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	7.0	6.0	5.0
Hydration Parameter - a	0.60	0.60	0.56
Ult. OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	4.2	3.6	2.8
Permeability, e-22 m <sup>2</sup> /s	20.0	16.5	13.0
Porosity, %	15.0	14.0	13.0

Table 11 – Concrete Mixture Properties with Type II/V Portland Cement, 15% Type F Fly ash and 5% Silica Fume

Mix Design Description	5000 psi w/15% Fly ash + 5% Silica Fume
Water-to-Binder Ratio	0.42
Portland Cement Content, lbs/cy	535
Fly Ash Content, lbs/cy	101
Silica Fume Content, lbs/cy	34
Total Cementitious Content, lbs/cy	670
Fine Aggregate, lbs/cy	1170
Coarse Aggregate, lbs/cy	1650
28 Day - OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	2.5
Hydration Parameter – a	1.0
Ult. - OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	1.0
Permeability, e-22 m <sup>2</sup> /s	15.0
Porosity, %	13.0

Table 12 – Concrete Mixture Properties with Type II/V Portland Cement, 20% Slag and 5% Silica Fume

Mix Design Description	5000 psi w/20% Slag + 5% Silica Fume
Water-to-Binder Ratio	0.42
Portland Cement Content, lbs/cy	502
Slag Content, lbs/cy	134
Silica Fume Content, lbs/cy	34
Total Cementitious Content, lbs/cy	670
Fine Aggregate, lbs/cy	1170
Coarse Aggregate, lbs/cy	1650
28 Day OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	2.0
Hydration Parameter - a	1.0
Ult. OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	2.0
Permeability, e-22 m <sup>2</sup> /s	15.0
Porosity, %	13.0

Table 13 – Concrete Mixture Properties with Type II/V Portland Cement, 20% Type F Fly ash and 5% Silica Fume

Mix Design Description	5000 psi w/20% Fly ash + 5% Silica Fume
Water-to-Binder Ratio	0.42
Portland Cement Content, lbs/cy	502
Fly Ash Content, lbs/cy	134
Silica Fume Content, lbs/cy	34
Total Cementitious Content, lbs/cy	670
Fine Aggregate, lbs/cy	1170
Coarse Aggregate, lbs/cy	1650
28 Day - OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	2.0
Hydration Parameter – a	1.0
Ult. - OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	1.0
Permeability, e-22 m <sup>2</sup> /s	15.0
Porosity, %	13.0

Table 14 – Concrete Mixture Properties with Type II/V Portland Cement, 35% Slag and 5% Silica Fume

Mix Design Description	5000 psi w/35% Slag + 5% Silica Fume
Water-to-Binder Ratio	0.42
Portland Cement Content, lbs/cy	402
Slag Content, lbs/cy	234
Silica Fume Content, lbs/cy	34
Total Cementitious Content, lbs/cy	670
Fine Aggregate, lbs/cy	1170
Coarse Aggregate, lbs/cy	1650
28 Day OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	1.5
Hydration Parameter - a	1.0
Ult. OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	1.5
Permeability, e-22 m <sup>2</sup> /s	15.0
Porosity, %	13.0

Table 15 – Concrete Mixture Properties with Type II/V Portland Cement, 25% Type F Fly ash and 5% Silica Fume

Mix Design Description	6000 psi w/25% Fly ash + 5% Silica Fume
Water-to-Binder Ratio	0.36
Portland Cement Content, lbs/cy	632
Fly Ash Content, lbs/cy	118
Silica Fume Content, lbs/cy	40
Total Cementitious Content, lbs/cy	790
Fine Aggregate, lbs/cy	1130
Coarse Aggregate, lbs/cy	1650
28 Day - OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	1.0
Hydration Parameter – a	1.0
Ult. - OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	1.0
Permeability, e-22 m <sup>2</sup> /s	12.0
Porosity, %	12.0

Table 16 – Concrete Mixture Properties with Type II/V Portland Cement, 50% Slag and 5% Silica Fume

Mix Design Description	6000 psi w/50% Slag + 5% Silica Fume
Water-to-Binder Ratio	0.36
Portland Cement Content, lbs/cy	355
Slag Content, lbs/cy	395
Silica Fume Content, lbs/cy	40
Total Cementitious Content, lbs/cy	790
Fine Aggregate, lbs/cy	1130
Coarse Aggregate, lbs/cy	1650
28 Day OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	0.75
Hydration Parameter - a	1.0
Ult. OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	0.75
Permeability, e-22 m <sup>2</sup> /s	12.0
Porosity, %	12.0

Table 17 – Concrete Mixture Properties with Type II/V Portland Cement and 10% Silica Fume

Mix Design Description	5000 psi w/10% Silica Fume
Water-to-Binder Ratio	0.42
Portland Cement Content, lbs/cy	603
Silica Fume Content, lbs/cy	67
Total Cementitious Content, lbs/cy	670
Fine Aggregate, lbs/cy	1210
Coarse Aggregate, lbs/cy	1650
28 Day - OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	2.5
Hydration Parameter – a	1.0
Ult. - OH- Diffusion Coeff., e-11 m <sup>2</sup> /s	1.0
Permeability, e-22 m <sup>2</sup> /s	15.0
Porosity, %	13.0

Hydration, the chemical reaction between the cementitious materials and water in concrete requires both favorable moisture and temperature to occur. Curing is the maintaining an adequate moisture content and temperature in concrete at early ages so that the concrete can develop the designed properties. Finished surfaces should be kept continuously wet or sealed to prevent loss of moisture for a period of at least seven days. Curing should begin immediately after placement and finishing so that the concrete may develop its intended properties.

### 1.8.1 Reinforced Concrete Materials

#### Portland Cement

Selecting appropriate cementitious materials is a critical step in the successful manufacture of durable concrete. Portland cement is the most widely used binding material in the manufacture of concrete. Portland cements are manufactured to meet various standards and optional physical and compositional requirements. ASTM C150, Standard Specification for Portland cement, provides for five principal types of portland cement, designated as follows:

- Type I – Normal
- Type II – Moderate sulfate resistance
- Type III – High early strength
- Type IV – Low heat of hydration
- Type V – High sulfate resistance

AASHTO M 85, Specification for Portland cement, provides for the same five type designations for portland cement. Type V (high sulfate resistance) is the most frequently used cement material in the manufacture of hydraulic cement concrete in California. Use of high sulfate resistant cement is principally due to the severe sulfate exposure zones found in California, where soils or groundwater may have high sulfate content.

For the purpose of this Guide, the terms “Type II/V” and “Type V” will be used interchangeably. Type II/V is a nonspecific term that is often referenced on cement mill reports. Type II/V refers to a portland cement meeting the standard physical and compositional requirements for Type II and Type V.

The high sulfate resistance of Type V cement is attributed to a lowered (maximum 5%) tricalcium aluminate content. However, as a result having a lower tricalcium aluminate content, Type V cement is less efficient in binding intrusive chloride ions. The introduction of one or more supplementary cementitious materials (SCMs) can be beneficial for improving the corrosion-resistance properties of concrete containing Type V portland cement.

#### Blended Hydraulic Cement

ACI defines blended cement as hydraulic cement consisting essentially of an intimate and uniform blend of granulated blast-furnace slag and hydrated lime; or an intimate and uniform blend of portland cement and granulated blast-furnace slag, portland cement and pozzolan, or portland blast-furnace slag cement and pozzolan, produced by intergrinding portland cement clinker with the other materials or by blending portland cement with the other materials, or a combination of intergrinding and blending. Blended hydraulic cement is produced under the ASTM C595 standard.

#### Expansive Cement

Expansive cements are used to counteract shrinkage, an inherent property of materials produced with portland cement. Expansive cement is a hydraulic cement that expands slightly during the early hardening period after initial set. Type K cement can be employed in situations where the shrinkage characteristics of the concrete mixture do not meet project specific requirements. Shrinkage-compensating cement is covered in ASTM C845 (Standard Specification for Expansive Hydraulic Cement).



## Aggregates

Fine and coarse aggregates generally occupy 60%-75% of the concrete volume. Hence, selecting quality aggregates is important in producing good-quality, durable concrete. Generally, aggregates must be clean, hard, strong, and durable. For concrete applications, normal weight aggregates should meet the requirements of ASTM C33; "Standard Specification for Concrete Aggregates." This standard provides specifications related to appropriate gradations, limits on friable particles and clay lumps, sulfate soundness, abrasion resistance, and material finer than the #200 sieve. The standard also notes methods to evaluate the above and to test properties such as freeze/thaw durability and the presence of organic impurities, amongst other parameters.

The specification also comments that aggregates that have been proven to work well in concrete can be considered for use even if some of the physical properties deviate somewhat from the prescribed standard.

One very important aspect of aggregate durability, and therefore concrete durability, is the susceptibility of an aggregate to Alkali-Aggregate Reactivity (AAR). There are two types of AAR. Alkali Carbonate Reactivity (ACR) and Alkali Silica Reactivity (ASR). Generally, aggregates that are susceptible to ACR are not suitable for use in concrete for other reasons. ASR is of more concern because aggregates containing reactive silica materials are more common. In ASR, aggregates containing certain forms of silica will react with alkali hydroxide in concrete to form a gel that swells as it adsorbs water from the surrounding cement paste or the environment. These gels can swell and induce enough expansive pressure to damage concrete. As the concrete is disrupted by these expansive forces, cracking often occurs which provides pathways for the ingress of chlorides which can lead to corrosion or other types of degradation associated with cracks.

Very few aggregates are completely innocuous. As a result, some reactivity is likely to occur in most concretes. However, the concern is when the reactivity is high and the expansion is excessive. A petrographer can examine the aggregates under microscope and identify mineralogies that are susceptible to ASR. Physically testing of the aggregate for reactivity or actual expansion potential is more challenging. Several ASTM test methods for ASR exist and other specifying bodies also have test methods. Some are longer term tests that take several years to conduct. Quicker test methods exist but may alter the aggregates by crushing, add excess alkali to the system, and elevate the temperature to drive any reaction that might occur. Despite these issues, the importance of ASR testing of both the aggregate source(s) and the concrete system is paramount. All aggregate sources used in concrete should have current (no more than 6 months old) ASTM C1260 (or equivalent) results showing that the aggregates do not result in excessive expansion, as defined by the project specifications.

Pozzolans can be used to mitigate the ASR reaction and can be used to eliminate or manage expansion to relatively minor levels. When pozzolans are used to address ASR issues, current (no more than 6 months old) ASTM C1567 test results should be available to show acceptable performance. Finally, given that the mineralogy and quality of aggregates can change as active locations in a quarry or pit changes, prior experience with a given aggregate should not be used as proof of performance.

### 1.8.2 Supplementary Cementitious Materials

SCMs are important materials that contribute to the properties of concrete when used in conjunction with portland cement by reacting either hydraulically or pozzolanically. SCMs addressed in this Guide include fly ash (Class F), ground-granulated blast furnace slag ("GGBFS" or "slag cement"), silica fume, and metakaolin. Depending on the SCM used, benefits derived may include reduced permeability,

control of alkali-aggregate reactivity, lower heat of hydration, higher early or later-age strength, and reduced mix cost.

Pozzolans are siliceous or alumino-siliceous materials that, by themselves, possess no hydraulic (cementing) value, but will, in finely divided form and in the presence of water, chemically react with calcium hydroxide, a non-cementing by-product of cement hydration, to form compounds having cementing properties.

### **Fly Ash**

Fly ash is the spherically shaped amorphous, glassy residue that results from the combustion of pulverized coal in electric generating stations.

During combustion, coal's mineral impurities, predominately silica, alumina and calcium-bearing compounds, fuse in suspension and are carried away from the combustion chamber by the exhaust gases. In the process, the fused material cools and solidifies into both hollow and solid spherical particles. The fly ash is then collected from the exhaust gases by electrostatic precipitators or bag filters. Fly ash is a finely divided powder with an average particle size similar in magnitude to that of portland cement.

ASTM C618 and AASHTO M 295 classifies two types of fly ash – Class F and Class C. Fly ashes generally fall into either the low calcium oxide or high calcium oxide groups depending on the type of coal combusted. Class F fly ashes, derived from combustion of anthracite or bituminous coal, generally contain less than 5% calcium oxide, but may contain as much as 10%. Class C fly ash, derived from the combustion of sub-bituminous or lignite coal, usually contains 20% to 30% calcium oxide, but may contain as little as 10%.

Class F fly ash is the most commonly used SCM in concrete in California. Class F fly ash has been found to improve concrete's resistance to sulfate attack and degradation due to alkali-silica reaction significantly better than Class C fly ash. As a result, this Guide does not recommend the use of Class C fly ash. Class F fly ash is beneficial for improving the corrosion-resistance properties of concrete containing Type V portland cement.

### **Silica Fume**

Silica fume is the ultra-fine non-crystalline silica produced in electric arc furnaces as a byproduct of the production of silicon metals and ferrosilicon alloys. Silica fume is generally dark gray to black in color. Most of the silica fumes used in concrete contain 85 to 95% amorphous SiO<sub>2</sub> in glassy spherical particles. The average particle size ranges from 0.1 - 0.3 μm (approximately 100 times smaller than portland cement particles).

Because of its extreme fineness, silica fume increases the water demand of concrete mixtures. This has made the addition of high-range water reducing admixtures a necessity when silica fume is used. Unless the water demand is offset using a high-range water-reducing admixture, the increase in water necessary to produce needed workability would destroy the properties desired with silica fume. Rather than thought of as a replacement to portland cement, silica fume is commonly used in addition to portland cement.

In high performance concretes, silica fume is typically used at dosages of 5 to 10% by weight of total cementitious materials. An amount of 5% silica fume should be sufficient in most cases, but amounts

greater than 5% should not be ruled out when special circumstances warrant. Silica fume has been found to be extremely beneficial for producing concrete with exceptionally high compressive strength and extremely low to negligible chloride permeability. Silica fume is specified under ASTM C1240 and AASHTO M307.

### **Ground-Granulated Blast Furnace Slag**

Ground-granulated blast furnace slag is the granulated (quenched) glassy by-product of iron production that is dried and ground into an off-white powder similar in size to portland cement. It is a slow reacting latent hydraulic cement considered to have negligible pozzolanic activity. In binary concretes, it typically comprises 20 to 50% of the cementitious material by mass. A multitude of acronyms and terms are used to describe this material, including, ground-granulated blast furnace slag, GGBFS, and slag cement. Slag cement is specified under ASTM C989 and AASHTO M302.

The benefits of concretes produced with slag cement include increased long-term strength and reduced chloride permeability. Slag cement is commonly available in markets in close proximity to steel manufacturing.

### **High-reactivity Metakaolin**

High-reactive metakaolin (HRM) is a highly reactive alumino-silicate with the capability of producing concrete with strength and permeability properties similar to silica fume. Unlike fly ash, slag cement, and silica fume, which are byproducts of major industrial processes, metakaolin is a specifically manufactured SCM. In high performance concretes, metakaolin is typically used at dosages of 8 to 12% by weight of total cementitious materials. In ASTM C 618, metakaolin is classified as a Class N (natural) pozzolan. Metakaolin has a high surface area, so it could require increased dosage rates of the high range water reducer to offset an increase in water demand.

### **1.8.3 Admixtures**

Admixtures are those ingredients in concrete other than Portland cement, pozzolans, water, and aggregates that are added before or during mixing. Admixtures serve an important role in the production and consistency of modern concrete. Specifically, admixtures are primarily used to make the cost of concrete construction more economical and to achieve unique concrete properties and attributes. However, no admixture is a substitute for good concrete practices and quality materials. Admixtures should be compliant with ASTM C494 or ASTM C260. In addition, the concrete producer should have some experience using particular admixtures and admixture combinations with their materials, in their mix designs. The use of admixtures in SANDAG projects is anticipated. The major types of admixtures include the following:

#### **Air Entraining**

These admixtures stabilize the air bubbles that are naturally generated during the mixing action of concrete during production. The presence of these tiny air bubbles has several purposes/benefits. They tend to improve concrete workability and decrease bleeding. In addition, properly sized and spaced air bubbles improve the durability of concrete in sulfate environments, alkali aggregate reactive concrete, and freeze-thaw conditions.

#### **Water Reducing**

One of the keys to concrete durability is water/cement (w/c) ratio. A certain amount of water is needed to hydrate the cement. Water in excess of this amount (termed water of convenience) raises the w/c ratio, which increases paste porosity and degrades durability but improves the workability of the

concrete. Water reducers are cement dispersants that allow the water needed for workability to be reduced or enhancing workability at a given water content. Admixtures of this type range from conventional water reducers and high range water reducers (superplasticizers). The fundamental difference is that amount of water that can be reduced. Conventional water reducer generally allow for a reduction of 5% water while superplasticizers can reduce in excess of 15% water or produce flowing concrete. Some of these admixtures also effect set time so dual effects are often noted (i.e., water reducer / set retarding) in product classifications.

### **Set Controlling**

The length of time that concrete is workable can be modified with admixtures. Set retarders can slow the set time of concrete down to enable more time for placement. Alternatively, since cement hydration is a chemical reaction and speeds up with heat, at higher ambient temperatures, set retarders can slow the set time down to achieve a more manageable workability time. In other circumstances, decreasing the set time (and accelerating strength gain) of concrete may be important and accelerating admixture speed up the set time of concrete.

### **Shrinkage Reducing**

Shrinkage reducing admixtures (SRAs) are added to the concrete primarily to reduce shrinkage due to the loss of water from concrete during drying. In high performance concretes SRAs reduce shrinkage due to the reaction of water with cementitious phases which can result in shrinkage. The mechanism by which they work is a reduction of the surface tension of the capillary water, resulting in less stress as the water leaves the capillary due to evaporation or chemical reaction. The stress increases as the capillary pores become smaller, so SRAs can significantly reduce shrinkage and shrinkage related cracking in higher performance concretes, with smaller capillary pores.

These admixtures are discussed in detail in ACI 212.3R-10 in Chapter 12. They are typically added at 0.75 to 1% by mass of cementitious, but do provide some reduction in shrinkage at even lower levels, so they can be used to reduce shrinkage to an acceptable level for a concrete mixture that demonstrates shrinkage slightly above the maximum allowed.

Other materials which can reduce shrinkage are expansive agents, super absorbing polymers, and lightweight aggregate fines. MgO and CaO of the right size and calcining temperatures will expand when they react in concrete with water without adversely affecting permeability. However, these materials need good water curing as in the case of expansive cement. They can be used in combination with SRAs to significantly reduce shrinkage and cracking.

Super absorbing polymers can absorb several times their weight with water. They can be used in high performance concretes to provide additional water for hydration and thus counter shrinkage stresses. The lightweight fines have a similar behavior as super absorbing polymers. While these materials have been found to be effective in low w/cm concretes, they do not have a major effect on normal drying shrinkage.

### **Corrosion Inhibiting**

One of the greatest service life challenges of concrete is preventing the corrosion of embedded reinforcing steel. Corrosion of steel initiates when several factors occur. One is sufficient chloride concentration at the steel to initial corrosion. Corrosion inhibitors either increase the corrosion threshold of the steel or reduce corrosion rates once corrosion has initiated. Both mechanisms delay

degradation due to corrosion activity. Corrosion inhibitors will be discussed in greater detail in the next section.

## 1.9 Corrosion Protection Systems

### Reinforcement

Various forms of reinforcement are addressed in this section. At the end of the section is a table with chloride threshold values for corrosion initiation and propagation times from corrosion initiation to first observable damage.

### Black Steel

Black steel reinforcing bars are specified by ASTM A615/A615M in Grades 40, 60, and 75. If the bars are to be welded or used in a welded wire or bar mat then ASTM A704/A704M is applicable. Additional specifications for prestressing strand are in ASTM A421. A complete list of standards and methods for black steel products are in ASTM Volume 1.04. Black steel products used should conform to the applicable standards.

As with all steel reinforcing systems, the chloride threshold value for black steel is not an absolute number as it depends on the amount and type of cementitious material, oxygen and moisture content, temperature, and time of exposure. For engineering purposes, the threshold value is chosen to be 500 ppm chloride by mass of the concrete. This is approximately 1.9 pcy for air-entrained concrete, and represents the typical values at which corrosion initiates.

The time to observable damage after corrosion initiates is the propagation time. This time is affected by the same factors that affect initiation rate as well as concrete cover. The value used for black steel is dependent on the amount of cover and varies from four to ten years, reference Table 18. The values in this table are tailored to the County of San Diego and are not applicable to other geographical regions of the USA.

Corrosion thresholds and propagation times are presented in Table 18 for black and epoxy coated reinforcement.

### Epoxy-Coated Steel (ECR)

Specifications for ECR are in ASTM A775/A775M for bars coated before fabrication (green bars), and ASTM A934/A934M for bars coated after fabrication (purple or grey). Additional specifications for wire-welded meshes and other fabricated items are in ASTM Volume 1.04. ECR should conform to these specifications.

The chloride threshold for ECR will depend upon the same factors as black steel and for combinations of ECR and black steel the same threshold as used for black steel holds, namely 500 ppm. If all the steel is ECR then the corrosion threshold can be raised to 900 ppm. The chloride threshold value for epoxy-coated reinforcement is not an absolute number as it depends on the amount and type of cementitious material, oxygen and moisture content, temperature, and time of exposure. ECR should not be used in exposures where the concrete does not go through wet/dry cycles. Propagation time is on the order of 15 years for ECR coupled to black steel and 20 years for all ECR bars.

