SERVICE LIFE IMPROVEMENTS OF HIGH-PERFORMANCE CONCRETE BRIDGE DECKS

REPORT FOR THE

ILLINOIS STATE TOLL HIGHWAY AUTHORITY

CTLGroup Project No. 057177

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1. PROJECT BACKGROUND

This report has been developed by CTLGroup to describe a research program that was carried out to improve service life of high-performance concrete (HPC) bridge decks used on Illinois Tollway (the Tollway) infrastructures projects.

In 2011, the Illinois Tollway adopted a 15-year, $12 billion capital program entitled, *Move Illinois: The Illinois Tollway Driving the Future*. The purpose of this program is to make much needed improvements to the Tollway system. The program is projected to create more than 120,000 permanent jobs and add $21 billion to the local economy. *Move Illinois* is expected to improve mobility, relieve congestion, reduce pollution, and link economies across Northern Illinois. The Tollway is committed to minimization of environmental impact, with initiatives to improve the sustainability of highway infrastructure through the use of recycled materials and the improvement of service life.

As part of this effort, CTLGroup has completed a research study between 2012 and 2013 that ultimately led to the development and implementation of a new performance-based specification[1](HPC specification) for class HPC concrete (D'Ambrosia, Slatter, & Van Dam, 2013). This specification implemented several performance criteria to ensure constructability and long-term durability of concrete bridge decks with specific focus on improving cracking resistance. As reported by the Tollway, bridge decks constructed with HPC designed based on this specification have shown improved field performance with reduced cracking present in the structure after several years of service life.

Although the service life of HPC bridge decks was deliberated as an integral part of long-term durability parameters that were considered during the initial research study, primary focus of the study was reduction concrete cracking potential and ensuring its superior long-term durability performance. A recommendation to adopt the use of stainless steel reinforcement to further improve the service life of structures designed in conformance with the newly developed specification was considered. However, primarily due to the inevitable increase in cost associated with use of stainless

[1] Illinois Tollway: Performance-Related Special Provision For High Performance Concrete Mix Designs For Concrete Superstructure

steel reinforcement, this recommendation was never implemented, although the stainless reinforcement was successfully used on several bridge projects, including the Fox River Bridge.

Even though the measures adopted as a result of our 2012-2013 research program resulted in increased service life of the new generation of concrete bridge decks, CTLGroup has conducted a follow-up study in between 2016 and 2017 to further increase the service life of Tollway infrastructure. Longer service life is valuable for many reasons, including reduction of life cycle cost of the Tollway infrastructure, reduction of the impact on the environment through sustainable construction, and reduction of interruptions to traffic and delays for Tollway users.

To this extent, two target service life levels were set for this research program: (1) service life of 50 years and (2) service life of 75 years. For purposes of this study, service life of a concrete bridge deck is defined as a period of time in which the structure will perform without extraordinary maintenance or repair. In order to achieve the target service life levels, two major principles were utilized to delay the corrosion initiation time in the structure, thereby improving the expected service life:

1. Further optimization of mixture proportions beyond the scope of the HPC specification to increase concrete resistance to chloride penetration while maintaining long-term durability characteristics (such as freeze-thaw or scaling resistance), and simultaneously not compromising the cracking resistance of the bridge deck.
2. Use of corrosion-inhibiting admixture to delay the onset of corrosion in the structure.

Although other approaches, such as use of stainless steel reinforcement, were considered, these two outlined approaches were found to be the most feasible and cost-effective. To evaluate effectiveness of these two strategies, two tasks were performed as part of this study: (1) laboratory program to evaluate performance and service life-related characteristics of various concrete mixtures, and (2) service life modeling based on the data obtained from the laboratory study. The third and final task of this study consisted of developing an upgraded specification to be used for future Tollway projects.

2. LABORATORY WORK

2.1 MIXTURE DEVELOPMENT

Four concrete mixtures, as shown in Table 1, were developed and tested in laboratory as HPC mixture candidates that would be, in conjunction with structural and design requirements of Tollway bridge decks, capable of achieving extended service life of 50 and 75 years.

