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Corrosion Protection Plan for Harry Nice/Middleton Bridge over Potomac River in Newburg, Maryland

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Abstract

This paper presents a comprehensive durability and service life evaluation of reinforced concrete bridge components of the Nice/Middleton Bridge located in Newburg, Maryland. Methodology for durability design is presented in this paper. Long-term performance, maintenance planning, and rehabilitation strategies for structures exposed to aggressive chloride environments are discussed. This paper focuses on analysis of chloride-induced corrosion mechanisms affecting concrete performance, with particular emphasis on chloride diffusion properties, exposure conditions, relevant design details, and the application of mechanistic service life modeling.

Concrete durability was assessed through laboratory-derived diffusion properties, chloride loading, and exposure classification. Reinforcing steel performance was evaluated using corrosion threshold limits, electrochemical principles, and propagation models.

A performance-based service life modeling approach was employed to evaluate time to corrosion initiation and subsequent deterioration under multiple scenarios, including existing environmental conditions and proposed plans to achieve service life. The results demonstrate how project-specific material data can be used to reduce uncertainty, compare durability alternatives, and support rational, risk-informed decisions for designing durable transportation infrastructure.

Key Words: Design Build, Progressive design build, CMAR, Performance based, Service life design, Durability analysis, Chloride Diffusion Analysis, Chloride Induced Corrosion, Concrete bridges, Tunnels, Marine/Port structures, Prestressed concrete girders, Reinforced concrete, drilled shafts, Steel pile piles, and Supplemental Cementitious Materials (SCMs).

1. Background and Introduction

1.1 Background

State Departments of Transportation increasingly require bridge structures to be designed for 100-year life or extended service lives up to and beyond 100 years. Achieving the target design life in regions exposed to deicing salts, freeze–thaw cycling, and moisture requires a shift from prescriptive material requirements toward performance-based durability design. Chloride-induced corrosion of reinforcing steel in concrete (and soil/water induced corrosion for steel elements) remains the primary deterioration mechanism for bridge elements in different climates, frequently governing long-term maintenance and rehabilitation needs. SCS has developed an approach to design build process that is holistic, cost-effective, and considers each bridge element individually while verifying the durability of the entire bridge. This paper discusses the Nice/Middleton Bridge in Newburg, Maryland, and how SCS created a comprehensive corrosion protection plan to meet the design service life, up to 100 years.

The Nice/Middleton Bridge on Route 301 spans Potomac River connecting Newburg, Maryland and King George, Virginia. The existing bridge was replaced by a new bridge. The overall view of the new bridge can be seen in Figure 1. The structure consists of 27 low level trestle spans on the west (Virginia) side, 15 high level approach spans on the west (Virginia) side, three main spans, and 14 high level approach spans on the east (Maryland) side.

Figure 1. Graphic of Bridge



1.2 Design Service Life Requirements

SCS developed a corrosion protection plan that would enable each element to meet the design service life (See Table 1) defined by The Maryland Department of Transportation (MDOT) and the Maryland Transport Authority (MDTA).

Table 1. Service Life requirements for Bridge Elements

Structure Element Service Life	Service Life
Deck, Barriers, Prestressed Concrete Girders including prestressing cables, Steel Girders, Concrete Piles, Piers, Bent Caps, Pier Caps, Expansion Joint Troughs, Footings, Abutment, Approach Slabs, Drain Trough	100
Tubs for Waterline Footings, Fender Cap and Piles	75
Bridge Bearings	40
Expansion Joints, Bridge Rail / Approach Guide Rail, Electrical and Mechanical Parts	30
Bridge Fender Access Ladders and Platforms	50
Overhead Sign Structures	40
Coatings, Galvanizing or Metalizing of Structural Steel	33

Approach to Service Life

According to the project controlling document (a.k.a. Minimum requirement) the bridge "shall be considered to be subjected to severe corrosive conditions" due to deicing salts. It is also required that the service life design of bridge concrete elements to be in accordance with the fib Bulletin 34 Model Code for Service Life Design [1].

For concrete components, chloride-induced corrosion was modeled using fib Bulletin 34 Model Code for Service Life Design (a.k.a. fib 34). The end of service life for the concrete elements was defined as 10% probability of initiation of corrosion of reinforcement [1]. This is the time when the concentration of chloride at the reinforcement level reaches the critical threshold value, initiating the corrosion of steel.