The use of ECR requires special handling to minimize damage to the coatings in shipping, handling, and placing of the concrete. Damage and cut areas are to be repaired as noted in the specifications. Bars should not be exposed to the environment prior to placement in the forms. If vibrators are used, they are to be plastic coated. Additional information is provided in the specifications and the CRSI Specialty Steel Product Guide.

**Other Corrosion Resistant Reinforcement**

There are other types of reinforcement that offer higher corrosion initiation thresholds that can be considered to enhance the service life: Galvanized reinforcement that comply to ASTM A767/A767M, low chromium micro-composite steel reinforcement that comply to ASTM A1035/A1035M, and stainless steel reinforcement that comply to ASTM A955.

Table 18 – Corrosion Threshold and Propagation Time for Reinforcement

Reinforcing Bar	Threshold <sup>1</sup>		Propagation <sup>1</sup>
	ppm	pcy	Years
Black Steel	500	1.9	6 <sup>2</sup>
ECR <sup>3</sup> (In contact with uncoated reinforcement)	500	1.9	15
ECR <sup>3</sup> (Not in contact with uncoated reinforcement)	900	3.5	20

Note:

1. Corrosion threshold and propagation time are estimated from literature research. Propagation times for black steel are based on field observations and extensive research studies. However, there is not enough history for ECR, so the estimates are best engineering judgment on laboratory test data and limited field experience.
2. Propagation is based on a reinforcement bar size of #4 and is based on a 2 inch minimum cover depth.
3. ECR – Should not be used in a continuously wet environment.

**Corrosion Inhibiting Admixtures**

Corrosion inhibitors are chemical substances that reduce the corrosion of embedded metal without reducing the concentration of the corrosive agents. This definition, paraphrased from ISO 8044-89, makes the distinction between a corrosion inhibitor and other additions to concrete that improve corrosion resistance by reducing chloride ingress into the concrete.

Corrosion inhibitors can influence the anodic, cathodic or both reactions. Since the anodic and cathodic reactions must balance, a reduction in either one will decrease the corrosion rate.

The long-term benefits of calcium nitrite are well documented. When calcium nitrite and other inhibitors are used, maximum w/cm should be 0.45 and the minimum depth of cover to reinforcement should be 1.5 inches. Inhibiting admixtures are produced to comply with the requirements of ASTM C1582 Standard Specification for Admixtures to Inhibit Chloride-Induced Corrosion of Reinforcing Steel in Concrete.

The chloride threshold level to initiate corrosion is a function of the calcium nitrite content. Table 19 shows the chloride protection level as a function of calcium nitrite content. Table 19 was developed from field and laboratory data, and is based on the lowest (most conservative) threshold values found and not averages. The chloride threshold value corresponds to the chloride content at the steel reinforcement surface to initiate corrosion.

Table 19 – Chloride Threshold for Cementitious Concrete Materials with CNI <sup>1,2</sup>

CNI Dose for Concrete Mixture (gal/ cy)	CNI Dose <sup>3</sup> % CNI Solids /Cement	Chloride Threshold	
		Cl <sup>-</sup> lbs/cy	Cl <sup>-</sup> Ppm <sup>4</sup>
2	1.00	6.0	1560
2.5	1.25	8.0	2080
3	1.50	10.0	2570
3.5	1.75	11.5	2990
4	2.00	13.0	3380

Note:

- <sup>1</sup> CNI = Calcium Nitrite Inhibitor
- <sup>2</sup> Source – TCG adapted from Published literature from CNI manufacturers (Sika, BASF & WR Grace) and research by Dr. Neal Berke.
- <sup>3</sup> 1 gal CNI is approximately 3 pounds CNI solids
- <sup>4</sup> Based on concrete unit weight of approximately 3850 pounds per cubic yard (lbs/cy).

Table 19 is used with service life modeling programs, e.g., STADIUM<sup>®</sup>, that predict the chloride level at the reinforcing steel as a function of concrete properties, time, environment and depth of cover. The values are based on the minimum amount of calcium nitrite required to offset a given chloride content, so in probabilistic models they should be used as a deterministic input.

**Surface Treatments**

The primary function of surface treatments is to reduce the ingress of moisture and chlorides. The manufacturer of these surface treatments should provide documentation regarding the benefit of their product with respect to the enhancement provided to the service life of the structural element, since their performance widely vary.

These materials are applied to a surface that has dried to reduce both moisture and chloride ingress. Their effectiveness reduces over time requiring a reapplication, which might not be practical once a structure is in operation. Therefore, in design life calculations only an initial application is considered. Typical times of effectiveness are 5 to 10 years with some organofunctional silanes reaching 15 years. To be effective these treatments require that the concrete go through drying cycles, and as such are not recommended for concrete in the tidal zone or below.

Silanes are applied to a concrete surface will penetrate and react with the cementitious phases to form a water repelling surface that lets vapor out of the concrete. The degree of penetration depends on the molecular size of the silane used, and on how fast it reacts with the cementitious matrix. The 100% silane products are preferred as they provide better results.

Siloxanes are similar to silanes but are larger molecules and as such cannot penetrate as deeply into dense concrete. They are best used with more porous substrates such as concrete block.

Organofunctional silanes have very low viscosities and surface tensions, and are modified not to react as quickly as silanes or siloxanes. As such they can penetrate to deeper depths in good quality concretes. Some have an active inhibitor component.

Silicates are typically used in crystalline modifications of the surface. They are applied to surface saturated dry concrete via brush or low pressure sprayer. The surface needs to remain wet for several days to assist in the penetration. Reactions occur with calcium hydroxide and other phases present in the concrete to form crystalline products to reduce porosity. The resulting concrete will have a lower permeability to water and chloride according to the manufacturers of these proprietary products. Actual enhancement in performance requires determining the transport properties of a treated concrete. In general, penetration and enhanced performance would be expected to decrease as the w/cm decreases.

### **Surface Barriers**

The primary function of surface barriers is to prevent the ingress of moisture, chlorides, and chemicals. The manufacturer of these surface barriers should provide documentation regarding the benefit of their product with respect to the enhancement provided to the service life of the structural element. Surface barriers can be broadly broken down into coating systems and membranes. These act as a barrier to water, moisture and chloride ingress, and as long as they are undamaged and functional, they are very effective in extending time to corrosion initiation. However, they can be difficult to apply initially, and as a subsequent application. Barrier performance is highly dependent on following the application procedures and a failure to do so will result in poor performance. Note for service life modeling it is assumed that the barrier will not be reapplied at failure. Service performance for the surface barrier needs to be provided by the manufacturer and supported by independent test data, as there are numerous coatings and membranes in each category. Typical service life of surface barriers if applied correctly would be 10 to 25 years. Coatings are either brushed or sprayed onto the surface where membranes are applied in sheet form. However, some membranes are liquid applied. In general, if the material is the outside surface, then it is a coating, if it is covered with a more permeable surface for traffic, it is a membrane.

Asphalt or coal tar epoxies are a common coating for concrete. They have a primer coat and finish coat typically applied to a surface-dry concrete. Service life extension will depend on the exposure conditions, and the manufacturer should provide data documenting effectiveness in blocking chloride and water ingress over time. Coal tar epoxies have good resistance to microbiological attack.

Polyurethanes and urethanes can be brush or spray applied and can have varying pot life. They offer good abrasion resistance and can be made UV durable. Cracks can be bridged, but cracks formed after coating could come through.

Epoxy coatings can offer good protection but are susceptible to UV damage.

Polyurea coatings are fast drying and less sensitive to moisture. They are spray applied and this can be performed at high RH. They can be VOC compliant and offer good protection between pH 4 to 12. They are often used with a penetrating primer system.



Methyl methacrylates (MMA) can be applied with squeegees and rollers. They are very effective in reducing chloride ingress into concrete.

Other materials include chlorinated rubber, butadiene copolymers, polyester resin, polyethylene copolymer, acrylics and vinyl's.

### **Concrete Cover**

Concrete cover over the reinforcement is a key element in the service life of the concrete. A two-fold increase in the concrete cover will increase the time to a critical chloride threshold value by approximately a factor of four. In the case of sulfate exposure increased cover will prevent the concrete from deteriorating to a depth of the reinforcement, and thus, maintain a protective environment around the reinforcement.

The minimum cover for corrosion will be a function of the concrete permeability, exposure conditions, time, and supplemental corrosion protection systems. These are determined in the guide for the exposure zones.

Sulfate attack is addressed by using sulfate resistant cement or the addition of fly ash, slag or silica fume. However, ACI 318-10 calls for 3 inches of minimum cover for concrete exposed to earth. This should be the minimum cover for submerged or soil conditions.

### **Cover Tolerances**

The minimum cover should be as noted in the design tables. This needs to address the allowed tolerances for precast and cast-in-place concrete. For example, if the minimum cover is 2 inches and the tolerance is ½-inch, the cover in the design should be 2.5 inches. This addresses the very nonlinear relationship between cover depth and time to a given chloride concentration.

The maximum tolerance for precast concrete is  $\pm 0.25$  inch, per PCI Manual MNL 135-00. The maximum tolerance for cast-in-place concrete is  $\pm 0.5$  inch, per ACI 318 Chapter 7 Section 7.5 "Placing Reinforcement." Tighter tolerances are allowed, especially on the minus end.

### **Placing and Finishing Concrete**

Placing and finishing practices impact the in-place properties of concrete, therefore, they are important steps in the process of producing durable concrete. Concrete should be deposited continuously and as near as possible to its final location without objectionable segregation. Concrete should not be dumped in separate piles and then leveled and worked together; nor should the concrete be deposited in large piles and moved horizontally into final position. Such practices promote mixture segregation and alter the properties. In vertical elements such as walls and columns, or thick horizontal elements, such as deep beams or slabs, concrete should be placed in horizontal layers of uniform thickness. Each layer should be adequately consolidated before the next is placed. The rate of placement should be rapid enough so that previously placed concrete has not yet set when the next layer of concrete is placed upon it, but not so rapidly than it cannot be spread, struck off, consolidated, and floated. Timely placement and adequate consolidation will prevent flow lines, seams, and cold joints. Special considerations are needed when placing concrete underwater.

Concrete surfaces should be finished in ways that are commensurate with the intended service use. Hard troweling is not recommended for air-entrained concrete for several reasons. The American

Concrete institute document ACI 304R-00 “Guide for Measuring, Mixing, Transporting, and Placing Concrete” can be referenced for additional information.

### **Curing Concrete**

Curing affects many major properties of concrete, and it is of particular importance with respect to the transport properties, along with other durability-related properties. This subject is further discussed in the Service Life Design Process. Curing is used to describe the process by which concrete matures and develops hardened properties over time as a result of the continued hydration of the cement paste in the presence of sufficient water and heat. Curing also describes the action taken to maintain moisture and temperature conditions in a freshly placed concrete to allow hydration of the cement paste and, if applicable, pozzolanic reactions to occur so that the potential strength and durability properties of the mixture may develop. The American Concrete institute document ACI 308 “Guide for Curing Concrete” can be referenced for additional information.

### **Control of Cracking in Concrete**

Control of cracking is important, because it is cracking that can lead to poor service life performance of a concrete structure. Cracking in concrete is attributed to many factors, including autogenous shrinkage, carbonation shrinkage, drying shrinkage, thermal, restraint, flexural loads, and tension loads. Cracking attributed to drying, thermal, and design conditions and parameters are discussed below.

### **Drying**

Drying shrinkage should be of concern during curing of concrete that has been placed. Freshly placed concrete should be protected from drying until final finishing using suitable methods, including misting and fogging the surface, application of evaporation retardants to the surface, or erection of windbreaks in order to avoid plastic shrinkage cracking. Drying shrinkage is related to the evaporation of water from the concrete after hardening. With respect to material properties, the magnitude of this volume change is a function of the volume of aggregate and paste, the w/cm, and the aggregate properties. If shrinkage is restrained, drying shrinkage cracks can develop.

Methods to mitigate the influence of drying shrinkage cracks include reducing sources of restraint, providing appropriately placed control joints and construction joints, reducing the total cementitious materials content, keeping the w/cm between 0.38 and 0.45, and / or use of shrinkage reducing admixtures.

Autogenous shrinkage cracks often negate the low permeability of high-strength concrete that is proportioned with a high cementitious materials content and a very low w/cm. Elements produced with mixtures designed to yield very high strength or very low permeability to the ingress of chloride ions could be more susceptible to cracking. Although this type of high strength concrete is believed to be of high quality, if it cracks, the cracked element may have reduced durability. A better solution may include reducing the cement paste volume and providing sufficient water for hydration as the keys to minimizing drying shrinkage cracking.

### **Thermal**

During the first few days after placement, concrete undergoes changes in temperature at a time when it has little strength. As the cementitious materials hydrate, heat is liberated and strength increases. Concrete undergoes chemical and thermal volume change during the exothermic chemical reactions known as cement hydration; elastic volume change in response to changes in temperature. If this volume change is restrained, either externally or internally by temperature or moisture gradients,

tensile stress is developed within the concrete, which will cause cracking if the stress exceeds the tensile capacity of the concrete.

The most common techniques to mitigate thermal cracking are related to mixture proportioning, temperature monitoring and timing of construction practices. Reducing the internal temperature rise through cement reduction and replacement with supplementary cementitious materials is the most common mixture proportioning practice.

### **Design-related**

It is beyond the scope of this document to discuss design, but avoiding potential cracking caused by flexural and tension forces should be considered.

Controlling cracking crosses many boundaries in design, materials specifications, and construction and ultimately depends on checks at each stage of the design and construction. The American Concrete Institute documents ACI209 “Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures” and ACI224 “Control of Cracking in Concrete Structures” can be consulted for additional information.

## **1.10 LOSSAN Rail Corridor Design Methodology**

### **Design Methodology Steps**

This section presents a service life design methodology for the LOSSAN Rail Corridor. The following are steps that should be followed in selecting a corrosion protection system.

Step 1 – Select service life for structure being considered.

Step 2 – Identify environmental exposure zone:

- a. Select exposure:
  - i. Inland Low (Dry) – Figure 23,
  - ii. Inland Moderate (Wet) – Figure 24 through Figure 26,
  - iii. Marine Moderate (Airborne) – Figure 27 through Figure 29, or
  - iv. Marine Severe – Figure 30 through Figure 51.
- b. Confirm selected exposure zone based on a “Condition Assessment,” unless directed otherwise by SANDAG.

Step 3 – Select preliminary concrete mixture:

- A. Concrete mixture:
  - a. Select required water-cementitious ratio based on required compressive strength.
  - b. Determine if precautions are required due to the levels of sulfates contained in the groundwater or soil, as defined in ACI 318. Precautions include strength, w/cm, and cementitious material type.

Step 4 – Select appropriate service life design chart based on exposure zone and selected concrete mixture.

- a. Select the service life in years on the left vertical axis
- b. Draw a line horizontally at the selected service life years until it intersects the appropriate concrete mixture performance design curve.

- c. Draw line vertically from intersection point horizontal axis at bottom of design aid to required cover to reinforcement.

Step 5 – Is required cover to the reinforcement bar acceptable for the structural element being considered? **If yes, service life design is complete and proceed to step 14, if no proceed to Step 6.**

Step 6 – Select concrete mixture with pozzolans that will enhance service life.

- a. Repeat step 4.

Step 7 – Is required cover to the reinforcement bar acceptable for the structural element being considered? **If yes, service life design is complete and proceed to step 14, if no proceed to Step 8.**

Step 8 – Select concrete mixture with calcium corrosion inhibitor that will enhance service life. Incorporate the minimum dosage of 2 g/cy calcium nitrite inhibitors into the concrete mixture.

Step 9 – Repeat step 4.

Step 10 – Is required cover to the reinforcement bar acceptable for the structural element being considered? **If yes, service life design is complete and proceed to step 14, if no proceed to Step 11.**

Step 11 – Select concrete mixture with calcium corrosion inhibitor that will enhance service life. Incorporate additional calcium nitrite inhibitors into the concrete mixture, if required to reach 100-year service life.

Step 12 – Is required cover to the reinforcement bar acceptable for the structural element being considered? **If yes, service life design is complete and proceed to step 14, if no proceed to Step 13.**

Step 13 – Check minimum cover required to mitigate sulfate attack. The depth of cover required can be proportioned based on the 100-year values required for each exposure zone.

Step 14 – Incorporate “Service Life Design” into contract drawings:

- a. Incorporate required cover into design and drawings,
- b. Incorporate concrete mixture requirements into the project specification.
- c. Incorporate “Submittal and Acceptance” requirements into the project specification, reference “Submittal and Acceptance” section of this Design Guide.

Step 15 – Check contractor’s “Service Life” submittal during the pre-construction phase of the project, reference “Submittal and Acceptance” section of this Design Guide

Step 16 – Perform “Quality Control/Quality Assurance” during the construction phase of the project, reference “Quality Control/Quality Assurance” section of this Design Guide.

**Corrosion Protection Systems**

The following components were considered in the corrosion protection systems:

- Covers between one (1) inch and four (4) inches,
- Concrete mixtures covered in the “Concrete Mixtures” section,
- Concrete mixtures with only portland cement,
- Concrete mixtures with fly ash 0%, 15%, 20%, and 25% Type F fly ash,
- Concrete mixtures with slag 0%, 20%, 35%, and 50% slag,
- Concrete with fly ash and silica fume
- Concrete with slag and silica fume
- Concrete mixtures with corrosion inhibitors,
- Concrete mixtures with fly ash and corrosion inhibitors,
- Concrete mixtures with slag and corrosion inhibitors,
- Concrete mixtures with fly ash, silica fume, and corrosion inhibitors,
- Concrete mixtures with slag, silica fume, and corrosion inhibitors,

**Corrosion Protection Enhancements**

The service life of a selected corrosion protection system can be enhanced by selecting a reinforcement material with a longer propagation time, or adding a sealer or coating that will delay the ingress of chlorides. Table 20 contains a list of materials and associated years of added protection. Material combinations and additional system protection components not covered in this Design Guide can be analyzed by experts in the service life field.

Table 20 – Additional Corrosion Protection Components

	<b>Material</b>	<b>Method for Increasing Service Life</b>	<b>Years of added protection</b>
Sealer	Surface Treatment	Reduce ingress of chlorides	2 to 15 years <sup>1</sup>
Coating	Surface Barrier	Prevent ingress of chlorides	5 to 25 years <sup>1</sup>

Note: 1. Confirm “Years of added protection” with product manufacturer and also consult approved list of CALTRANS qualified product suppliers.

**Design Example**

Design drainage culvert for 100-year service life. The drainage culvert is located at MP 253.0 and exposed to moisture, soil, and groundwater. For structural design reasons, the required concrete compressive strength is 4000 psi. A condition assessment of a nearby concrete structure and testing of soil and groundwater provide the following information about the exposure condition:

- Concrete – Maximum acid-soluble chloride content is 2500 ppm,
- Groundwater and soil – The sulfate (SO<sub>4</sub>) content is 3000 ppm.

Design:

- The drainage structure is located at MP 253.0 and is exposed to moisture, so should be designed using Inland Moderate with a relative humidity of 68 to 100%,
  - Acid-soluble chloride is less than 4500 ppm.

- The required compressive strength is 4000 psi, so select a concrete mixture with a water-cementitious ratio of 0.50.
- Consider a depth of cover to reinforcement of 2 inches for the inside curtain of reinforcement and 3 inches for the exterior curtain of reinforcement exposed to earth.
- Review of service life design charts for 4000 psi, 5000 psi, and 6000 psi concrete mixtures show that none of the concrete mixtures provide a 100-year service life with an acceptable cover and a relative humidity of 68 to 100%, 6000 psi concrete mixture chart given in Figure 21.

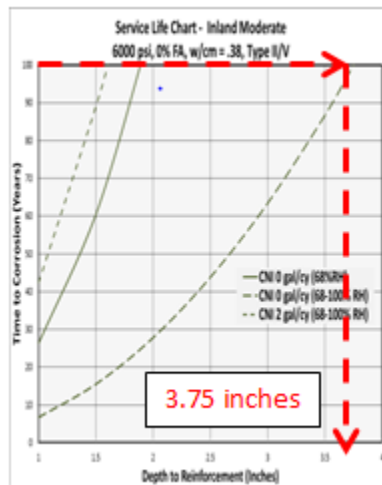


Figure 21 - 6000 psi concrete mixture with 0% FA for inland moderate exposure

- Consider a concrete mixture with an SCM of 15% fly ash. Review of the Moderate Inland Exposure Zone design charts shows that a 6000 psi concrete mixture with 15% fly ash will provide a 100-year service life with an acceptable cover.