Table 1 – Laboratory Study, Mixture Proportions

Mixture ID:	1	2	3	4
Material	*lbs / yd³*			
Cement	320	400	440	300
Fly Ash	113	--	153	150
Slag	157	200	--	150
Silica fume	--	16	18	--
Coarse Aggregate, CM-11	1274	1274	1287	1274
Coarse Aggregate, CM-16	332	332	336	332
Saturated Lightweight	242	260	263	285
Fine Aggregate, FM01	1064	922	932	962
Water	190	228	214	210
Total Cementitious Content	591	616	611	600
w/cm (including water in admixtures)	0.33	0.38	0.36	0.36
Paste Content Volume (including air), %	30.9%	33.6%	32.9%	31.7%
Chemical Admixtures	*fl oz / cwt*			
Air Entraining Agent	0.50 - 1.11	0.5-2.75	0.50	0.35
Water Reducer	3.00	3.00	3.00	3.00
Shrinkage Reducing Agent	33.2	33.2	33.2	--
High Range Water Reducer	8.0	8.0	8.0	8.0
Hydration Stabilizer	3.0	3.0	3.0	3.0
Target Properties	**Design Values**			
Target Slump	6 to 8 in.			
Design Air Content	7%			

Mixture 1 was developed as part of the effort during our initial HPC study, and showed very good performance in all durability aspects as well as in terms of chloride penetrability. Based on this mixture, mixtures 2 thru 4 were developed. All of these mixtures were designed as ternary blends with comparable total amount of cementitious materials (591 to 616 lbs/yd³), optimized aggregate gradation, and implementing measures to control early-age cracking potential such as use of saturated lightweight aggregate and/or shrinkage-reducing agent (SRA). Additionally, the internal curing was implemented in order to promote the hydration characteristics of the mixture, thereby reducing porosity of the hardened cementitious system and subsequently reducing the ability of chloride ions to penetrate the cement paste. Various combination cementitious materials (i.e. cement, fly ash, slag and silica fume) were used. Mixture 4 was designed without the shrinkage-reducing admixture.

2.2 RESULTS

2.2.1 Concrete Fresh Properties

Measured concrete fresh properties for all four mixtures are shown in Table 2. All mixtures were designed to have initial slump[2] at the upper limit of the slump allowed by the current Tollway HPC specification. In some instance, this value has been exceeded. Additionally, slump loss was evaluated. For mixtures 1 and 2, measured slump loss was within the limit of the specification. The slump loss of mixture 4 was quantitatively higher than slump loss of other mixtures included in this program.

Table 2 – Laboratory Study, Concrete Fresh Properties

Mixture ID:	**1**	**1**	**1**	**1**	**2**	**3**	**4**
Date Fabricated:	3/14/2017	5/25/2017	5/25/2017	6/8/2017	3/15/2017	3/16/2017	3/16/2017
Slump - initial, in.	7.75	8.25	9.00	10.00	9.50	8.00	7.00
Slump - after 20-30 mins, in.	6.00	--	7.00	9.75	9.00	5.00	3.50
Slump - after 40-50 mins, in.	--	--	5.50	9.00	8.50	--	--
Air Content, %	10.0	8.0	8.5	7.4	12.0	10.0	8.5
SAM Number, -	--	--	0.16	0.10	--	--	--
Temperature, °F	76.0	76.4	76.2	75.5	73.5	74.6	73.7
Fresh Density, lb/ft³	137.0	138.9	137.3	140.4	130.9	135.4	138.2
Initial Set, mins.		480					
Final Set, mins.		595					

Similarly, all mixtures were aimed to have initial fresh air content[3] at the upper limit of the typical design air content range. The measured initial air content ranged from 7.4% to 12.0%. Additionally,

[2] ASTM C143 / C143M-15a, *Standard Test Method for Slump of Hydraulic-Cement Concrete*, ASTM International, West Conshohocken, PA, 2015

[3] ASTM C231/C231M-17a Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method, ASTM International, West Conshohocken, PA, 2017

time of set[4] was determined for mixture 1. Initial set was detected after 480 minutes from water addition whereas the final set was determined to be 595 minutes.

2.2.2 Compressive Strength

Compressive strength[5] was determined at various ages for all tested mixtures, as shown in Table 3. Measured compressive strength at 14 days significantly exceeded the 4,000 psi specification requirement.