Avoidance or mitigation methods are used to achieve the required service life for deterioration mechanisms other than chloride-induced corrosion. One of the following methods was identified for providing the required design service life for each potential degradation mechanism:

- Avoidance: Avoid the degradation mechanism.
- Deemed-to-Satisfy: Apply supplementary protective measures that are deemed-to-satisfy the durability requirements for the required period of time.
- Material Resistance: Select materials and details which resist the degradation mechanism for the required period.

For buried pipe piles, sacrificial thickness was calculated based on the steady state rate of corrosion of steel (exposed to in-situ soil/water) and the required service life. For paint systems, service life was defined as time "until 5 to 10% coating breakdown occurs and active rusting of the substrate is present".

An example of avoidance is use of non-reactive aggregates to avoid the possibility of alkali-silica reaction degradation. An example of deemed-to-satisfy is air-entrainment in concrete (that has minimum required porosity, the size and distribution air voids within hardened concrete) that is deemed satisfactory to resist freeze-thaw damage based on industry standards and experience. An example of material resistance is the fib 34 service life model for chloride induced corrosion of reinforced concrete elements (concrete/reinforcement or a combination of both). Chlorides are allowed to enter the concrete; however, a model and limit state are used to determine the type of concrete (mix) that will resist corrosion degradation over the design life.

2. Methodology

2.1 Overall Assessment Framework

The durability evaluation followed a systematic, multi-step framework for performance-based service life assessments. The primary steps included:

1. Review of structural details, material specifications, and exposure conditions as well as discussion with the design team.

2. Laboratory testing to characterize concrete mix properties

3. Separate analytical evaluation of concrete and steel deterioration mechanisms

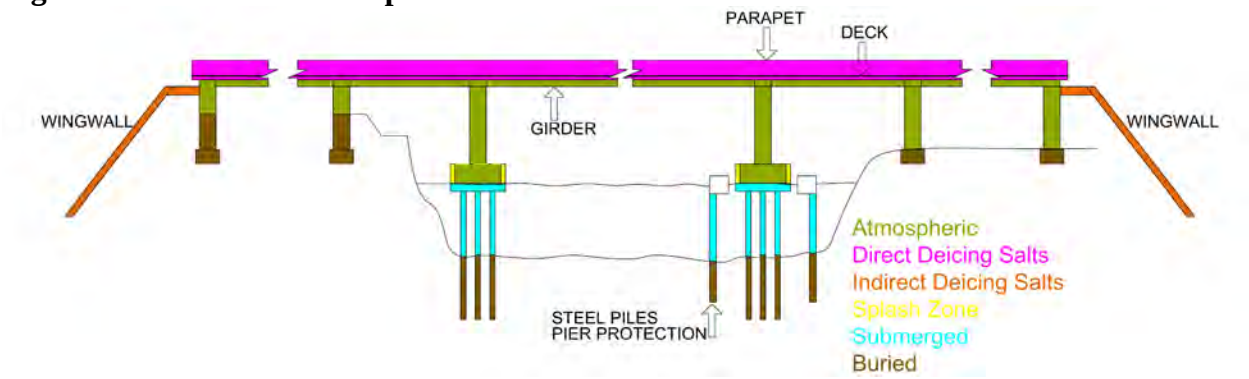
4. Integration of results through service life modeling

This approach allows individual deterioration mechanisms to be examined independently while still capturing their interaction in the overall service life analysis.

2.2 Exposure Zone Analysis

The components of the new Nice/Middleton Bridge will be subject to multiple environments (see Figure 2).

Figure 2. Environmental Exposure Conditions



The following exposure zones have been defined for various components:

- Buried: Component permanently buried in soil.
- Submerged: Component permanently submerged in brackish river water.
- Splash Zone: Component at the river water line, subject to wet-dry cycles.

- Direct de-icing salts: Components directly exposed to the use of de-icing salts.
- Indirect de-icing salts: Components exposed to runoff water or spray containing de-icing salts.
- Atmospheric: Components not exposed to any of the above environments.

SCS separated each element by its exposure condition. Each environment has unique conditions that, unless addressed individually, could have a negative impact on the service life of the bridge.

2.3 Service Life Modeling Methodology

Service life modeling was conducted using mechanistic-empirical approaches.

Deterioration was divided into two phases:

- **Initiation Phase:** Time required for chlorides to penetrate the concrete cover and reach the reinforcing steel at a critical threshold

- **Propagation Phase:** Time from corrosion initiation to the development of unacceptable damage, such as cracking, spalling, or section loss

The primary emphasis was placed on characterizing the initiation phase, as this period typically represents a significant part of the service life for well-designed concrete structures and is most directly influenced by material selection, detailing, and construction practices.