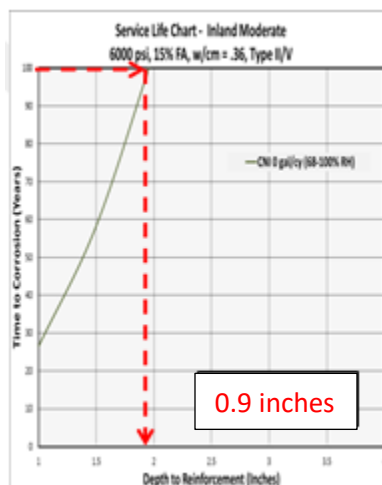


Figure 22 - 6000 psi concrete mixture with 15% FA for inland moderate exposure

- The SO<sub>4</sub> content in the groundwater and soil is between 2000 and 4000 ppm. Check minimum depth of cover required to mitigate sulfate attack in a moist environment, per Figure 22, a 6000 psi concrete mixture with 15% fly ash to find that a cover of 1.75

inches of cover is required for sulfate resistance. 1.75 inches is an acceptable cover for the interior curtain of reinforcement that is exposed to groundwater runoff and 3 inches for the exterior curtain of reinforcement that is exposed to earth.

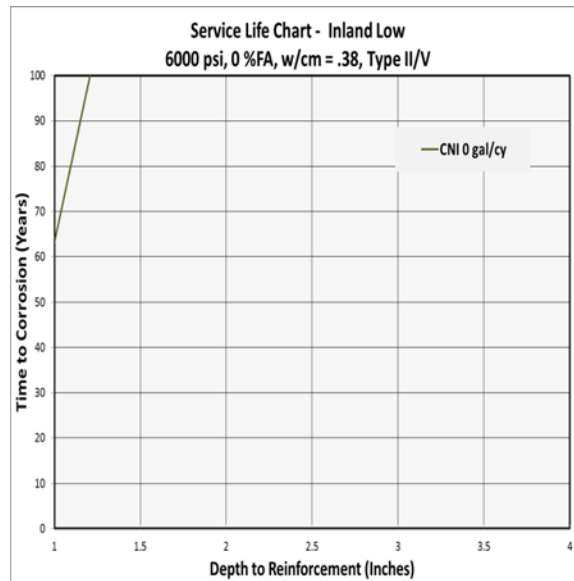
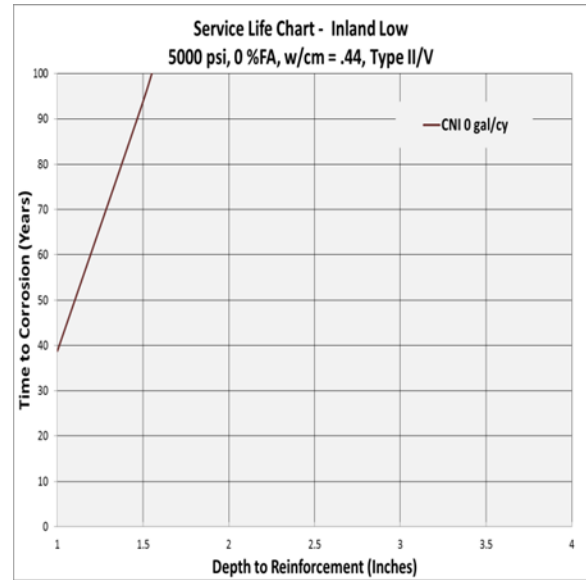
- Specify mixture with a water-cementitious ratio of 0.36 and a minimum of 15% Type F fly ash content and 2 inches of cover for the interior curtain of reinforcement and 3 inches of cover for the exterior curtain of reinforcement. It should be noted that the compressive strength requirements for the concrete mixture will not control.
- Incorporate “Service Life Design” into contract drawings and specifications.
- Incorporate “Submittal and Acceptance” requirements into the project specification, reference “Submittal and Acceptance” section of this Design Guide.

### **1.11 Service Life Design Charts**

This section presents design charts for the four exposure zones.

#### **1.11.1 Inland Low Exposure**

The Inland Low Exposure zone can be used for structures that may be exposed to airborne chloride that has a magnitude much less than that of the Marine Moderate Exposure Zone and structural elements that are not exposed to ground water or soil. Service life curves are presented in Figure 23 for this exposure. Sulfate exposure is considered negligible for this exposure condition where the structural elements are not exposed to groundwater or soil.



- Note:
1. Type II/V portland cement
  2. Corrosion threshold is equal to 500 ppm for black reinforcement.

Figure 23 – Inland Low Exposure – Service Life Chart for 4000 psi, 5000 psi, and 6000 psi Concrete Mixtures



**1.11.2 Inland Moderate Exposure**

The Inland Moderate Exposure Zone should be used for structural elements that are exposed to airborne, seasonal ground water conditions and soil, and ground water. A relative humidity of 68% was considered which could be used for an airborne chloride exposure that falls between the Inland Low and Marine Moderate Exposure Zones. The relative humidity was varied from 68% to 100% for elements exposed to periods of wet and dry conditions, and a 100% humidity was considered for elements immersed in soil/groundwater. There will be limited amounts of oxygen available to drive corrosion for elements immersed in soil/ground water with a neutral pH, so the main concern with these elements is the exposure to sulfate attack. Service life curves are presented in Figure 24, Figure 25, and Figure 26 for this exposure.

Sulfate attack should be considered for this exposure zone. The parameters presented in Table 2 were used in STADIUM® to arrive at the minimum depth of cover for two different SO<sub>4</sub> exposures. A minimum depth of cover is required to provide a 100-year service life with respect to sulfate attack, and is presented in Table 21, Table 22 and Table 23 for each concrete mixture. Use the maximum sulfate exposure of 4000 ppm, if a condition assessment is not performed. A minimum 3 inches depth of cover has been specified if the concrete is in contact with soil.

Table 21– Minimum Cover to Mitigate Sulfate Attack Degradation at 100-years with Relative Humidity of 68%.

Concrete Mixture	Minimum Cover	Minimum Cover
RH = 68%	Inches	Inches
SO <sub>4</sub>	2000 ppm	4000 ppm
4000 psi	1.625	2.0
4000 psi with 15% FA	1.0	1.0
4000 psi with 20% FA	1.0	1.0
4000 psi with 25% FA	1.0	1.0
4000 psi with 20% Slag	1.0	1.0
4000 psi with 35% Slag	1.0	1.0
4000 psi with 50% Slag	1.0	1.0
5000 psi	1.625	2.0
5000 psi with 15% FA	1.0	1.0
5000 psi with 20% FA	1.0	1.0
5000 psi with 20% Slag	1.0	1.0
5000 psi with 35% Slag	1.0	1.0
6000 psi	1.625	2.0

Table 22 – Minimum Cover to Mitigate Sulfate Attack Degradation at 100-years with Relative Humidity of 68 to 100%.

Concrete Mixture	Minimum Cover	Minimum Cover <sup>1</sup>	Minimum Cover	Minimum Cover <sup>1</sup>
RH = 68% to 100%	Inches	Inches	Inches	Inches
	Groundwater	In contact with soil	Groundwater	In contact with soil
SO <sub>4</sub>	2000 ppm	2000 ppm	4000 ppm	4000 ppm
4000 psi	2.75	3.0	3.25	3.25
4000 psi with 15% FA	1.5	3.0	1.75	3.0
4000 psi with 20% FA	1.5	3.0	1.75	3.0
4000 psi with 25% FA	1.5	3.0	1.75	3.0
4000 psi with 20% Slag	1.5	3.0	1.75	3.0
4000 psi with 35% Slag	1.5	3.0	1.75	3.0
4000 psi with 50% Slag	1.5	3.0	1.75	3.0
5000 psi	2.75	3.0	3.25	3.25
5000 psi with 15% FA	1.5	3.0	1.75	3.0
5000 psi with 20% FA	1.5	3.0	1.75	3.0
5000 psi with 20% Slag	1.5	3.0	1.75	3.0
5000 psi with 35% Slag	1.5	3.0	1.75	3.0
6000 psi	2.75	3.0	3.25	3.25
6000 psi with 15% FA	1.5	3.0	1.75	3.0
6000 psi with 20% FA	1.5	3.0	1.75	3.0
6000 psi with 20% Slag	1.5	3.0	1.75	3.0
6000 psi with 35% Slag	1.5	3.0	1.75	3.0

Notes:

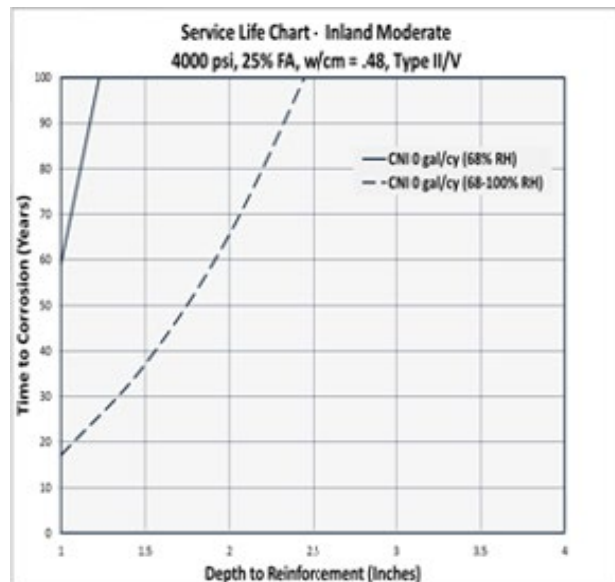
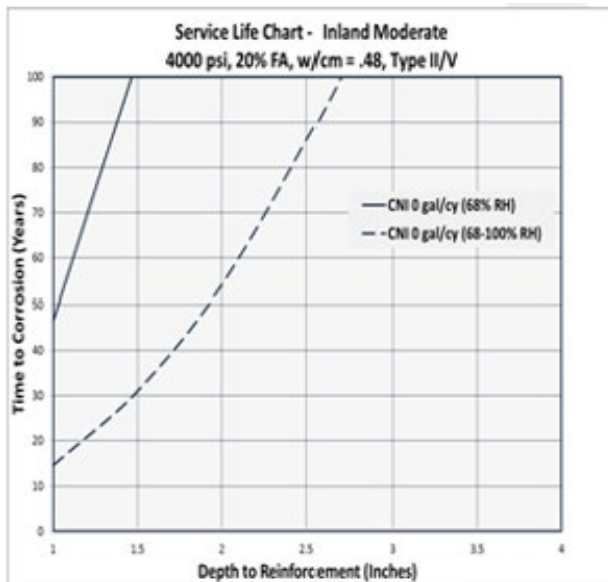
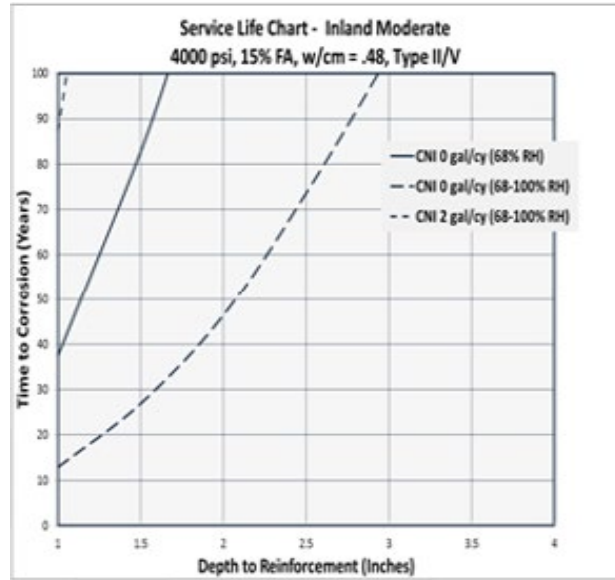
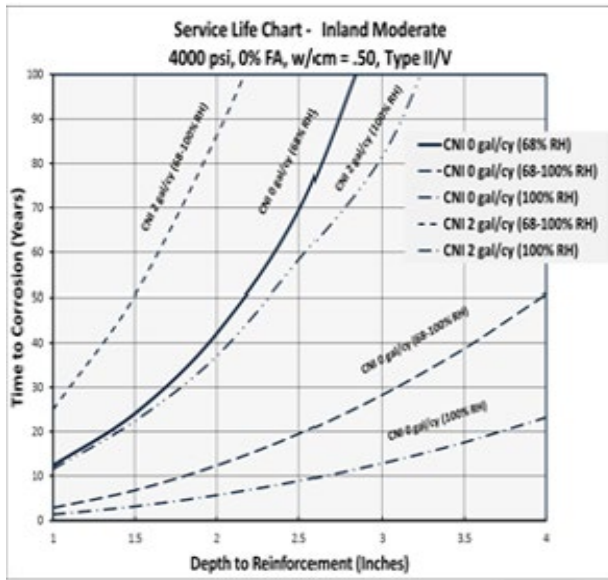
1. Minimum of 3 inches of cover controls, reference CALTRANS.

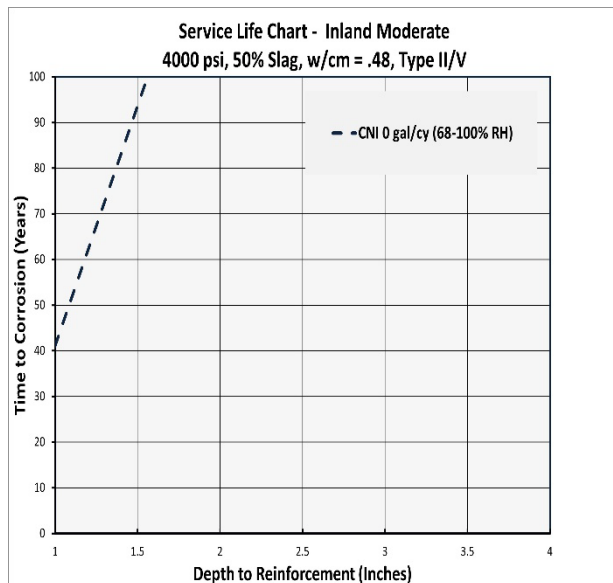
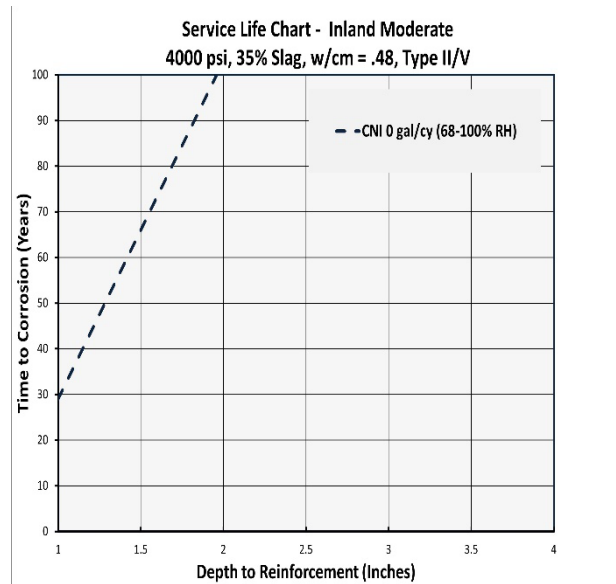
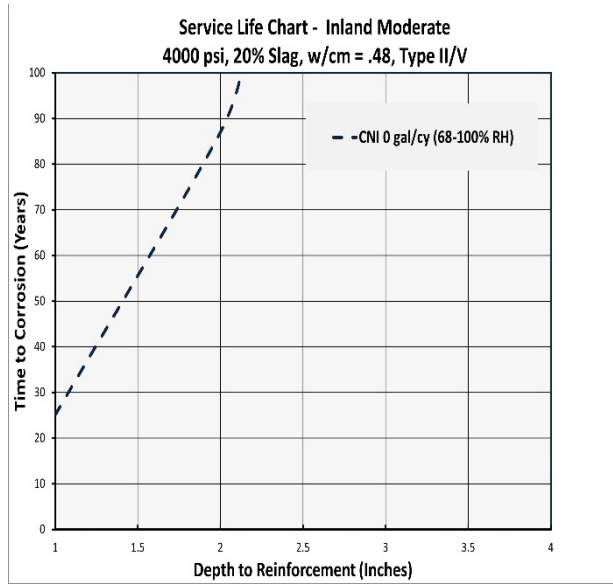
Table 23 – Minimum Cover to Mitigate Sulfate Attack Degradation at 100-years at a humidity of 100%.

Concrete Mixture	Minimum Cover <sup>1</sup>
<b>RH = 100%</b>	<b>Inches</b>
	In contact with soil and ground water
SO <sub>4</sub>	4000 ppm
4000 psi	5.0
4000 psi with 15% FA	3.0
4000 psi with 20% FA	3.0
4000 psi with 20% Slag	3.0
4000 psi with 35% Slag	3.0
5000 psi	4.0
5000 psi with 15% FA	3.0
5000 psi with 20% FA	3.0
5000 psi with 20% Slag	3.0
5000 psi with 35% Slag	3.0
6000 psi	3.0
6000 psi with 15% FA	3.0
6000 psi with 20% FA	3.0
6000 psi with 20% Slag	3.0
6000 psi with 35% Slag	3.0

Notes:

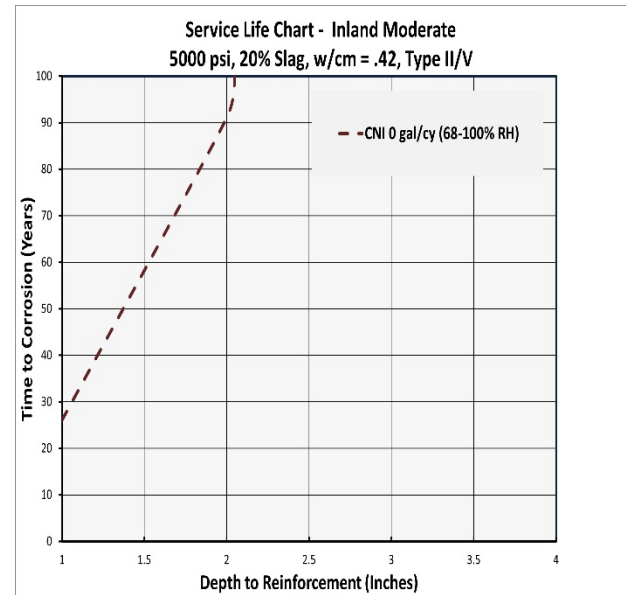
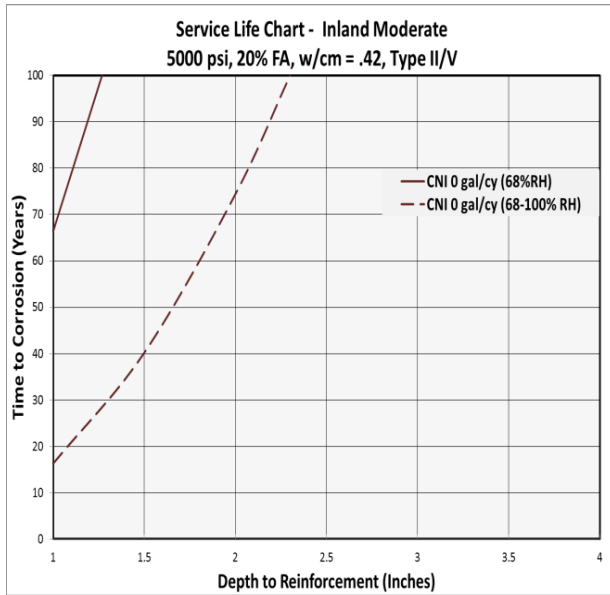
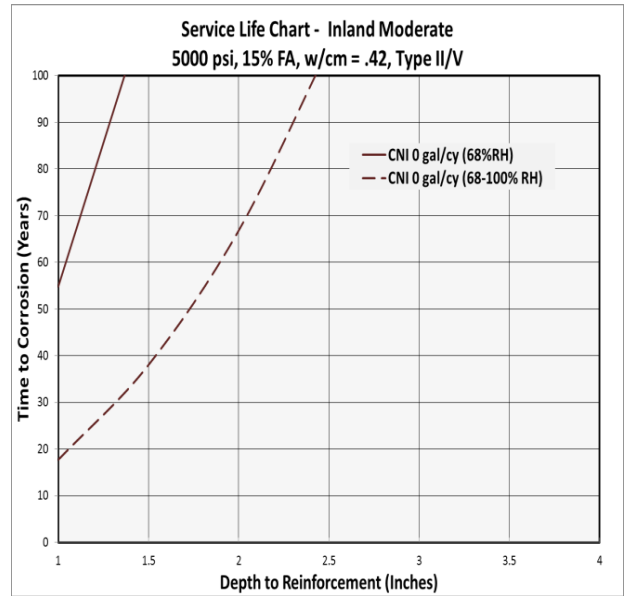
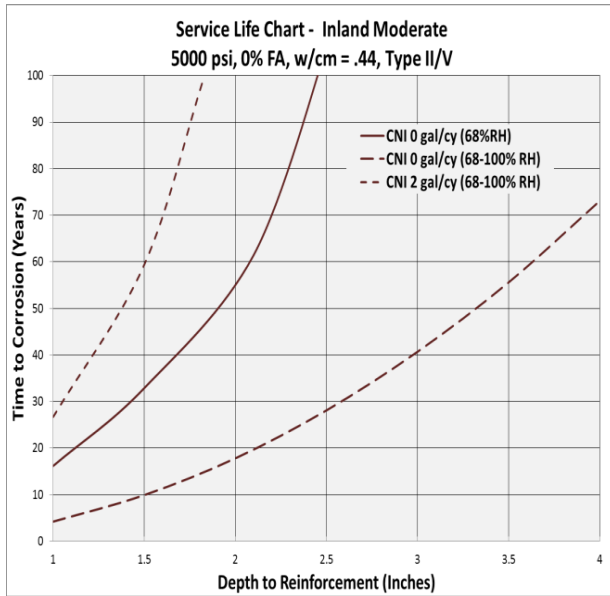
1. Minimum of 3 inches of cover controls, reference CALTRANS.