Table 3 – Laboratory Study, Compressive Strength

Mixture ID:	**1**	**1**	**1**	**1**	**2**	**3**	**4**
Date Fabricated:	3/14/2017	5/25/2017	5/25/2017	6/8/2017	3/15/2017	3/16/2017	3/16/2017
1 day	1,770	--	2,580	2,360	1,690	2,720	1,780
3 days	4,140	--	--	--	3,900	4,420	3,810
4 days	--	--	6,040	5,760	--	--	--
7 days	5,870	--	7,530	7,120	5,410	5,880	5,590
14 days	--	--	9,460	8,680	6,380	6,800	6,830
28 days	7,910	10,570	--	9,640	6,860	7,000	7,840
56 days	--	--	10,140	10,480	7,590	--	--

2.2.3 Volumetric Stability

Mixture 1 was evaluated for drying shrinkage[6] and cracking potential[7]. Obtained results are shown in Table 4. Measured values indicated that the requirements of the HPC specification were met. The volumetric stability of mixtures 2, 3 and 4 was not determined.

Table 4 – Laboratory Study, Volumetric Stability

Mixture ID:	**1**	**1**	**1**	**1**	**2**	**3**	**4**
Date Fabricated:	3/14/2017	5/25/2017	5/25/2017	6/8/2017	3/15/2017	3/16/2017	3/16/2017
Ring Test, days	--	28+	--	--	--	--	--
Length Change, %	--	--	0.023	0.023	--	--	--

4 ASTM C403 / C403M-16, *Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance*, ASTM International, West Conshohocken, PA, 2016

5 ASTM C39 / C39M-17b, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*, ASTM International, West Conshohocken, PA, 2017

6 ASTM C157 / C157M-17, *Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete*, ASTM International, West Conshohocken, PA, 2017

7 ASTM C1581 / C1581M-16, *Standard Test Method for Determining Age at Cracking and Induced Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage*, ASTM International, West Conshohocken, PA, 2016

2.2.4 Durability

The results of hardened air void analysis[8], freeze-thaw[9] and salt scaling[10] resistance testing are shown in Table 5. Air loss between 1.2% and 4.9% occurred in all considered mixtures. Freeze-thaw resistance was evaluated for mixture 1. Although the parameters of the air void system of the tested mixture did not meet the HPC specification requirements, performance of the mixture in freeze-thaw was good with RDM not dropping below 90% after 300 freeze-thaw cycles. Similarly, the mixture showed good salt scaling resistance.

Table 5 – Laboratory Study, Durability

Mixture ID:	**1**	**1**	**1**	**1**	**2**	**3**	**4**
Date Fabricated:	3/14/2017	5/25/2017	5/25/2017	6/8/2017	3/15/2017	3/16/2017	3/16/2017
Total Air Content, %	8.0	--	3.6	4.7	7.1	7.1	7.3
Spacing Factor, in.	0.005	--	0.011	0.008	0.005	0.005	0.003
Specific Surface, 1/in.	642	--	485	618	588	711	989
Freeze Thaw, RDM	--	--	95	96	--	--	--
Deicer Scaling	--	--	1	1	--	--	--

2.2.5 Concrete Transport Properties

The following tests were carried out to characterize all mixtures with respect to their transport properties. Concrete transport properties investigated include chloride ion penetrability[11], STADIUM® ionic diffusivity, apparent chloride diffusivity[12], concrete surface resistivity[13], and concrete bulk resistivity[14]. Results of testing related to the concrete transport properties are shown in Table 6. In general, all concrete mixtures were highly resistant to chloride ion penetration.

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8 ASTM C457 / C457M-16, *Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete, ASTM International, West Conshohocken*, PA, 2016

9 ASTM C666 / C666M-15, *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing, ASTM International, West Conshohocken*, PA, 2015

10 ASTM C672 / C672M-12, *Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals, ASTM International, West Conshohocken*, PA, 2012

11 ASTM C1202-17 *Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration, ASTM International, West Conshohocken, PA, 2017*

12 ASTM C1556-11a(2016) *Standard Test Method for Determining the Apparent Chloride Diffusion Coefficient of Cementitious Mixtures by Bulk Diffusion, ASTM International, West Conshohocken, PA, 2016*

13 AASHTO T 358 *Standard Test Method for Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration, American Association of State Highway and Transportation Officials, 2015*

14 ASTM C1760-12 *Standard Test Method for Bulk Electrical Conductivity of Hardened Concrete, ASTM International, West Conshohocken, PA, 2012*