3. Concrete Durability Analysis

3.1 Concrete Materials and Exposure Conditions

Concrete durability depends on concrete mix composition, curing conditions, and in-service exposure. Factors influencing performance included cementitious material type and content, water-to-cementitious materials ratio, curing duration, and environmental condition. Exposure conditions such as deicing salt application, splash and spray zones, buried, and submerged conditions were explicitly considered.

Structural elements were grouped by exposure severity to reflect differing risks of chloride ingress and surface deterioration. Elements subjected to direct or indirect deicing salt exposure were analyzed using chloride loading (data from nearby structures or based on authors database) than those located in atmospheric or buried/submerged environments.

Other deterioration mechanisms considered were Alkali Silica Reactivity (ASR), Sulfate Attack, Delayed Ettringite Formation (DEF), Freeze-Thaw/Scaling, and Carbonation Induced Corrosion.

These deterioration mechanisms are deemed-to-satisfy the service life requirement should they meet the criteria listed in Table 2. All concrete mixes were required to pass the criteria listed in Table 2 unless specified otherwise.

Table 2. Required Concrete Testing

Test No.	Test Name	Criteria	Prequalification Test	Control and Acceptance Tests
ASTM C1260	Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)	≤0.10% No mitigation Required >0.10% and ≤0.30% Mitigation Required, >0.30% Not Allowed	x	
ASTM C1567	Standard Test Method for Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar-Bar Method)	≤0.10%	x	
AASHTO T 104	Standard Method of Test for Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate	12% Max. Soundness Loss in Coarse Aggregates and 10% Max. Soundness Loss in Fine Aggregates using 5 Cycles Testing	x	
ASTM C457	Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete	≥ 600in²/in³ ≤ 0.008" Spacing Factor	x	
ASTM C666	Standard test method for resistance of concrete to rapid freezing and thawing	Minimum Durability Factor of 80 (SCC)	x	
ASTM C231/C173 or AASHTO T152/ T196	Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure\Volumetric Method Standard Method of Test for Air Content of Freshly Mixed Concrete by the Pressure\ Volumetric Method	SCC 6.0% ± 1.0% air content CIP (Deck & Barriers Pier Protection and Tubs) 7.0% ±1.0% air content CIP Pier Cap & Columns & Footing 6.0% ±1.0% air content	x	x
ASTM C157 or AASHTO T160	Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete Standard Method of Test for Length Change of Hardened Hydraulic Cement Mortar and Concrete	400 microstrain at 28 days	x	
ASTM C672	Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals	0-1 rating @ 50 Cycles	x	
NT Build 492	Chloride Migration Coefficient From Non-Steady-State Migration Experiments	Perform NT Build 492 and submit results	x	
AASHTO T277	Standard Method of Test for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration	<1500 coulombs	x	x
ASTM C1152	Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete	Initial Chloride Concentration <0.1 % mass of the binder	x	
ASTM C39	Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens	Compressive Strength varies per mix	x	

3.2 Probabilistic Analysis for Chloride Induced Corrosion

The process for achieving service life is listed below:

- SCS calculated the maximum allowable Chloride Migration Coefficient (CMC) using the fib 34 probabilistic service life model.
- Contractor then developed several test concrete mixes.
- SCS tested various trial concrete mixes to document the CMC value and its suitability for specific concrete elements.
- SCS summarized (in a tabular form) the maximum allowable CMC for the specific element and concrete mix combination.

Based on modeling results, concrete mix (permeability), rebar cover, and/or rebar type (e.g., black bar vs. low chromium) can be adjusted to achieve the design life.

3.3 Service life Analysis of Concrete

Since the superstructure is exposed to deicing salts and the substructure is exposed to brackish water, the main deterioration mechanism for reinforced concrete elements is deicing chloride induced corrosion and for the substructure it is the brackish water. As described in previous sections, the maximum allowable chloride migration coefficients were calculated based on minimum required service life, exposure zone, and cover parameters.