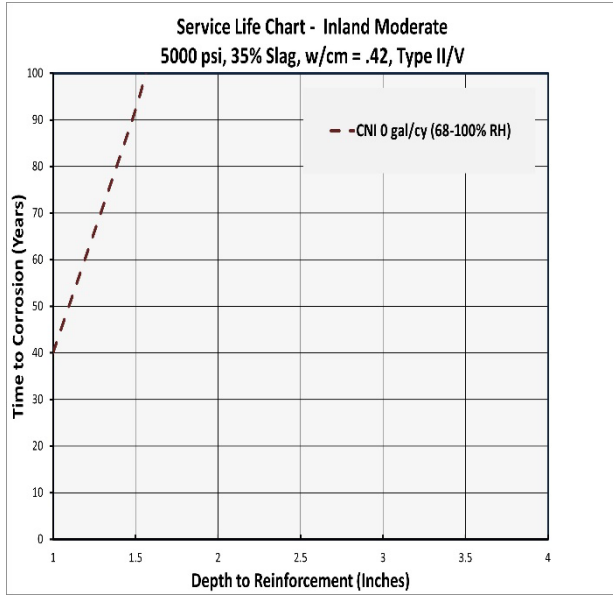




- Notes:
1. Type II/V portland cement
  2. Corrosion threshold is equal to 500 ppm for black reinforcement.
  3. A corrosion inhibitor dosage of 2 G/CY will provide a predicted service life of 100-years at all covers for the 4000 psi concrete mixtures, for a relative humidity of 68%.
  4. A corrosion inhibitor dosage of 3 G/CY will provide a predicted service life of 100-years at all covers for the 4000 psi concrete mixtures with and without fly ash, for a relative humidity of 68% to 100% and 100% humidity.
  5. 4000 psi concrete mixtures with slag will provide a predicted service life of 100-years at all covers greater than 1 inch for a relative humidity of 68%.
  6. A corrosion inhibitor dosage of 2 G/CY or more will provide a predicted service life of 100-years at all covers for 4000 psi concrete mixtures with slag.

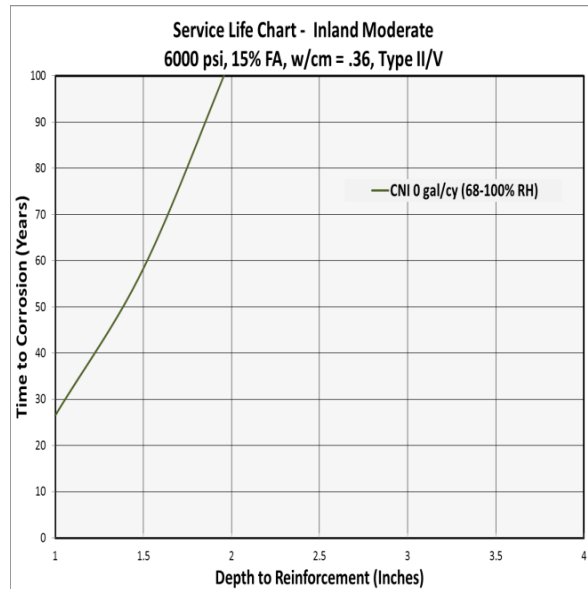
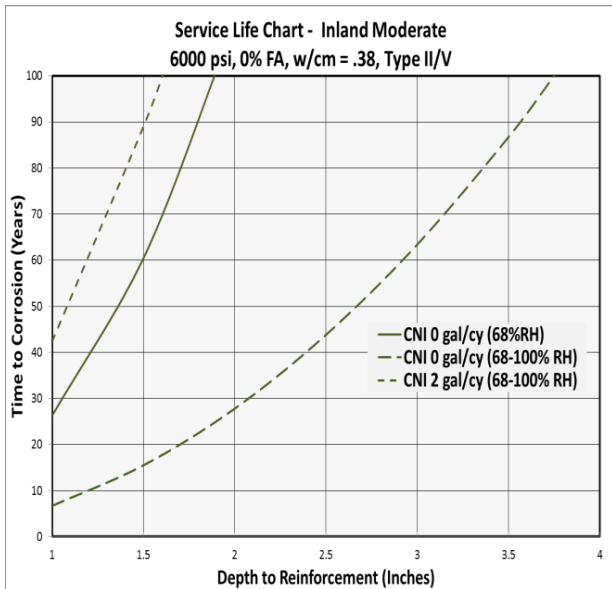
Figure 24 – Inland Moderate Exposure – Service Life Chart for 4000 psi Concrete Mixtures

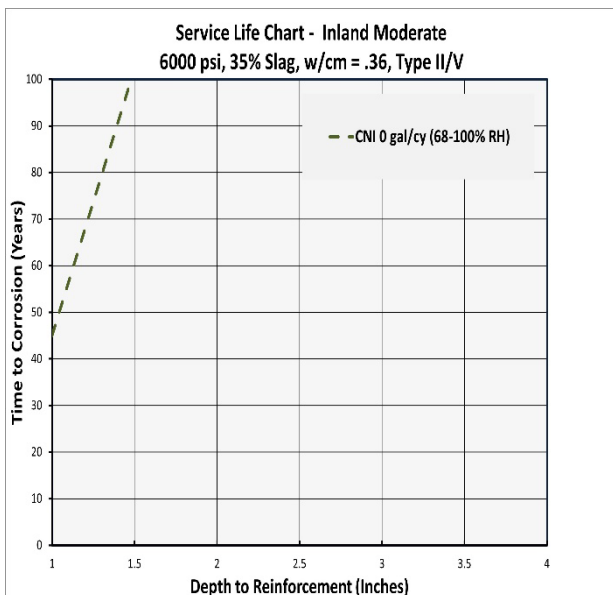
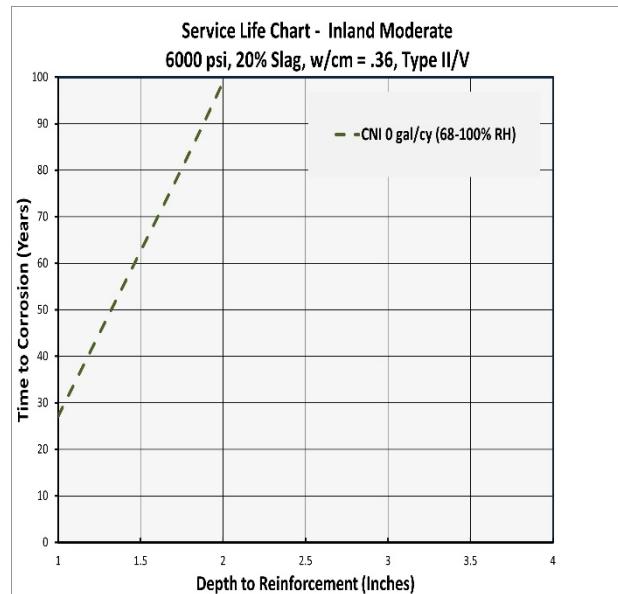
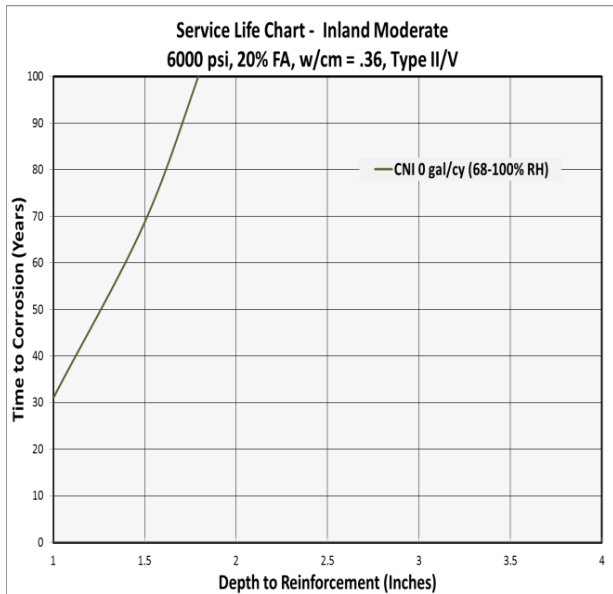




- Note:
1. Type II/V portland cement.
  2. Corrosion threshold is equal to 500 ppm.
  3. 5000 psi concrete mixtures with slag will provide a predicted service life of 100-years at all covers greater than 1 inch for a relative humidity of 68%.

Figure 25 – Inland Moderate Exposure – Service Life Chart for 5000 psi Concrete Mixtures





- Notes:
1. Type II/V portland cement.
  2. Corrosion threshold is equal to 500 ppm.
  3. A corrosion inhibitor dosage of 2 G/CY will provide a predicted service life of 100-years at all covers for the 6000 psi concrete mixture, for a relative humidity of 68%.
  4. A corrosion inhibitor dosage of 2 G/CY will provide a predicted service life of 100-years at all covers for the 6000 psi concrete mixtures with fly ash or slag, for a relative humidity of 68 to 100%.
  5. 6000 psi concrete mixtures with slag will provide a predicted service life of 100-years at all covers greater than 1 inch for a relative humidity of 68%.

Figure 26 – Inland Moderate Exposure – Service Life Chart for 6000 psi Concrete Mixture



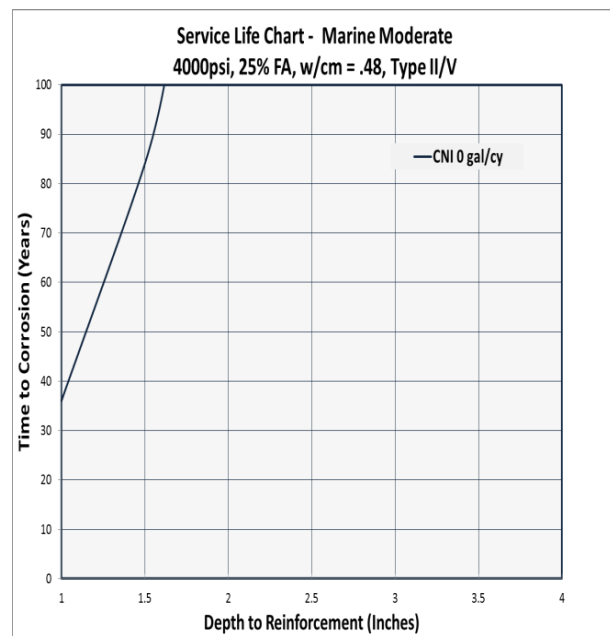
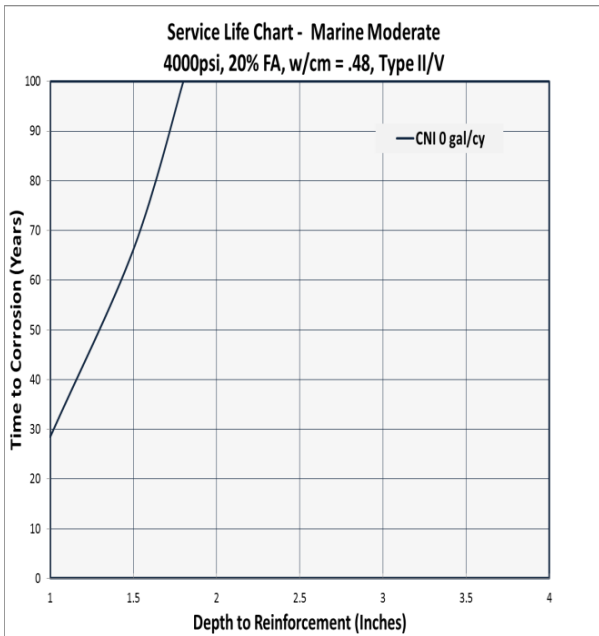
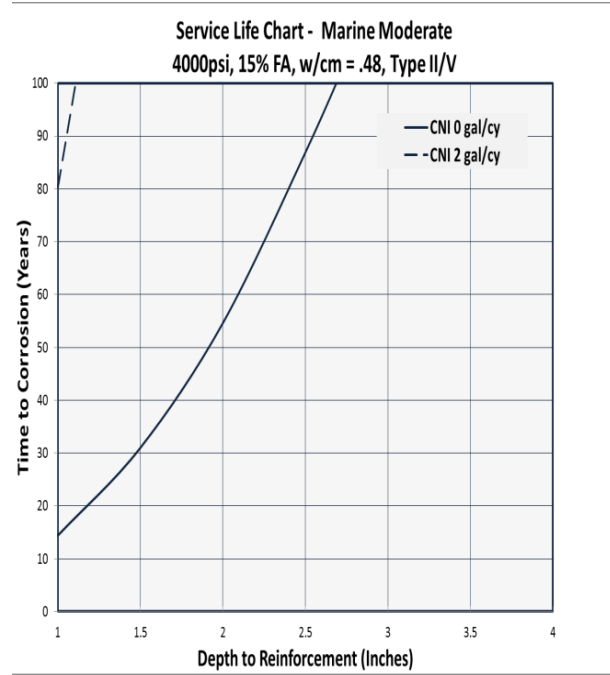
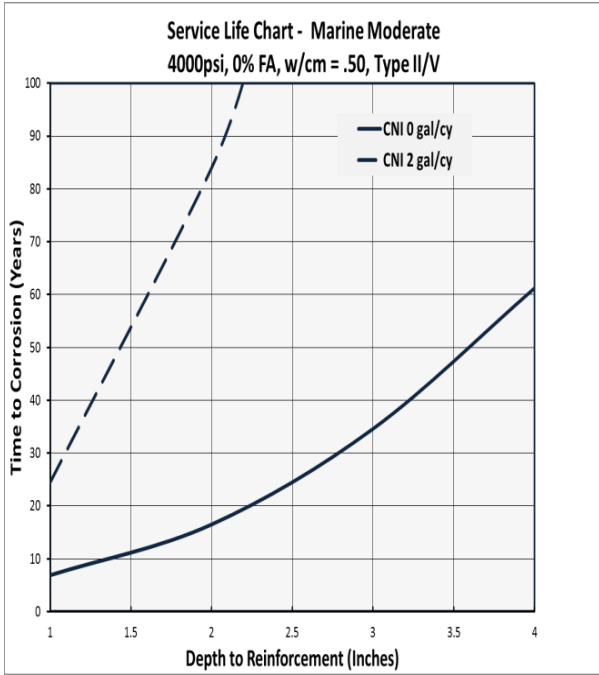
**1.11.3 Marine Moderate (Airborne) Exposure**

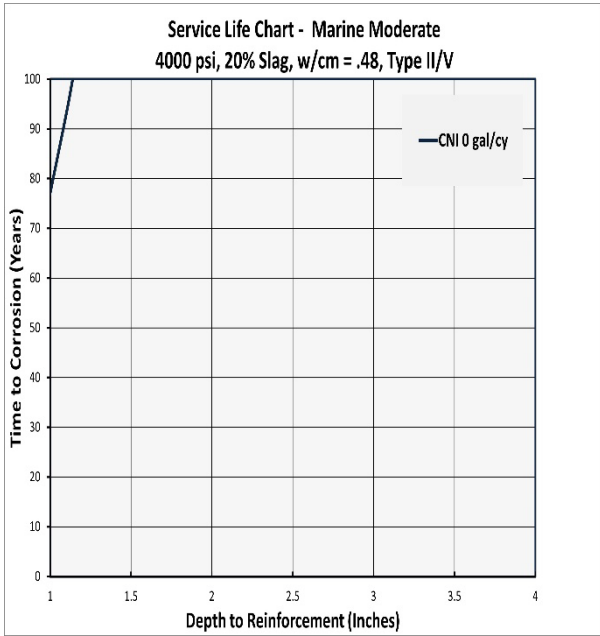
The Marine Moderate (Airborne) Exposure should be used for structural elements exposed to airborne chlorides along the Pacific Coast. Concrete mixtures with water-cementitious ratios of 0.50, 0.44, and 0.38 were considered for the Marine Moderate (Airborne) exposure condition, for the service life curves presented in Figure 27, Figure 28, and Figure 29.

Sulfate attack is negligible for this airborne chloride exposure. A minimum depth of cover is presented in Table 24 for each concrete mixture.

Table 24 – Minimum Cover to Mitigate Sulfate Attack Degradation at 100-years

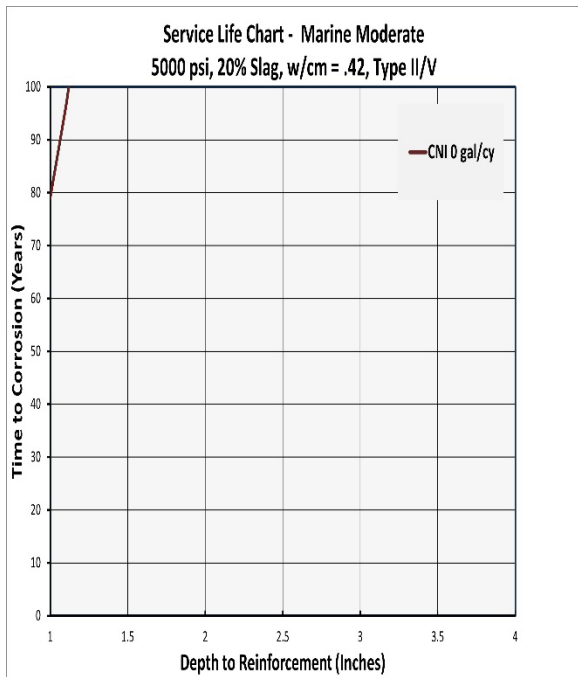
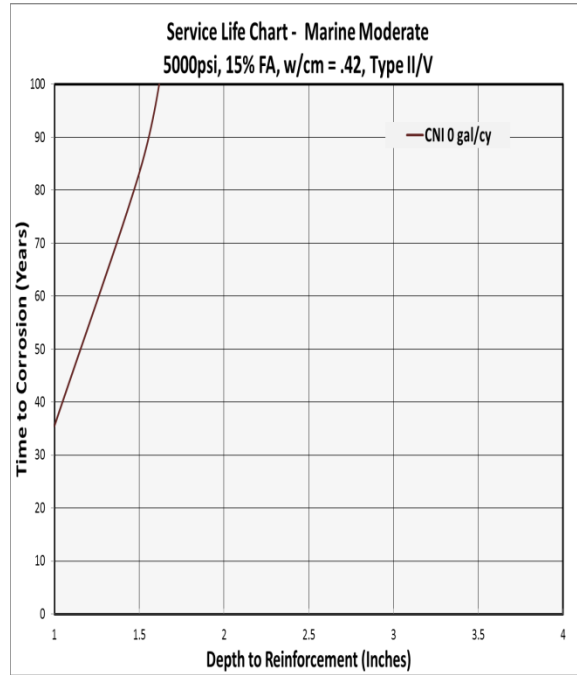
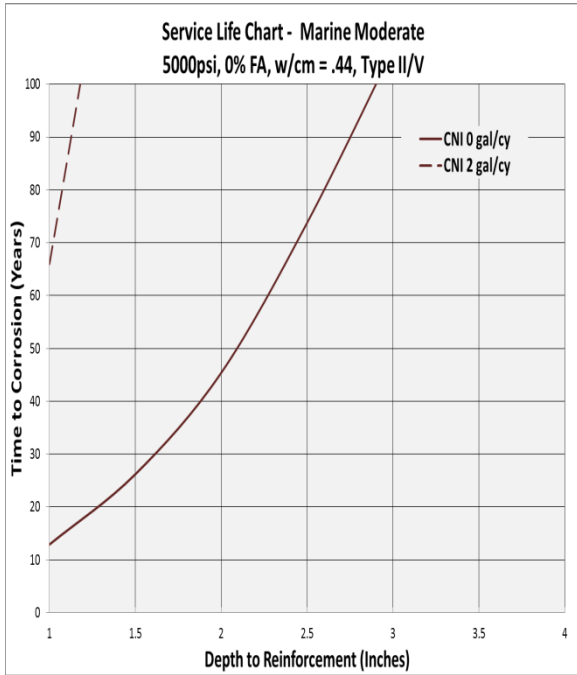
Concrete Mixture	Minimum Cover
	Inches
4000 psi	1.875
4000 psi with 15% FA	0.5
4000 psi with 20% FA	0.375
4000 psi with 25% FA	0.375
4000 psi with 20% Slag	0.50
4000 psi with 35% Slag	0.375
4000 psi with 50% Slag	0.375
5000 psi	0.75
5000 psi with 15% FA	0.75
5000 psi with 20% Slag	0.75
6000 psi	0.5
6000 psi with 15% FA	0.25
6000 psi with 20% Slag	0.25





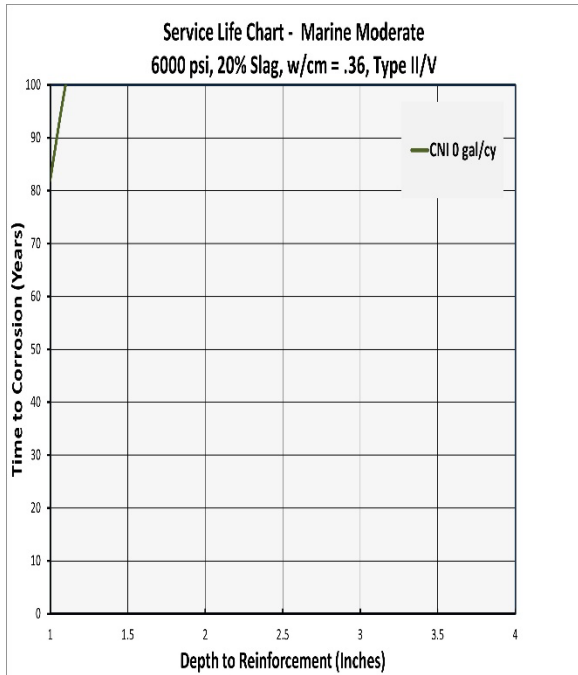
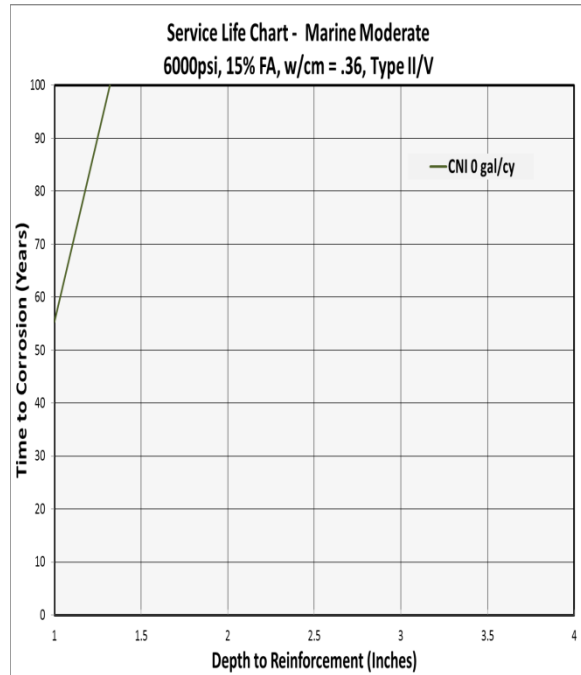
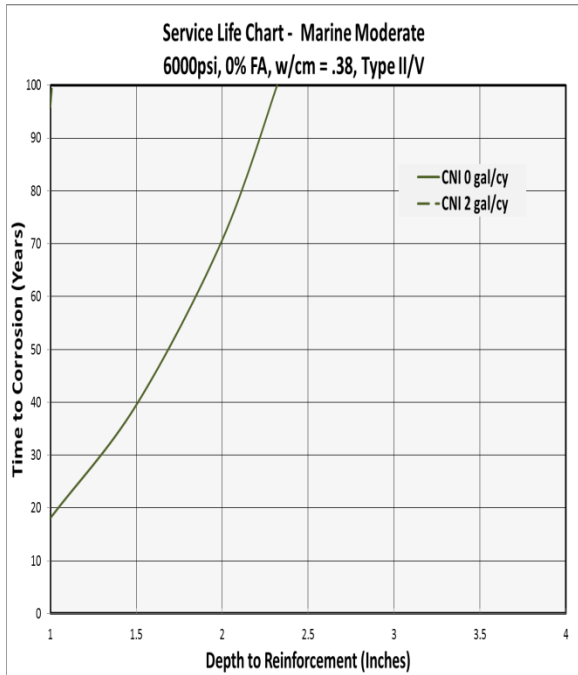
- Note:
1. Type II/V portland cement
  2. Corrosion threshold is equal to 500 ppm for black reinforcement.
  3. A corrosion inhibitor dosage of 2 G/CY will provide a predicted service life of 100-years at all covers, for the 4000 psi concrete mixtures.
  4. 4000 psi concrete mixtures with 35% and 50% slag will provide a predicted service life of 100-years at all covers greater than 1 inch.

Figure 27 – Marine Moderate Exposure – Service Life Chart for 4000 psi Concrete Mixtures



- Note:
1. Type II/V portland cement
  2. Corrosion threshold is equal to 500 ppm for black reinforcement.

Figure 28 – Marine Moderate Exposure – Service Life Chart for 5000 psi Concrete Mixtures



- Notes:
1. Type II/V portland cement
  2. Corrosion threshold is equal to 500 ppm for black reinforcement.
  3. A corrosion inhibitor dosage of 2 G/CY will provide a service life of 100-years at all covers, for the 6000 psi concrete mixture.

Figure 29 – Marine Moderate Exposure – Service Life Chart for 6000 psi Concrete Mixtures

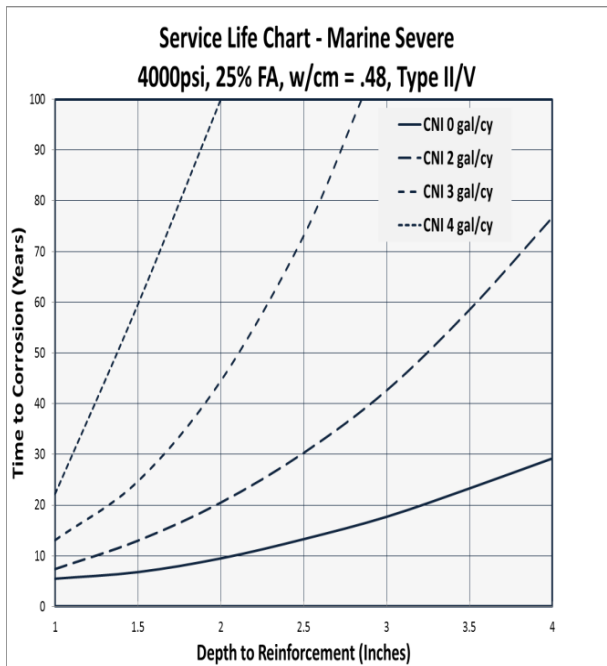
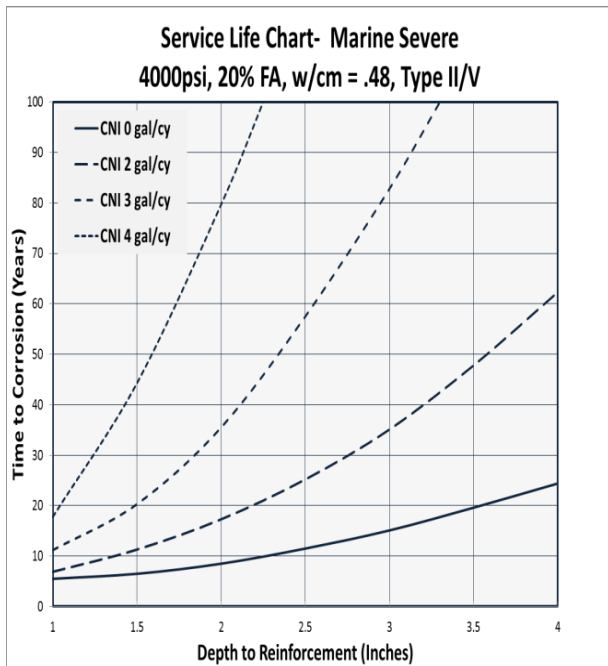
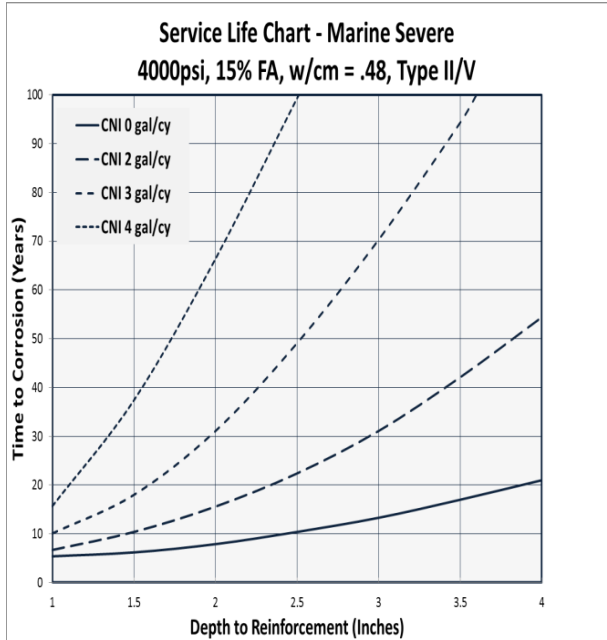
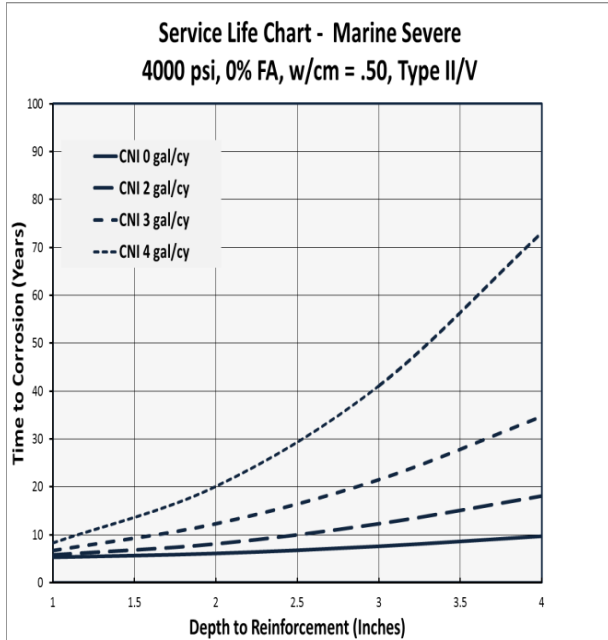
**1.11.4 Marine Severe Exposure**

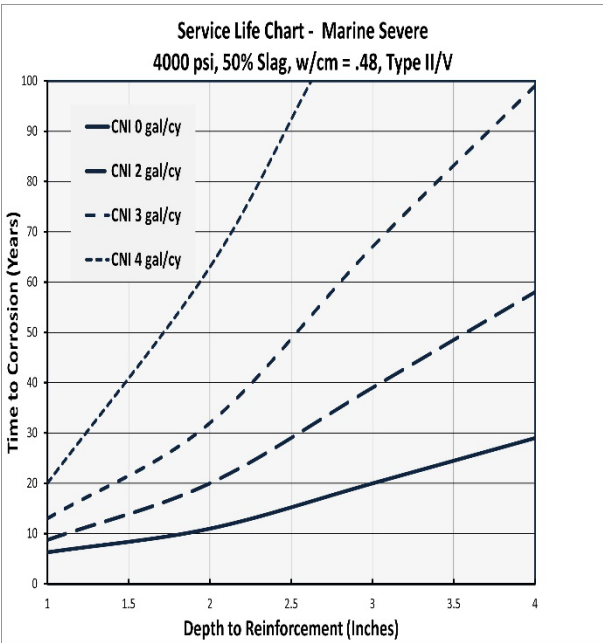
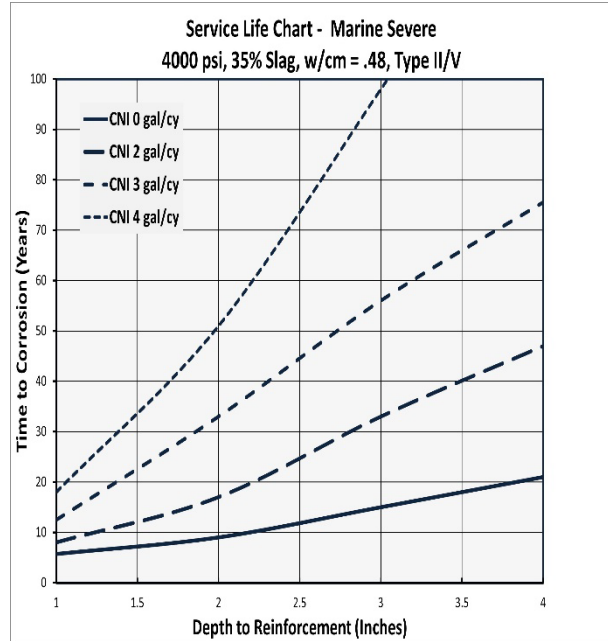
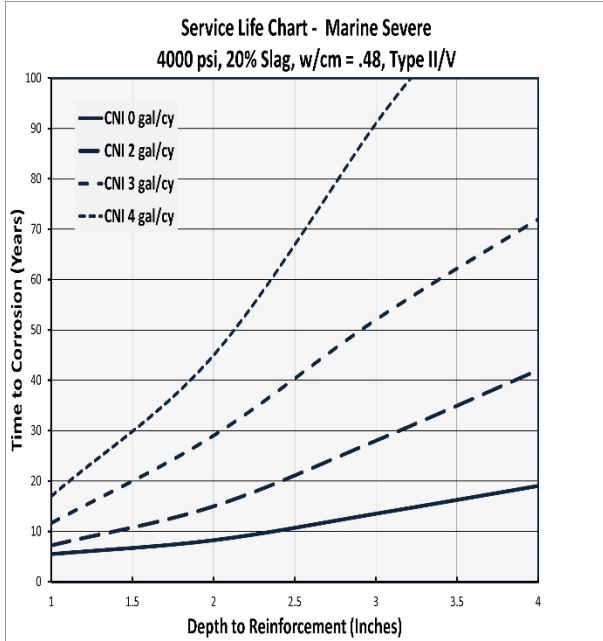
Service life curves are presented in Figure 30 through Figure 51 for this exposure.

Sulfate attack should be considered for this exposure condition. A minimum depth of cover is required to provide a 100-year service life with respect to sulfate attack and is presented in Table 25 for each concrete mixture. The reader will note that the required depth of cover decreases with increased amounts of Class F Fly ash, except for the 6000 psi mix containing either 25 percent fly ash or 50 percent slag.

Table 25 – Minimum Cover to Mitigate Sulfate Attack Degradation at 100-years

Concrete Mixture	Minimum Cover
	Inches
4000 psi	6.5
4000 psi with 15% FA	3.0
4000 psi with 20% FA	2.75
4000 psi with 25% FA	2.5
4000 psi with 20% Slag	3.0
4000 psi with 35% Slag	2.75
4000 psi with 50% Slag	2.5
5000 psi	5.5
5000 psi with 15% FA	2.5
5000 psi with 20% FA	2.25
5000 psi with 25% FA	1.75
5000 psi with 20% Slag	2.5
5000 psi with 35% Slag	2.25
5000 psi with 50% Slag	1.75
6000 psi	5.25
6000 psi with 15% FA	1.75
6000 psi with 20% FA	1.5
6000 psi with 25% FA	2.5
6000 psi with 20% Slag	1.75
6000 psi with 35% Slag	1.5
6000 psi with 50% Slag	2.5
6000 psi with 25% FA and 5% SF	1.25
6000 psi with 50% Slag and 5% SF	1.25

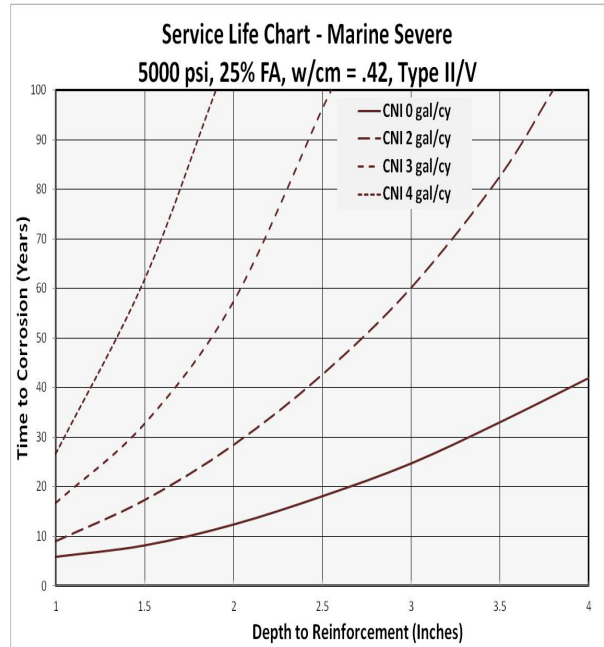
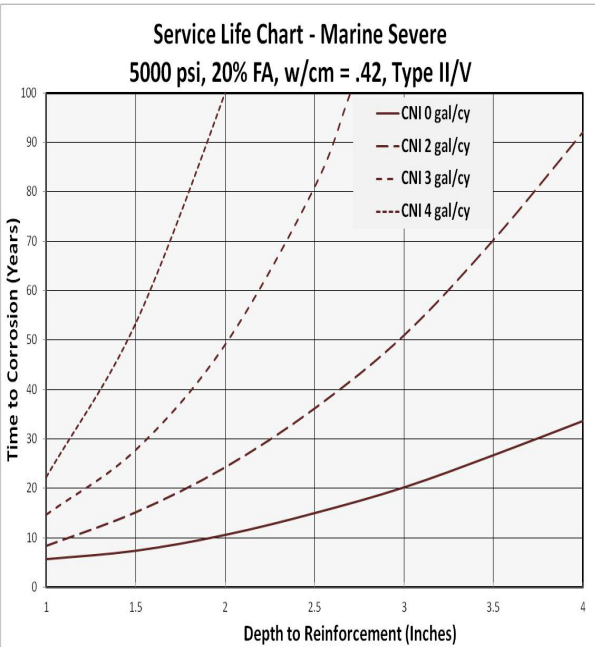
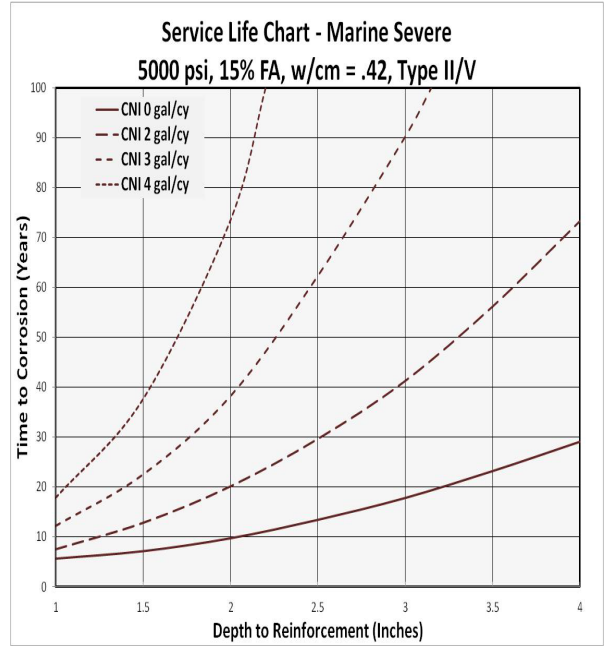
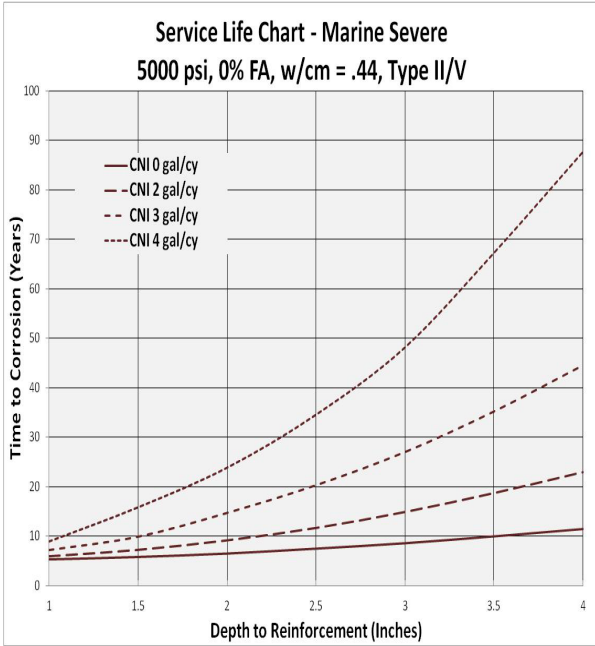


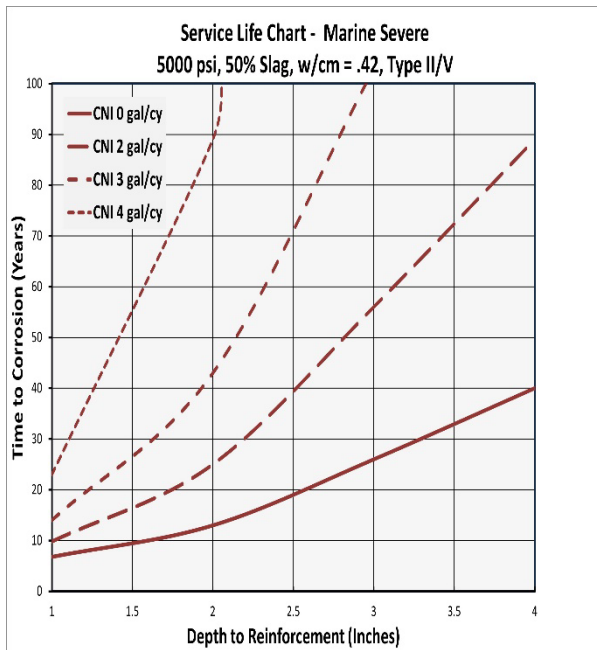
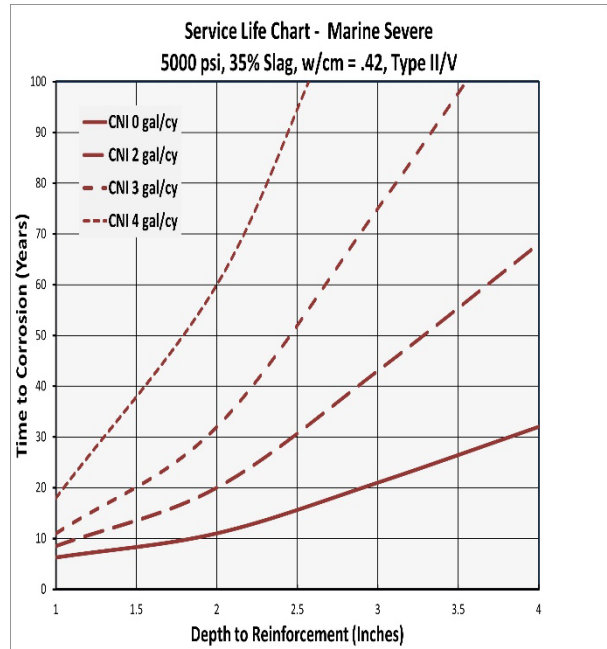
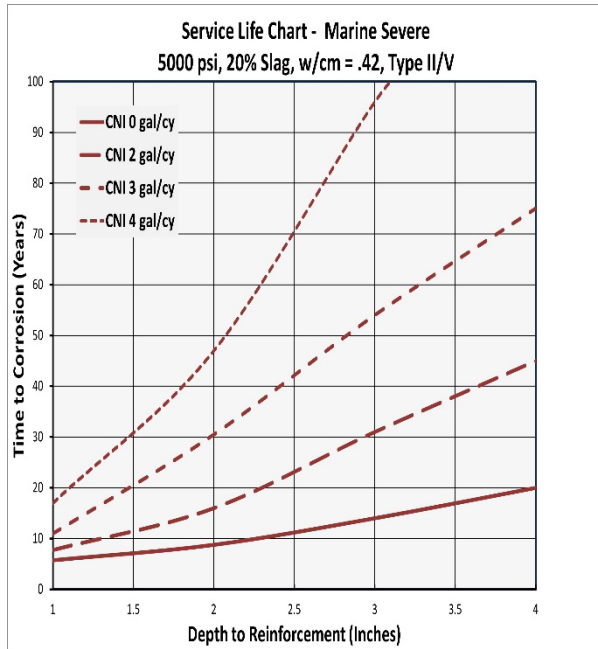


- Note:
1. Type II/V portland cement
  2. Corrosion threshold is equal to 500 ppm for black reinforcement.