3.3.1 Probabilistic Approach and the Parameters

The chloride diffusion is calculated based on the following equation which is the solution to Fick's differential equation:

$$C_{crit} = C(x = a, t) = Co + (Cs, \Delta x - Co) \cdot \left[1 - \operatorname{erf}\left(\frac{a - \Delta x}{2\sqrt{Dapp, C \cdot t}}\right)\right] \quad (1)$$

Which can be calculated based on the following equations:

$$D_{app,C} = k_e \cdot D_{RCM,0} \cdot k_t \cdot A(t) \quad (2)$$

$$k_e = \exp\left(b_e\left(\frac{1}{T_{ref}} + \frac{1}{T_{real}}\right)\right) \quad (3)$$

$$A(t) = \left(\frac{t_o}{t}\right)^{\alpha} \quad (4)$$

$$k_t = 1 \quad (5)$$

The parameters of these equations are defined as follows:

Chloride Migration Coefficient ($D_{RCM,0}$) – This variable describes the rate at which chloride ions can migrate through concrete.

Ambient Temperature (T_{real}) – This is the average temperature distribution of the location in which the structure will be built.

Aging Exponent (α) – The change in chloride migration coefficient over time. Based on the fib 34 [1] and fib 76 [2], a beta distribution was assumed for this parameter with mean and standard deviation values as shown in Table 3.

Table 3. Aging Exponent Mean and Standard Deviation Values

Cement type:	Mean	Standard deviation	Beta Parameters
Type I Portland Cement (PCC)	0.30	0.12	α = 4.08, β = 9.51; interval [0,1]
PCC w/ ≥ 20% Flyash	0.60	0.15	α = 5.8, β = 3.87; interval [0,1]
PCC w/ 5% Silica Fume	0.40	0.16	α = 3.35, β = 5.03; interval [0,1]
PCC w/ Blast Furnace Slag (36% – 65%)	0.40	0.18	α = 2.56, β = 3.84; interval [0,1]
PCC w/ Blast Furnace Slag (66% – 80%)	0.45	0.20	α = 2.33, β = 2.85; interval [0,1]
All types in atmospheric zone	0.65	0.15	α = 5.92, β = 3.19; interval [0,1]

Initial Background Chloride Content (C_o) – This parameter refers to the initial chloride concentration present in the concrete at the time of construction.

Surface Chloride Concentration ($C_{s,\Delta x}$) – This describes chloride content at the environment-to-structure interface.

Transfer Function (Δx) –This corresponds to the convection zone where there is moisture fluctuation and the governing chloride transport phenomenon is not diffusion.

Rebar Cover (a) – The dimension of concrete cover. This is the distance chloride ions must migrate from concrete surface to the top layer of rebar.

Critical Chloride Concentration (C_{crit}) – The amount of chloride required at the rebar level to initiate corrosion. See Table 4.

Table 4. The C_{crit} Parameters for Different Reinforcing Steel Types

Beta Distribution Parameters of C_{crit} (% mass of the binder)	Steel Type		
	Carbon Steel	ASTM A1035	316LN
Mean	0.6	1.49	4.79
Standard Deviation	0.15	0.53	1.53
Lower Bound	0.2	0.9	3.0
Upper Bound	2	3.8	7.1
Beta Parameters	$\alpha = 5.31, \beta = 18.58$	$\alpha = 0.78, \beta = 3.07$	$\alpha = 0.33, \beta = 0.43$

Design Service Life (t_{SL}) – The time that the structure can be expected to function in a passive state (without corrosion initiation).

Target Reliability Index (β) – The recommended value of Reliability Index for corrosion initiation is 1.3, which is approximately equivalent to a probability of corrosion initiation of 10%, or a reliability (of passive state) of 90%.

Regression Variable (b_e): This variable has normal distribution with mean and standard deviation of 4800 °K and 700 °K, respectively.

T_{ref}: Standard test temperature defined as a constant 292.2 °K

Reference Point of Time t_o: This is a constant value of 28 days.

3.3.2 Summary of Probabilistic Assessment

To achieve the design service life with 90% confidence level, the target CMC values were calculated based on the cover, type of reinforcement, surface chloride, exposure zones, ambient temperature etc. The service life analysis was performed for each element using the appropriate concrete cover. The CMC values were calculated based on chlorides reaching threshold value at the reinforcement closest to the concrete surface.

The following sections describe a summary of the key parameters defined for each element's service life analysis.

Low Level Trestle Spans - 36-inch Square Piles:
Material: Self-Consolidating Concrete is used for the low-level square piles, rebar conforming to ASTM A615 grade 60, and tie wires conforming to ASTM A1064.

High Level Spans - 36 in Square Piles:
Material: Self-Consolidating Concrete is used for the piles. Rebars conforming to ASTM A615 grade 60, and the tie wires conforming to ASTM A1064.