Figure 30 – Marine Severe Exposure – Service Life Chart for 4000 psi Concrete Mixtures

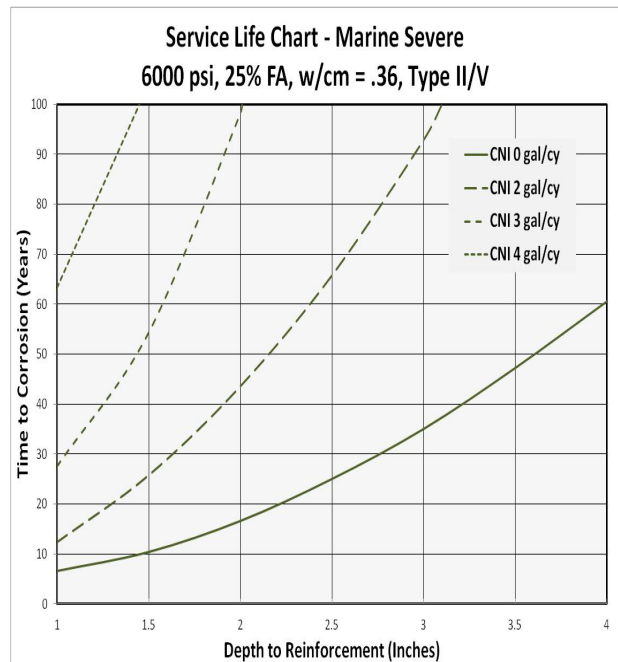
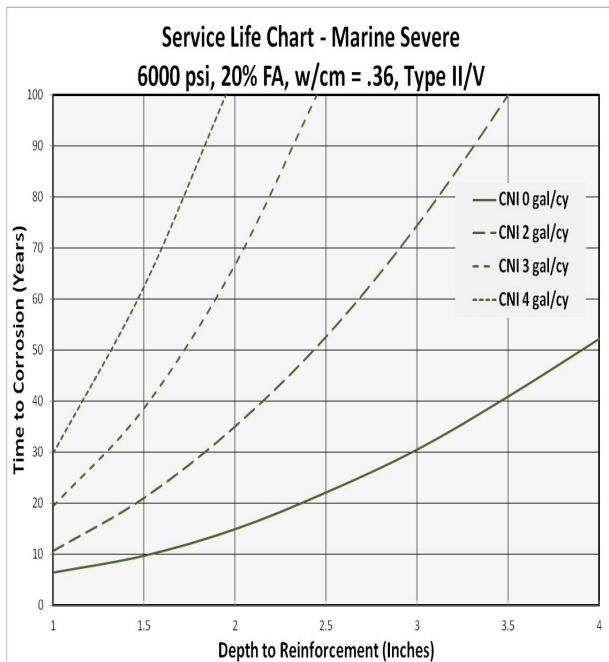
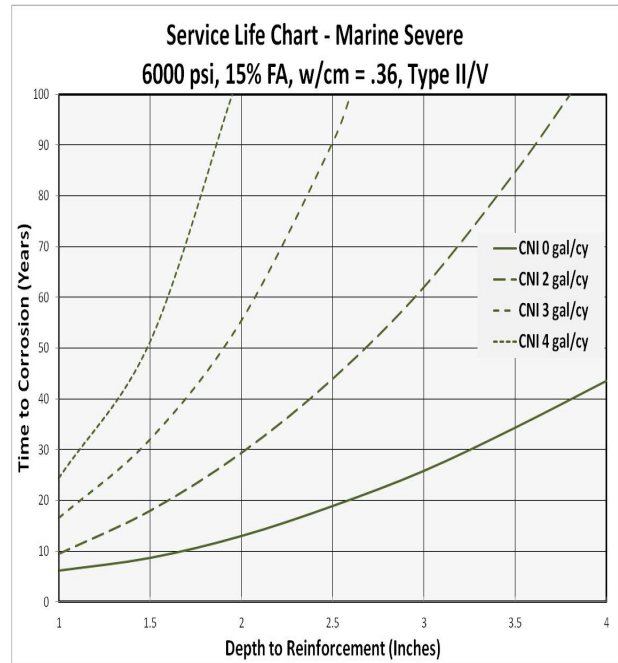
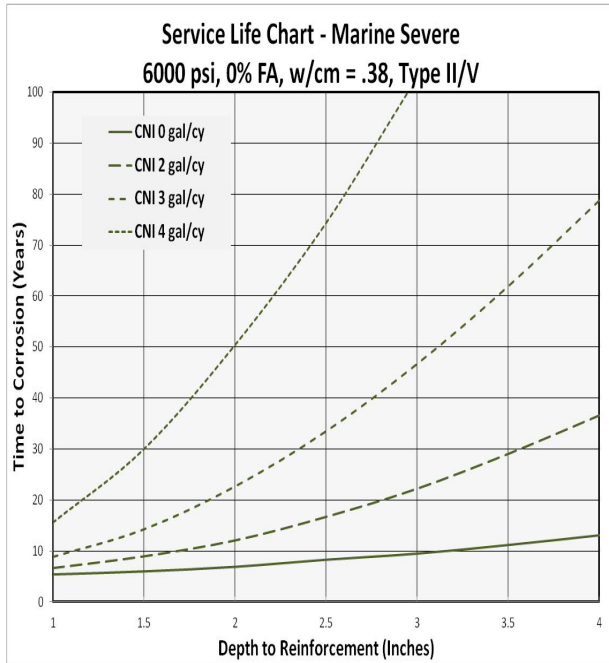


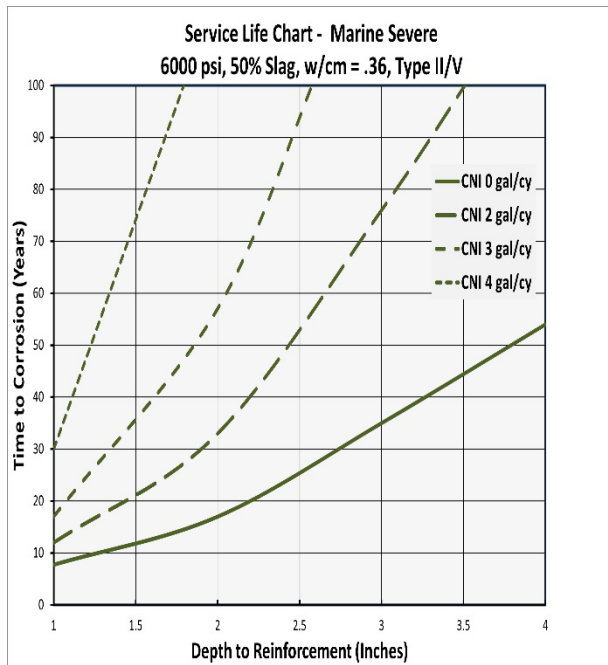
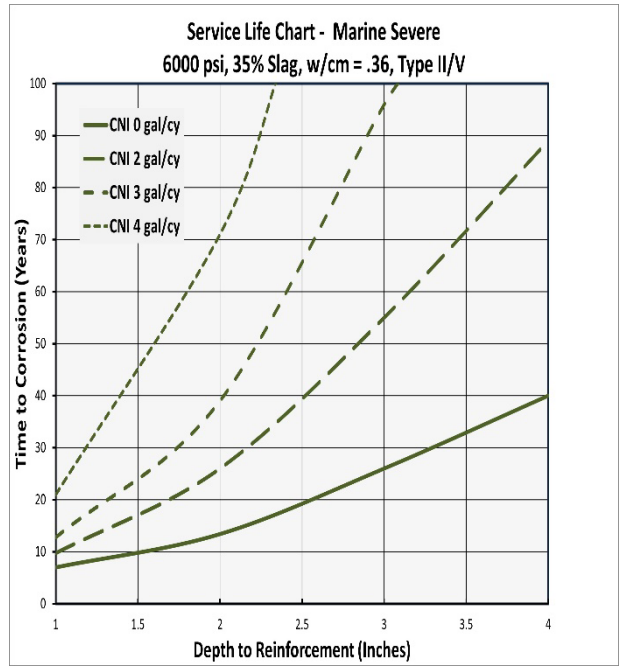
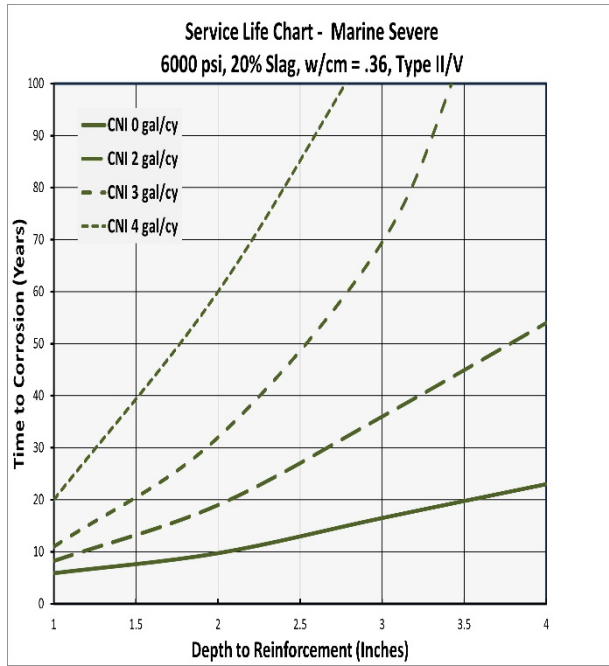




Note: 1. Type II/V portland cement  
2. Corrosion threshold is equal to 500 ppm for black reinforcement.

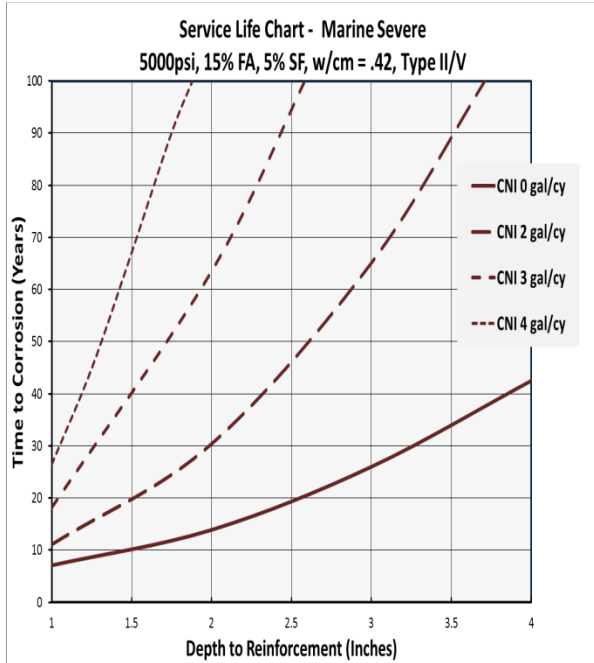
Figure 31 – Marine Severe Exposure – Service Life Chart for 5000 psi Concrete Mixtures





- Note:
1. Type II/V portland cement
  2. Corrosion threshold is equal to 500 ppm for black reinforcement.

Figure 32 – Marine Severe Exposure – Service Life Chart for 6000 psi Concrete Mixtures



- Note:
1. Type II/V portland cement
  2. Corrosion threshold is equal to 500 ppm for black reinforcement.

Figure 33 – Marine Severe Exposure – Service Life Chart for 5000 psi Concrete Mixture with 15% FA and 5% SF

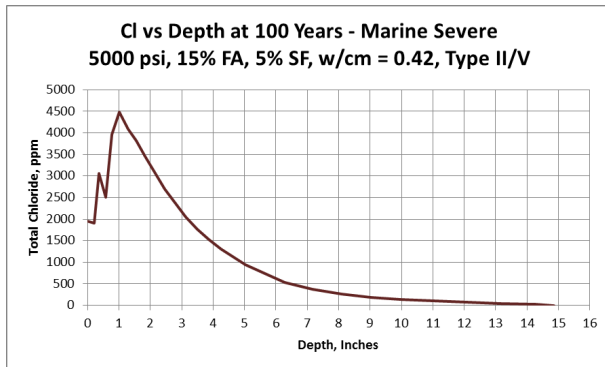


Figure 34 – Marine Severe Exposure – Service Life Chart Displaying the Chloride Ingress by Depth for 5000 psi Concrete Mixture with 15% FA and 5% SF

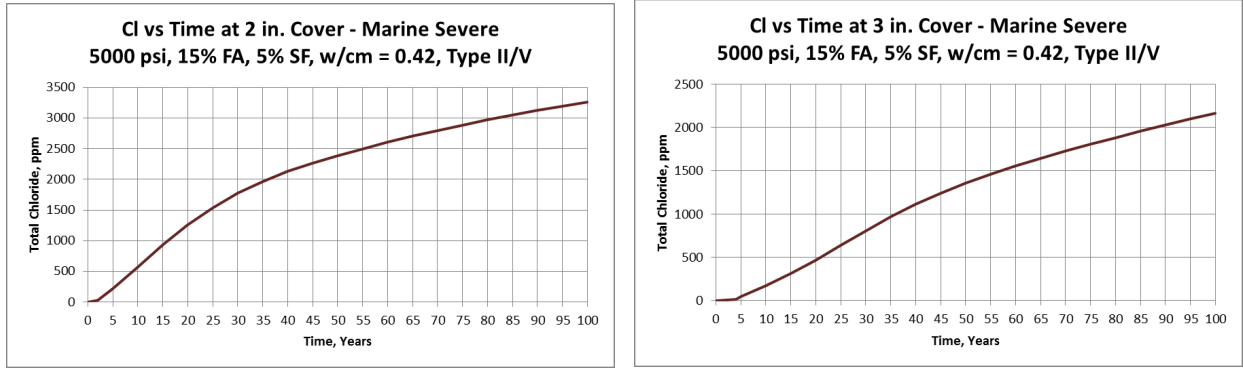
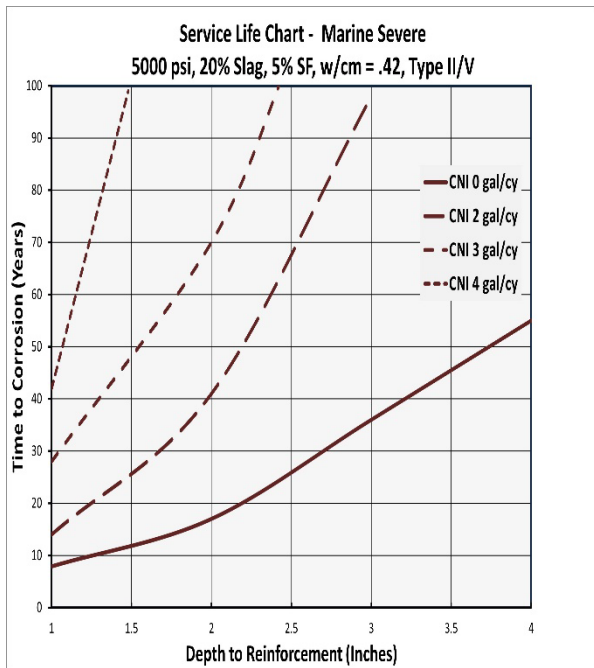


Figure 35 – Marine Severe Exposure – Service Life Chart Displaying Chloride ingress by Time at 2 in. and 3 in. cover depth for 5000 psi Concrete Mixture with 15% FA and 5% SF



- Note:
1. Type II/V portland cement
  2. Corrosion threshold is equal to 500 ppm for black reinforcement.

Figure 36 – Marine Severe Exposure – Service Life Chart for 5000 psi Concrete Mixture with 20% Slag and 5% SF

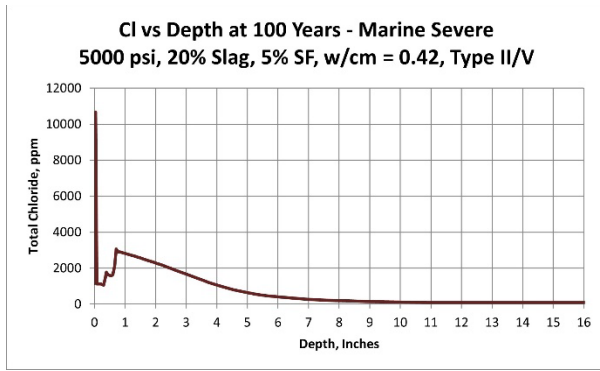


Figure 37 – Marine Severe Exposure – Service Life Chart Displaying the Chloride Ingress by Depth for 5000 psi Concrete Mixture with 20% Slag and 5% SF

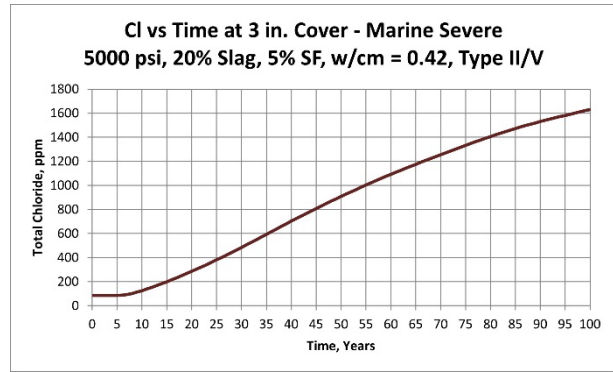
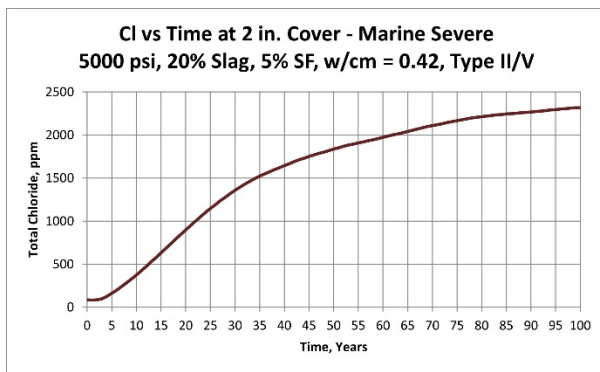
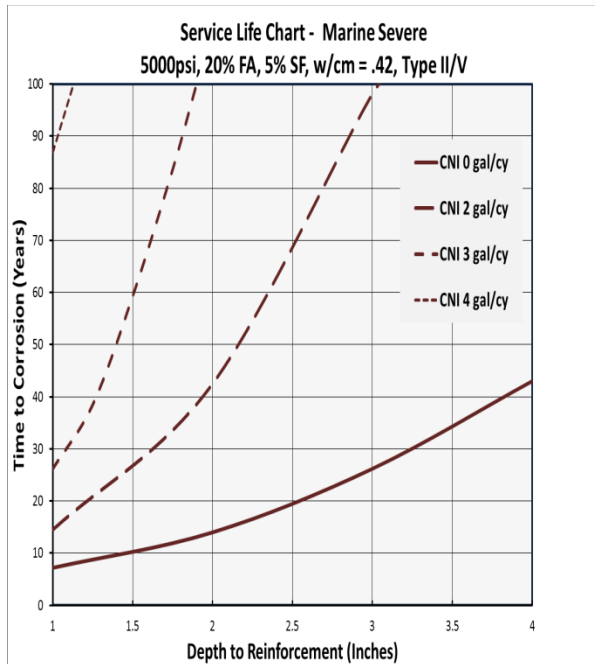


Figure 38 – Marine Severe Exposure – Service Life Chart Displaying Chloride ingress by Time at 2 in. and 3 in. cover depth for 5000 psi Concrete Mixture with 20% Slag and 5% SF



Note: 1. Type II/V portland cement  
 2. Corrosion threshold is equal to 500 ppm for black reinforcement.

Figure 39 – Marine Severe Exposure – Service Life Chart for 5000 psi Concrete Mixture with 20% FA and 5% SF

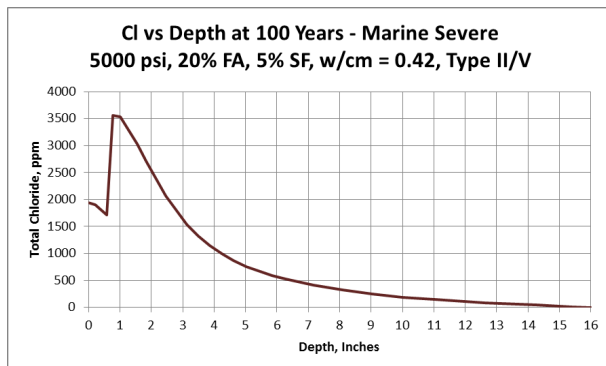


Figure 40 – Marine Severe Exposure – Service Life Chart Displaying the Chloride Ingress by Depth for 5000 psi Concrete Mixture with 20% FA and 5% SF



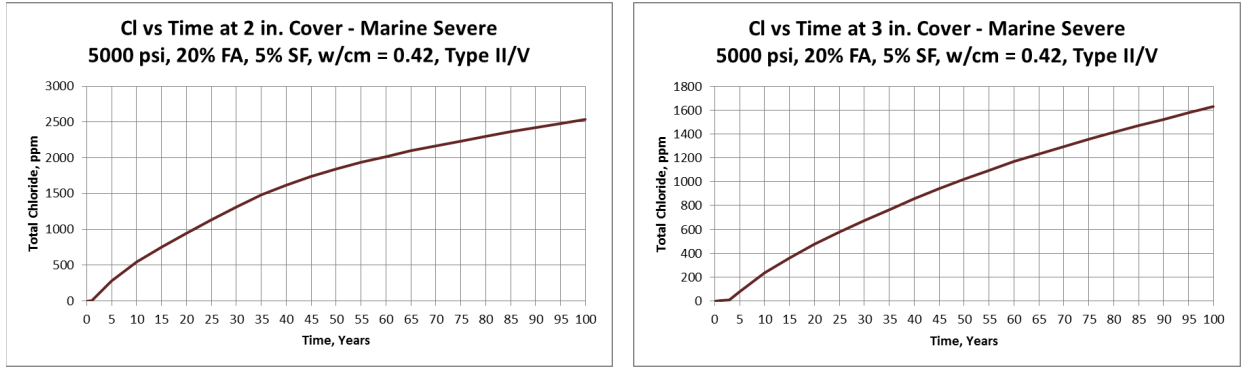
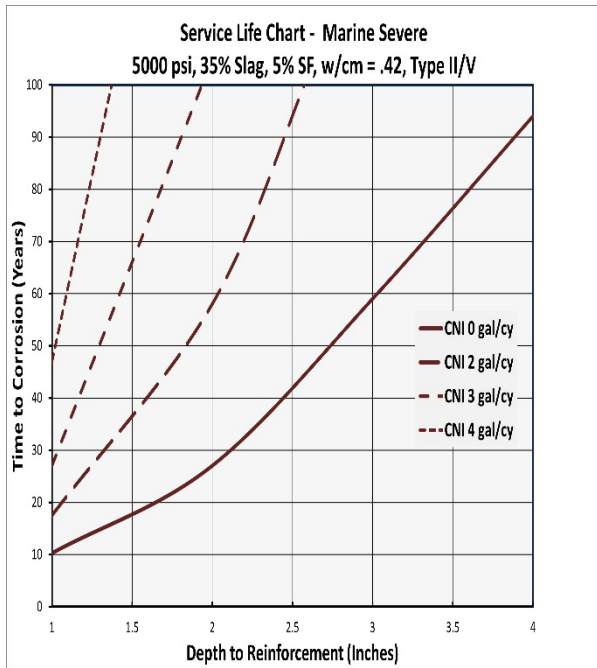


Figure 41 – Marine Severe Exposure – Service Life Chart Displaying Chloride ingress by Time at 2 in. and 3 in. cover depth for 5000 psi Concrete Mixture with 20% FA and 5% SF



Note: 1. Type II/V portland cement  
2. Corrosion threshold is equal to 500 ppm for black reinforcement.

Figure 42 – Marine Severe Exposure – Service Life Chart for 5000 psi Concrete Mixture with 35% Slag and 5% SF

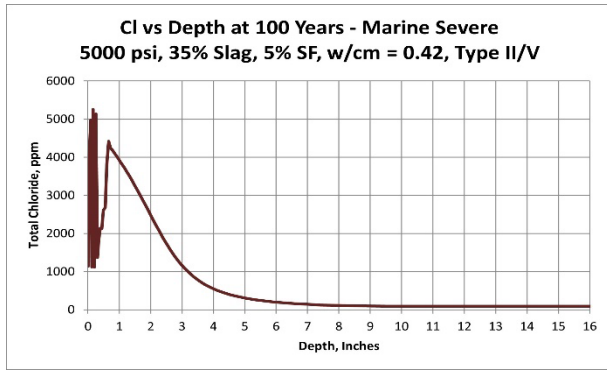


Figure 43 – Marine Severe Exposure – Service Life Chart Displaying the Chloride Ingress by Depth for 5000 psi Concrete Mixture with 35% Slag and 5% SF

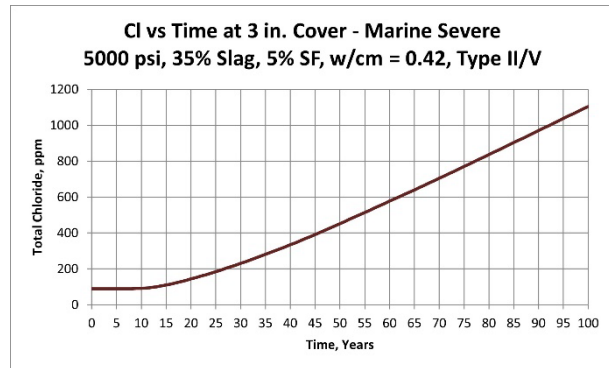
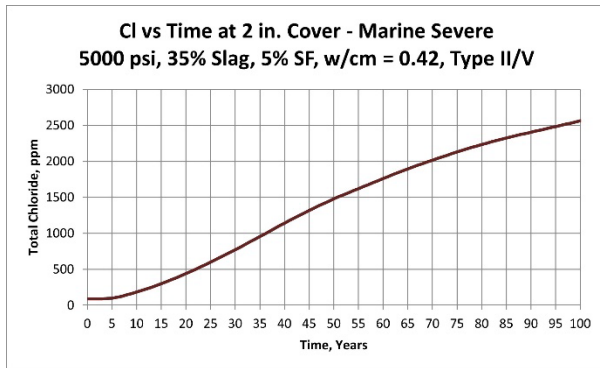
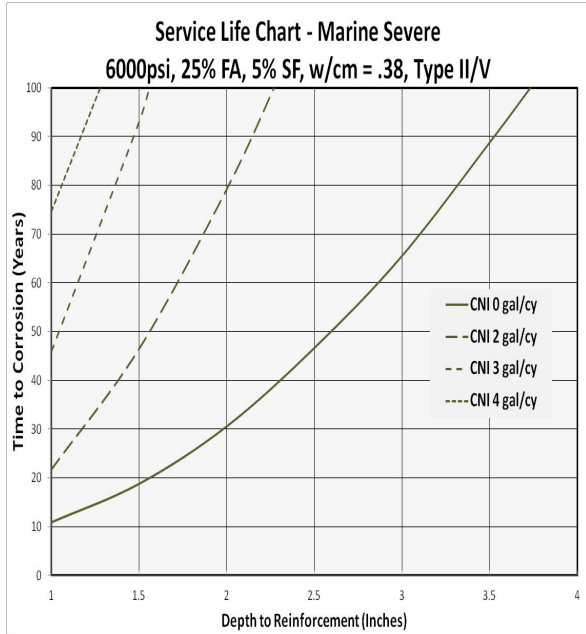
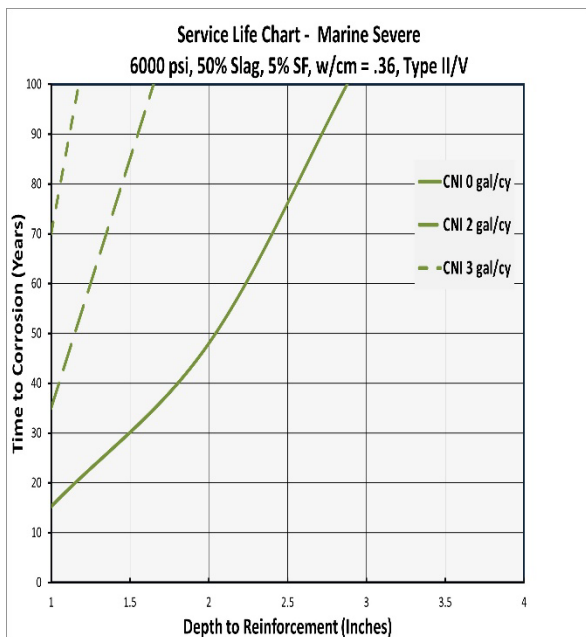


Figure 44 – Marine Severe Exposure – Service Life Chart Displaying Chloride ingress by Time at 2 in. and 3 in. cover depth for 5000 psi Concrete Mixture with 35% Slag and 5% SF



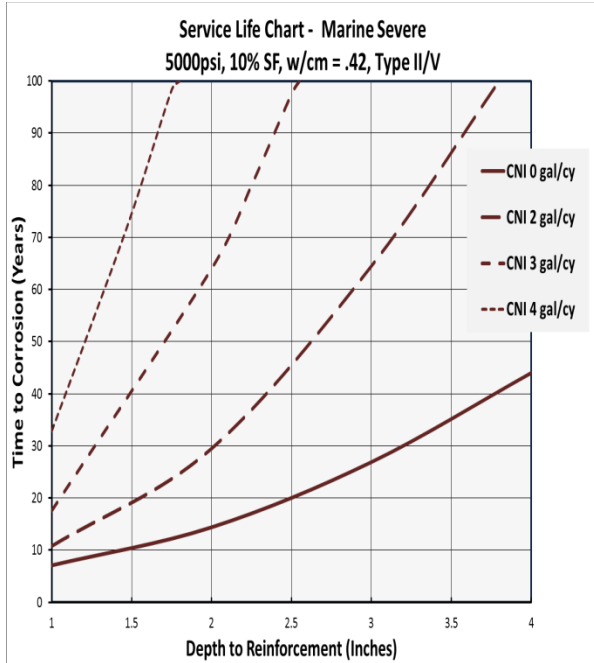
Note: 1. Type II/V portland cement  
 2. Corrosion threshold is equal to 500 ppm for black reinforcement.

Figure 45 – Marine Severe Exposure – Service Life Chart for 6000 psi Concrete Mixture with 25% FA and 5% SF



Note: 1. Type II/V portland cement  
 2. Corrosion threshold is equal to 500 ppm for black reinforcement.

Figure 46 – Marine Severe Exposure – Service Life Chart for 6000 psi Concrete Mixture with 50% Slag and 5% SF



- Note:
1. Type II/V portland cement
  2. Corrosion threshold is equal to 500 ppm for black reinforcement.

Figure 47 – Marine Severe Exposure – Service Life Chart for 5000 psi Concrete Mixture with 10% SF

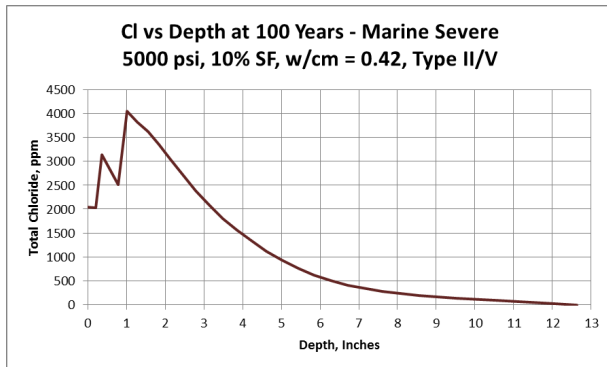


Figure 48 – Marine Severe Exposure – Service Life Chart Displaying the Chloride Ingress by Depth for 5000 psi Concrete Mixture with 10% SF

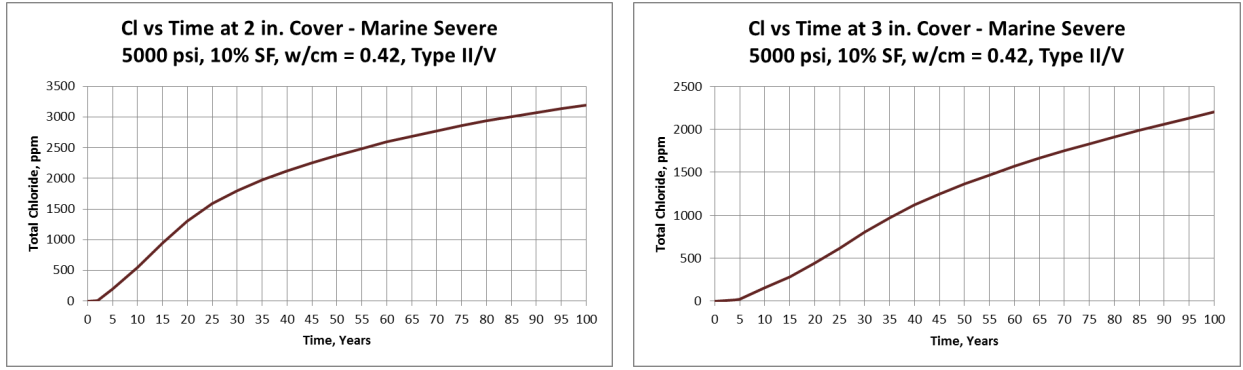
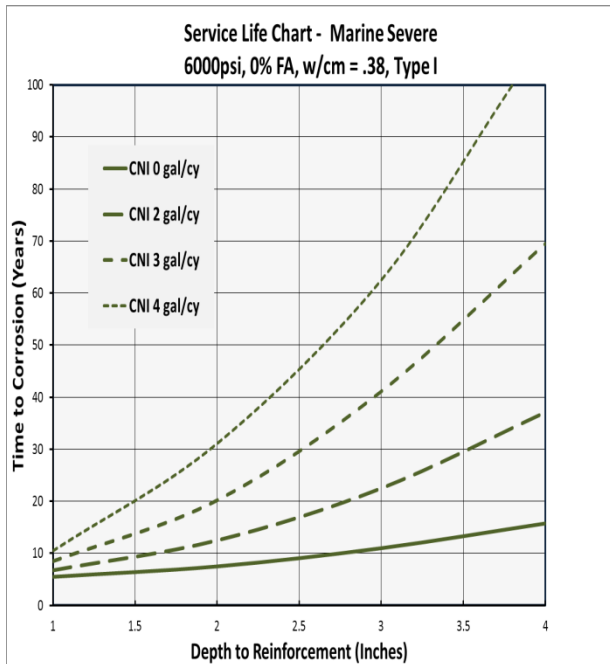
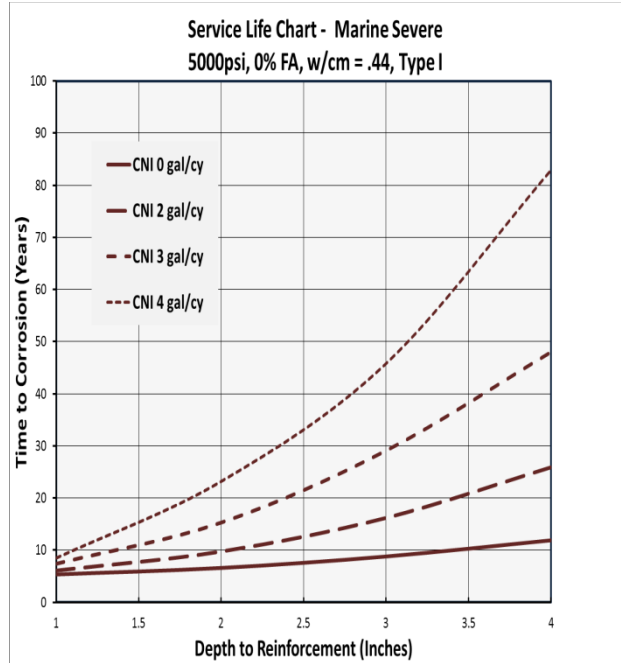
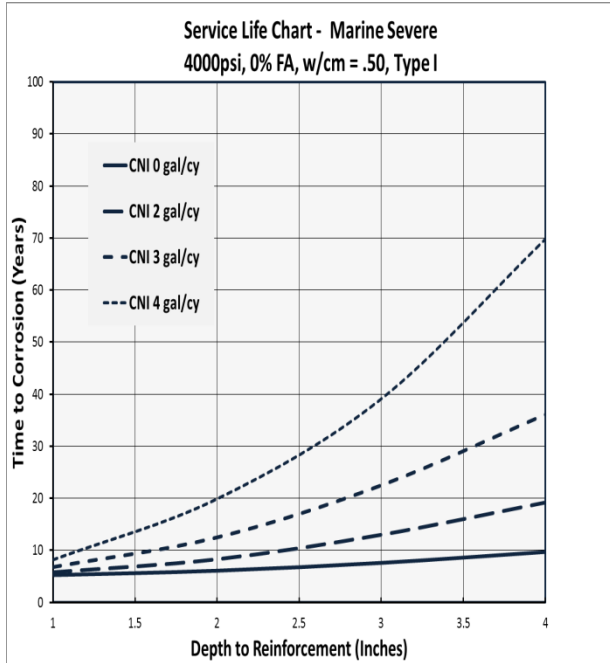
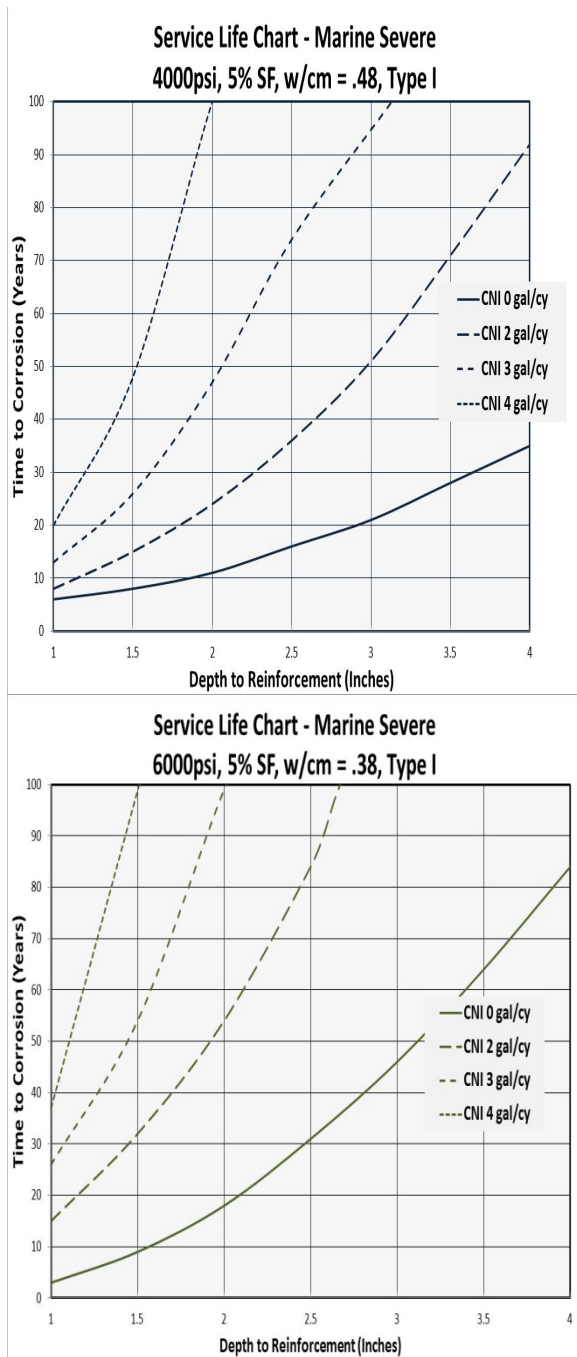


Figure 49 – Marine Severe Exposure – Service Life Chart Displaying Chloride ingress by Time at 2 in. and 3 in. cover depth for 5000 psi Concrete Mixture with 10% SF



- Note:
1. Type I portland cement
  2. Corrosion threshold is equal to 500 ppm for black reinforcement.