Main Spans - 66 in Diameter Cylinder Piles and Concrete Plugs:
Carbon Fiber Reinforced Polymer (CFRP) strands in the cylinder piles. The reinforcing for the concrete plug is ASTM A615 grade 60 and is surrounded by cylinder pile with CFRP reinforcement.

Pile Bent Caps and Pier Caps:
Material: The MDOT Mix No.3 Concrete and ASTM A615 grade 60 rebars are required. The reinforcement at the expansion joints (directly underneath the expansion joint and within the elevation of the superstructure) were epoxy coated.

Deck:
Material: The MDOT Mix No.6 Concrete is used. The rebars are low carbon\chromium ASTM A1035. The exposure zone is the direct deicing. The surface chloride concentrations at the surface were set at 2.5% mass of the binder with a coefficient of variation of 0.75. The cover is 2" with a tolerance of ±0.25". The deck is cast on the metal pan and therefore, the underside is not exposed to chlorides.

Barriers – Interior and Exterior Face:
Material: The MDOT Mix No.6 Concrete and low carbon/chromium ASTM A1035 was used. For the Interior Face, the exposure zone classification is the direct deicing. The chloride concentration at the interior surface was conservatively set at 2.5% mass of the binder with coefficient of variation of 0.75. The cover is 2 in, with a tolerance of ±0.5".

For the exterior part of the barrier, the indirect deicing with a lower surface chloride loading compared to the interior surface. Based on all other variables being the same for both the interior and exterior surface of the barriers, the interior face of barrier controls the chloride migration values since it has higher chloride loading.

PSC Girders - 95” and 79” deep

Material: Self-consolidating 10 ksi concrete is used. The rebars are ASTM A615 grade 60. All girders are at least 2’ above the 100-year flood level, therefore they are categorized as atmospheric zone. The chloride concentration at the surface was set conservatively at 1.5 % mass of the binder. The minimum cover is the nominal 1.25" with the tolerance of ±0.25”.

Drain Trough:

The VDOT drainage trough details are used to completely seal the superstructure from exposure to deicing salt. VDOT -PERM 7.5 KSI mix is used for all precast drain trough and MDTA mix #6 is used for CIP drain trough. The reinforcement is ASTM A615 with epoxy coated. The chloride concentration at the surface was set at 2.5% mass of the binder. The cover is 2” for the typical precast troughs and 2-1/2" for the CIP troughs at the abutments with ±0.5” tolerance.

Footers:

Material: Mix No.3 (3.5 ksi) concrete is used. The rebars are ASTM A615 grade 60.

Footer Casting Tubs:

Material: 7.5 KSI concrete mix was used. Concrete cover was at least 2.0” for stainless steel, ASTM A995 grade 75 Type 316 LN or 2.5” for black steel, ASTM A615 Grade 60.

Abutment Wingwalls:

Material: Mix No. 6 Concrete and the rebars are ASTM A615 grade 60 with epoxy coating was used. The exposure zone that governs is indirect deicing. Surface chloride concentration for indirect deicing conditions was set at 1.875% mass of the binder. The cover is 3” with ±0.5” tolerance.

Abutment Backwall:

Material: The design team decided to use Mix No.6 for backwall. The rebars used for backwall are low carbon chromium steel. The exposure zone that governs is indirect deicing exposure. The chloride concentration at the surface was set at 1.88% mass of the binder. The cover is 3” with ±0.5” tolerance.

Abutment Stemwall:

Material: The design team decided to use Mix No.6 for stemwall. The rebars used for stemwall are ASTM A615 grade 60 with epoxy coating. The exposure zone that governs is atmospheric/buried. The chloride concentration at the surface was set at 1.5% mass of the binder. The cover is 3” with ±0.5” tolerance.

Pier Fender Cap:

The piers are protected from collision using a PC/CIP unit. The concrete is 8 ksi self-consolidating concrete while the rebars were ASTM A615 OR A706 GRADE 80.

The exposure zone that governs is the atmospheric. The chloride concentration at the surface was set at 2 % of the binder. The cover is 2.5” with ±0.5” tolerance.

Approach Slabs:

The exposure zone that governs is the direct deicing, while the chloride concentration at the surface was set at 2.5% mass of the binder with coefficient of variation of 0.75. The cover is 3" with ±0.5" tolerance.

Maximum Allowable Chloride Migration Coefficient for Concrete Elements:

Table 5 shows a summary for maximum allowable chloride migration coefficient in different elements. This was developed based on fib 34 probabilistic service life model. It is necessary that the actual CMC values (based on the results of NT Build 492 test) for concrete mixes should have a mean value less than maximum CMC values. The maximum and the measured CMC values are listed in Table 5.