Figure 50 – Marine Severe Exposure – Service Life Chart for 4000, 5000, and 6000 psi Concrete Mixture



- Note:
1. Type I portland cement
  2. Corrosion threshold is equal to 500 ppm for black reinforcement.

Figure 51 – Marine Severe Exposure – Service Life Chart for 4000 and 6000 psi Concrete Mixture with 5%SF

## 1.12 Specification Requirements

### Submittal and Acceptance

While many aspects of concrete performance impact the ultimate durability of a structure, there are several key concrete parameters that require confirmation prior to initiating concrete placement. Minimum performance requirements such as compressive or flexural strength are set by the project specification based on the engineering requirements of structure. This document addresses unique requirements related to the durability aspects of concrete. Specifically, drying shrinkage, chloride content, alkali aggregate reactivity, and ion & moisture transport properties. These are considered critical aspects of concrete durability and need to be assessed as part of the submittal and acceptance process.

It is recommended that at least three batches should be produced for each mixture and tested by an accredited laboratory experienced in this field. At least 120 days prior to concrete placement is recommended, submit concrete mixture proportions, ingredient material certificates, trial batch test data, and service life report for each class of concrete proposed for use on the project. Submittal should clearly indicate where each mixture will be used when more than one mix design is submitted. Durability and service life approval must be obtained prior to placement.

If the contractor or materials supplier changes materials, material type, material class, chemical composition, material sources or suppliers, and or mix proportions, the contractor should provide the specifier with the necessary material information to determine the significance of the change on the durability of the material and additional testing requirements.

### Drying Shrinkage

Determine drying shrinkage of concrete mixture(s). A drying shrinkage test result should be the mean value from three or more individual specimens constituting a test set. Casting more than three specimens for each set is permitted. Test procedures and test specimens should conform to the following:

- Drying shrinkage specimens, typically 3 by 3 by 11.25 inch prisms for 1-inch maximum size aggregate or smaller, should be fabricated, cured, dried, and measured in the manner outlined in ASTM C157/C157M except as modified herein:
- Obtain original length measurements after demolding and conditioning as described in the standard.
- Cure specimens in lime water for 7 days maintained at 73 degrees F and then obtain the initial length measurements.
- Place specimens in the drying environment as described in the standard and obtain periodic length measurements after 7, 14, 21, and 28 days of drying (36 days age).
- Calculate the drying shrinkage length change as a percentage of the initial length obtained after 7 days of curing.
- Report individual measurements in tabular format and plot the mean value at each test age.
- The drying shrinkage after 21 days of drying (28 days) should be less than 0.04%.

### Alkali-Aggregate Reactivity

Aggregate tests should be conducted within 120 days from the date of concrete mixture submittal.



- Provide ASTM C1260 or ASTM C1567 test results conducted with 4 months of the submittal date showing the proposed coarse and fine aggregates are either: innocuous to alkali silica reaction; or that reactivity has been mitigated by the proposed cementitious materials as modified herein.
- Conduct ASTM C1260 tests on each aggregate source separately. The maximum allowable expansion should be the limit currently specified by CALTRANS. If this criterion is not met, conduct ASTM C1567 tests on the aggregate using the proposed blend of cementitious materials intended for use on the project. The maximum allowable expansion for the ASTM C1567 test should be the limit currently specified by CALTRANS.
- Aggregates that fail to meet the alkali silica reactivity criteria should be rejected.
- Historic usage or listing on State DOT approved source list is not a substitute for this requirement.

**Water Soluble Chloride Ion Content**

Determine the chloride ion content of concrete mixture(s). Determine water soluble chloride ion content in accordance with ASTM C1218/C1218M. The maximum water soluble chloride ion concentrations in hardened concrete at ages from 28 to 42 days contributed from the ingredients including water, aggregates, cementitious materials, and admixtures should not exceed the limits of Table 26 below.

Table 26 – Maximum Chloride Ion Content for Corrosion Protection

Type of Member	Maximum water soluble chloride ion (C1) in concrete, percent by weight of cement
Prestressed concrete	0.06
Reinforced concrete exposed to chloride in service	0.08
Reinforced concrete that will be dry or protected from moisture in Service	0.15

**Transport properties**

**Transport properties** are required as inputs into the service life modeling software include: The volume of permeable voids (porosity); the ion diffusion coefficient (IDC); the moisture transport coefficient (MTC), and an aging factor. A brief description of the test procedures is provided below.

- Porosity: The volume of permeable voids (porosity) of concrete is determined in accordance with ASTM C642. Porosity is determined for pre-qualification of concrete mixtures and for quality acceptance testing.
- Ion Diffusion Coefficient (IDC): This test method uses an electrical field to migrate chloride ions through concrete specimen for approximately 14 days. The electrical charge flowing through the concrete is related to the diffusion coefficient of ionic species in cementitious materials. A test is defined as the average of two specimens run together and who’s results are analyzed together

to produce a single IDC value. The IDC is determined for pre-qualification of concrete mixtures and for quality acceptance testing. This test method is not yet adopted by ASTM, so the test method will be provided by the modeling software provider.

- Moisture Transport Test: This test method determines the drying rate of pre saturated cementitious materials by measuring the evaporative mass loss of concrete slices with different thickness exposed to constant temperature and relative humidity environment. This test method is not yet adopted by ASTM or AASHTO, so the test method is provided by the modeling software provider. The MTC is most useful in modeling chloride ingress in cyclic wet-dry exposures, or when ions move primarily through evaporative transport.
- Aging factor: The aging factor is used by service life modeling software to estimate the change in diffusion coefficient over time. The aging factor is defined as the ratio of the ultimate IDC to the 28-day IDC. Since we cannot wait for two or more years to characterize project concrete, a surrogate aging factor is used in this specification. This surrogate aging factor is calculated based on the 28-day IDC value, 90-day IDC value, and accelerated 90-day cure IDC value.

The submittal will include a report on the service life of the proposed concrete system using service life modeling software and: the environmental exposures, the characteristic porosity and IDC and MTC values; the aging factor; the modeling concrete cover; the chloride threshold for the proposed reinforcing steel and corrosion propagation time.

#### **Quality control/ Quality assurance**

After initial submittal and acceptance, the following QC/QA testing is required for durability purposes. Other, more traditional, QC/QA testing, such as compressive strength or plastic concrete properties, is not addressed in this durability guide. This QC/QA section is a SUPPLEMENT to, not a replacement for, the QC/QA requirements of the project specification.

Any significant changes materials, material type, material class, chemical composition, material sources or suppliers, and or mix proportions, necessitates the full durability submittal testing (drying shrinkage, chloride content, alkali aggregate reactivity, and ion & moisture transport properties) be repeated.

For ongoing placement of existing approved concretes, the following additional testing requirements are suggested: Test representative concrete cylinders for porosity and ion diffusion coefficient for each day of production or 100 CY of concrete placed, whichever is greater. Determine the ionic diffusion coefficient (IDC). When tested concrete with IDC and porosity values greater than the acceptance values determined during submittal, retesting using spare cylinders and/or re-simulation of the service life using these higher values will be conducted. If the retest exceeds the quality acceptance limit or the revised simulation does not meet the service life requirements, this should be grounds to stop concrete placement and to review quality control issues.

## Appendix

### Appendix A - Terminology

The following terminology is used throughout this chapter.

**AASHTO** – American Association of State Highway and Transportation Officials.

**ACI** – American Concrete Institute.

**Air entraining admixture** – an admixture that causes the development of a system of microscopic air bubbles in concrete, mortar, or cement paste during mixing, usually to increase its workability and resistance to freezing and thawing.

**Alkali-silica reaction (ASR)** – the reaction between the alkalis (sodium and potassium) in portland cement and certain siliceous rocks or minerals, such as opaline chert, strained quartz, and acidic volcanic glass, present in some aggregates; the products of the reaction may cause abnormal expansion and cracking of concrete in service.

**ASTM** – American Society for Testing and Materials (ASTM International).

**CALTRANS** – California Department of Transportation

**Carbonation** – Reaction between carbon dioxide and a hydroxide or oxide to form a carbonate, especially in cement paste, mortar, or concrete; the reaction with calcium compounds to produce calcium carbonate.

**Chemical admixtures** – Chemical additives produced in accordance with ASTM C494 that are capable of altering the fresh or hardened properties of concrete. ASTM C494 designates chemical admixtures as follows:

Type A – Water-reducing

Type B – Retarding

Type C – Accelerating

Type D – Water-reducing and retarding

Type E – Water-reducing and accelerating

Type F – Water-reducing, high range

Type G – Water-reducing, high range and retarding

Type S – Specific performance (ex: corrosion-inhibiting or shrinkage-reducing)

**Chloride** – There are two measurable chloride contents:

**Acid-soluble chloride ions** – In most cases, equal to the total amount of chloride in hydraulic-cement systems. This includes chlorides that have chemically reacted to form cementitious phases and are absorbed onto the cementitious phases, as well as chloride in the capillary pores.

**Water-soluble chloride ions** – Chloride ions dissolved in the capillary pore water. These chloride ions capable of leading to the initiation or acceleration of the corrosion of reinforcement in hydraulic-cement systems.

**Chloride profile** – The distribution of acid-soluble or water-soluble chloride ions contained in the body of concrete as a function of depth

**Corrosion** – The deterioration of metal by a chemical, electrochemical, or electrolytic reaction within its environment.

**Corrosion threshold** – The amount of chloride ions sufficient to initiate corrosion of reinforcement.

**Time to corrosion** – The time required for initiation of corrosion in reinforcement to occur.

**Corrosion inhibiting admixture** – A chemical admixture capable of decreasing the corrosion rate of reinforcement in concrete. Corrosion inhibiting admixtures should meet the requirements of ASTM C 1582 Standard Specification for Admixtures to Inhibit Chloride-Induced Corrosion of Reinforcing Steel in Concrete.

**Corrosion Protection System (CPS)** – The components used in the service life design to mitigate degradation of the concrete. The basic Corrosion Protection System is typically, the depth of cover, concrete mixture, and type of reinforcement.

**Corrosion Threshold** – An amount of chloride content in the concrete at which corrosion is initiated.

**Degradation mechanism** – A distressful process that damages concrete.

**Expansive Hydraulic Cement** – Hydraulic cement that, when mixed with water, produces a paste that, after setting, increases in volume to a significantly greater degree than does portland-cement paste; used to compensate for volume decrease due to shrinkage or to induce tensile stress in reinforcement (post-tensioning).

**Exposure zone** – Relative degree of exposure severity to chloride and/or sulfate ions. This Design Guide contains four exposure zone classifications: (1) Inland (Low), (2) Inland (Moderate), (3) Marine (Moderate), and (4) Marine (Severe). The exposure zone is also referred to in the Design Guide as a “Boundary Condition.”

**Fly ash** – Fly ash is the spherically shaped amorphous, glassy residue that results from the combustion of pulverized coal in electric generating stations. Classified as a supplementary cementitious material.

**Granulated blast-furnace slag** – The glassy, granular material formed when molten blast-furnace slag is rapidly chilled, as by immersion in water.

**Ground granulated blast-furnace slag (GGBFS)** – Granulated blast-furnace slag that has been finely ground and is a low reactivity hydraulic cement. Classified as a supplementary cementitious material.

**High-reactivity metakaolin (HRM)** – a processed highly reactive aluminosilicate pozzolan. Classified as a supplementary cementitious material.

**Hydraulic cement** – A cement that sets and hardens by chemical interaction with water and is capable of doing so underwater.

**Portland cement** – Cement that are manufactured in accordance with ASTM C 150.

Type I – For use when the special properties specified for any other type are not required.

Type II – For use when moderate sulfate resistance or moderate heat of hydration is desired.

Type III – For use when high early strength is desired.

Type IV – For use when a low heat of hydration is desired.

Type V – For use when high sulfate resistance is desired.

**Permeability** – The property that defines the rate of flow of a fluid, or ions in the fluid into concrete.

**Porosity** – See Transport Properties.

**Propagation time** – The time between corrosion initiation and corrosion product build up to first spalling.

**Reinforcement:**

Black steel – Reinforcement steel fabricated in accordance with ASTM A615.

Epoxy coated reinforcement (ECR) – ASTM A 775

Galvanized reinforcement – ASTM A767

Stainless steel reinforcement – ASTM A955

**Segregation** – The differential concentration of the components of mixed concrete, aggregate, or the like, resulting in nonuniform proportions in the mass.

**Service life** – The time in service until a defined unacceptable state is reached, such as spalling of concrete, safety level below acceptable, or failure of elements.

**Shrinkage** – Decrease in either length or volume.

Carbonation shrinkage – shrinkage resulting from carbonation.

Drying shrinkage – shrinkage resulting from loss of moisture.

Plastic shrinkage – shrinkage that takes place before cement paste, mortar, grout, or concrete sets.

Settlement shrinkage – a reduction in volume of concrete before the final set of cementitious mixtures, caused by settling of the solids and displacement of fluids.

Chemical shrinkage – the reduction in volume due to the hydration products of cement having a smaller volume than the original materials before hydration.

Autogenous shrinkage – the reduction in volume that occurs when water is consumed in the cement hydration process causing the RH in the pores to decrease resulting in shrinkage stresses. This contrasts drying shrinkage in which the water evaporates from the concrete causing the shrinkage stresses. The chemical shrinkage is included in the autogenous shrinkage as defined by ACI.

**Silica fume** – very fine non-crystalline silica produced in electric arc furnaces as a byproduct of the production of elemental silicon or alloys containing silicon. Classified as a supplementary cementitious material. Also known as microsilica.

**Slag cement** – see Ground granulated blast-furnace slag (GGBFS).

**Spall** – a fragment, usually in the shape of a flake, detached from a larger mass by a blow, by the action of weather, by pressure, or by expansion, such as by corrosion of reinforcement, within the larger mass.

**Sulfate attack** – either a chemical reaction, physical reaction, or both between sulfates usually in soil or ground water and concrete or mortar; the chemical reaction is primarily with calcium aluminate hydrates in the cement paste matrix, often causing deterioration.

**Supplementary cementitious material (SCM)** – SCM's are materials that contribute to the properties of concrete when used in conjunction with portland cement by reacting either hydraulically or pozzolanically.

**Type K Cement** – Shrinkage-compensating cement.

**Ternary mixtures** – The use of two or more supplementary cementitious materials in a concrete mixture.

#### **Transport Properties:**

**Ionic diffusion coefficient (IDC)** – Ion Diffusion Coefficient (IDC): IDC is a measure of the tortuosity of concrete and relates to ion migration without chemical reactions. Lower values correspond to lower permeability. This test method uses an electrical field to migrate chloride ions through a vacuum saturated concrete specimen for approximately 14 days.

**Porosity** – The volume of permeable voids (porosity) of concrete is determined in accordance with ASTM C642.

**Moisture transport coefficient (MTC)** – MTC is a property related to the rate of moisture loss at an external relative humidity of 50%. The tortuosity of the concrete has a major effect on this value. The MTC decreases as the permeability of the concrete decreases. This test method determines the drying rate of pre saturated cementitious materials by measuring the evaporative mass loss of concrete slices with different thickness exposed to constant temperature and relative humidity environment.

**Type II/V cement** – Portland cement meeting the standard physical and compositional requirements for both Type II and Type V, as specified in ASTM C 150.

**Type 1-modified cement** – There are pre-blended cements commercially available but at this time, they are not widely used in Southern California. Modified cements typically have a pozzolan interground with the cement. This guide includes durability curves for a Type I/Fly ash blend to illustrate the performance of this cement. However, additional modeling is required to confirm the performance if a Type1 modified cement is considered for use.

**Appendix B - STADIUM<sup>®</sup> MODEL**

SIMCO Technologies Inc. has developed, in collaboration with Laval University, a numerical modeling software, called STADIUM<sup>®</sup>, for the prediction of the degradation of concrete structures exposed to chemically aggressive environments. In addition to the diffusion of ions and moisture, the model also accounts for the effects of dissolution/precipitation reactions on the transport mechanisms.

During the course of its development, the numerical results yielded by STADIUM<sup>®</sup> were systematically validated on the basis of laboratory test results and field exposure observations (Marchand 2001, Marchand et al. 2001, Maltais 2002) and field exposure observations (Marchand et al. 2002a, Marchand et al. 2002b, Marchand et al. 2002c). The model was also used to predict the behavior of numerous existing structures exposed to various forms of chemical degradation phenomena.

**DESCRIPTION OF STADIUM<sup>®</sup>** Over the past few years, the mechanisms of ionic transport in cement systems have been the subject of increasing attention. Most of the reports published on the topic have clearly emphasized the intricate nature of the problem. Given the number of parameters involved, analytical models have difficulty describing the process of ionic transport, and therefore, numerical modeling required.

The main features of STADIUM<sup>®</sup> (a numerical model) are described below. STADIUM<sup>®</sup> predicts the transport of ions in cement-based materials and the chemical modifications occurring to the material as a result of these ionic movements. The current version of STADIUM<sup>®</sup> is applied to 1D and axisymmetric problems.

STADIUM<sup>®</sup> has two main modules. The first module accounts for the coupled transport of ions and water without considering any chemical (dissolution/precipitation) reactions. The transport is modeled with a volume-averaged version of the extended Nernst-Planck equation. This equation accounts for the electrical coupling between ions as well as for the chemical activity of the various species in solution. The equation is coupled to Poisson's equation, which gives the electrical potential of the solution as a function of the local concentrations in ions. The following ionic species are usually considered: OH<sup>-</sup>, Na<sup>+</sup>, K<sup>+</sup>, SO<sub>4</sub><sup>2-</sup>, Ca<sup>2+</sup>, Al(OH)<sub>4</sub><sup>-</sup>, Cl<sup>-</sup>. These ions allow STADIUM<sup>®</sup> to treat a wide range of chemical degradation phenomena that may affect concrete structures in service: chloride penetration, external sulfate attack, calcium leaching, etc. If needed, additional ionic species can be considered to predict the performance of concrete subjected to various deterioration mechanisms: Mg<sup>2+</sup> for magnesium sulfate attack, NO<sup>2-</sup> for concrete made with corrosion inhibitor, etc.

To account for the simultaneous transport of water, the previous equations are coupled to Richard's equation. It is a diffusion-type equation that gives the distribution of water content throughout the material. The effect of water movement of ions is modeled by adding an advection term to the extended Nernst-Planck equation.

To solve the system of nonlinear equations, a numerical algorithm must be used. All the transport equations are solved simultaneously. The spatial discretization of this coupled system is performed through the finite element method, according to the standard Galerkin procedure. A Euler implicit scheme is used to discretize the transient part of the model. The nonlinear set of equations is solved with a Newton-Raphson algorithm. This second order algorithm gives a good convergence rate and is robust enough to handle the electrical coupling between the ionic species as well as the non-linearities coming from the coupling between the ionic flux and the water movement.



The second module in STADIUM® is a chemical equilibrium code. After a transport step, the chemical equilibrium module verifies if the concentrations at each node of the finite element mesh are in equilibrium with the different solid phases of the hydrated cement paste: calcium hydroxide, calcium silicate hydrates, ettringite, and hydrogarnet. If the solution is not in equilibrium with the paste, solid phases are either dissolved or precipitated in order to bring back equilibrium. Solid phases can also be formed as a result of the penetration of aggressive species into the porous network of the material: brucite, ettringite, gypsum, Friedel's salt, hydrated sodium sulfate, and halite. The variation of solid phases will lead to a local alteration of the porosity. This is likely to affect the transport properties of the material. STADIUM takes this phenomenon into account in its transport module.

### **STADIUM® Modeling**

STADIUM® was originally developed in the late 1990s and uses time-step finite element analysis to simulate the ingress of harmful ions (including chloride, sulphate, and hydroxide) through concrete, by considering the chemical and physical properties of the concrete being studied. The modelling program was developed using funding from the US Navy, which needed a tool to accurately predict the service life of waterfront concrete structures at its many Naval bases around the World.

The modelling program is now used worldwide by owners, engineers, and contractors to predict service life of concrete infrastructure exposed to aggressive environments, such as deicing salts, seawater, and sulfate-bearing soils.

With STADIUM®'s comprehensive approach, SANDAG will achieve a substantial return on investment. Based on sound technical analysis, use of this model and guide can reduce lifecycle costs, avoid unnecessary inspection and maintenance costs, and extend the service life of structures.

STADIUM® has been thoroughly validated through extensive laboratory experiments and field observations. More than 20 peer-reviewed papers published in leading international journals document the model's performance. STADIUM® has been designed to meet the specific requirements of the US Navy as well as for the needs of transportation, marine, and parking infrastructure and provides comprehensive databases of structural elements, concrete mixture properties, exposure conditions, and standards that enable users to make informed decisions leading to the optimal service life of concrete infrastructure.