Table 5. Maximum Allowable Chloride Migration Coefficients for Different Concrete Elements

Concrete Structure Element	Extended Service Life (yrs.)	Reinforcement or Embedded Items	Cover (inches)	Cover Tolerance (inches)	Exposure Zones	Cement Admixture Requirement	Maximum Allowable Chloride Migration Coefficient (in^2./yr)		Mix Passing fib 34 Service Life Criteria	Measured CMC Values per NT Build 492, (in^2./yr) (Must Be Lower Than Max. Listed)
							Mean, μ	Std Dev, σ		
Low Level 36" Square PSC Piles	100	Strand-Plain, Galvanized Spiral	2.4375	0.25	Submerged	Flyash=15%, SF=3%	0.35	0.07	7.5 KSI VDOT PERM	0.198
High-Level 36" Square PSC Piles	100	Strand-Plain, Galvanized Spiral	2.4375	0.25	Submerged	Flyash=15%, SF=3%	0.35	0.07	7.5 KSI VDOT PERM	0.198
Pile Bent Caps & Piers & Pier Caps	100	Plain	2	0.5	Atmospheric	N.A.	0.60	0.12	HNBA3001	0.371
Diaphragms	100	Plain	2	0.5	Atmospheric	N.A.	0.60	0.12	HNBA3001	0.371
66" PSC Cylinder Piles	100	CFRP	2.9	0.25	N.A				DSM Mix, 10 KSI	0.126
Plug in 66" Piles	100	Plain	8.75	0.5	Submerged	N.A.	1.00	0.20	HNBA3001	0.371
Deck	100	Low-Carbon, chromium, Type 1035	2	0.25	Deicing	Flyash>20%	0.50	0.10	HNBA6001	0.381
Barriers	100	Low-Carbon, chromium, Type 1035	2	0.5	Direct & Indirect Deicing	Flyash>20%	0.50	0.10	HNBA6001	0.381
PSC Girders - PCEF-79/95 Typical	100	Plain	1.2	0.25	Atmospheric	N.A.	0.21	0.04	DSM Mix, 10 KSI	0.126
V- Trough at Abutments (CIP)	100	Epoxy-Coated	2.5	0.5	Indirect Deicing	N.A.	0.382	0.076	HNBA6001	0.381
V- Trough at Piers (Precast)	100	Epoxy-Coated	2.	0.5	Indirect Deicing	N.A.	0.20	0.04	7.5 KSI VDOT PERM	0.198
Footing	100	Plain	3	0.5	Atmospheric	N.A.	1.00	0.20	HNBA3001	0.37
Footing tub	75	Plain	2.5	0.5	Splash zone	Flyash=15%, SF=3%	0.20	0.04	7.5 KSI VDOT PERM	0.198
Footing tub	75	Stainless steel	2.0	0.5	Splash zone	Flyash=15%, SF=3%	1.00	0.2	7.5 KSI VDOT PERM	0.198
Abutment Wing Walls	100	Epoxy-Coated	3	0.5	Indirect Deicing	Flyash>20%	0.52	0.10	HNBA6001	0.38
Abutment Backwall	100	Low-Carbon, chromium, Type 1035	3	0.5	Indirect Deicing	Flyash>20%	1.00	0.2	HNBA6001	0.38
Abutment Stemwall	100	Epoxy-Coated	3	0.5	Atmospheric	N.A.	1.00	0.20	HNBA6001	0.38
Pier Fender Cap	75	Plain	2.5	0.5	Atmospheric	N.A.	1.00	0.20	8KSI VDOT	0.168
Approach Slabs	100	Low-Carbon, chromium, Type 1035	3	0.5	Deicing	Flyash>20%	1.00	0.20	HNBA6001	0.380

6. Conclusions

It is important to dissect the entire structure into different elements. The non-replaceable ones must be designed rigorously to achieve the defined service life. The integration of testing, modeling, and engineering interpretation supports informed decision-making for new construction. Analyzing several different material combinations can minimize the overall construction cost while still achieving the defined service life. The durability or corrosion protection report lays out in detail the contractor's approach on achieving the defined service life.

The report also provides the owners with an opportunity to review and comment prior to construction of precast or CIP elements. The methodology provides a defensible framework for extending service life, managing risk, and optimizing long-term infrastructure investments.

SCS effort in developing the corrosion protection plan help achieve the required service life by using locally available materials. The service life report listed all material sources and locked in those sources to ensure timely completion of the project.

7. References

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