Table 6 – Laboratory Study, Concrete Transport Properties

	Test Method	**Mixture ID:**	**1**	**2**	**3**	**4**
		Date Fabricated:	3/14/2017	3/15/2017	3/16/2017	3/16/2017
Rapid Chloride Penetrability, coulombs	AASHTO T 277	28d, accelerated	323	423	479	317
Chloride Ionic Diffusion Coefficient, x10-11 m²/s	STADIUM IDC	28 days	1.9	1.8	2.7	2.1
		56 days	1.6	1.7	1.9	1.5
		90 days	1.6	--	--	--
		104 days	--	1.7	1.8	1.3
Apparent Chloride Diffusion Coefficient, $x10^{-12}$ m²/s	ASTM C1556	35 days exposure	0.78	0.91	1.13	0.52
Surface Resistivity, Ωm	AASHTO T 358	27 days	462	435	406	455
Bulk Resistivity, Ωm	ASTM C1760	28 days	437	380	339	434
Bulk Resistivity , Ωm	RILEM TEM	28 days	389	335	301	386

3. SERVICE LIFE MODELING

Service life predictions of cast-in-place bridge decks following the Illinois Tollway Specifications were the primary focus of this work. The objective of the testing and service life prediction analysis was to estimate the time necessary for reinforcement corrosion to be cause structural deterioration.

According to information provided by the Illinois Tollway, bridge deck concrete elements considered in this report had a concrete cover thickness of 2 inches over a mat of epoxy coated steel reinforcement[15]. Concrete mixtures evaluated in Section 2 were used to predict the service life of the bridge decks utilizing Fick's second law and the formation factor. Mixture 1 and Mixture 2 were evaluated using STADIUM®. A corrosion inhibitor admixture (calcium nitrite-based) at dosage of 2 and 5 gallons per cubic yard of concrete were incorporated into the prediction of the service life of Mixture 2.

3.1 SERVICE LIFE REQUIREMENT

According to the Illinois Tollway specifications, the design life is defined as the time frame until significant structural repairs are required to sustain a continuous life of 50 to 75 years[16]. The precise definition of what constitutes the design life of the structure has not been explicitly stated since it does not provide specific limit states that would constitute the end of service life.

According to ACI 365R-17[17], the service life of a structure can be divided into three different categories:

- Technical service life is the time in service until a defined unacceptable state is reached, such as spalling of concrete, safety level is below acceptable, or failure of elements occurs.
- Functional service life is the time in service until the structure no longer fulfills the functional requirements or becomes obsolete due to change in functional requirements, such as the needs for increased clearance, higher axle and wheel loads, or road widening.

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15 ASTM A775/A775M-17 *Standard Specification for Epoxy-Coated Steel Reinforcing Bars*, ASTM International, West Conshohocken, PA, 2017

16 Illinois Tollway Communications

17 ACI 365.1R-17, *Service-Life Prediction—State-of-the-Art Report*, American Concrete Institute

- Economic service life is the time in service until replacement of the structure (or part of it) is economically more advantageous than keeping it in service.

Technical service life pertains to serviceability, durability and structural characteristics of structures and therefore was the focus of our analysis. Functional and economic service life considerations are not covered in this report.

3.1.1 Limit State for Elements and Components

The causes that would result in structural repair of concrete can be widely varied. In most cases, material related distress and deterioration mechanisms are responsible for the reduction in service life in concrete structures. Construction and maintenance practices, excessive or unexpected loading conditions, and numerous deterioration mechanisms can be responsible for reducing the technical service life of structural elements and components but are not part of this work.

In terms of material distress, commonly accepted limit state conditions include cracking of the concrete cover, spalling of the concrete cover, and collapse. These limit states are specific for reinforcement corrosion deterioration (see Section 3.1.3) and may not be directly applicable for other degradation mechanisms.

3.1.2 Degradation Mechanisms

Degradation processes of concrete can be classified as: physical (caused by natural thermal variations such as freeze-thaw cycles, or artificial ones, such as fire), mechanical (abrasion, erosion, impact, explosion), chemical (attack by acids, sulfates, ammonium and magnesium ions, pure water, or alkali-aggregate reactions), biological (fouling, biogenic attack) and structural (overloading, settlement, cyclic loading). In practice these processes may occur simultaneously, frequently giving rise to a synergistic action. Amongst the mentioned deterioration processes, reinforcement corrosion is predominant in terms of occurrence and economical costs (Koch, Brongers, Thompson, Virmani, & Payer, 2002). The implications of other or combined degradation processes are much more complex and have not been included in this research program.

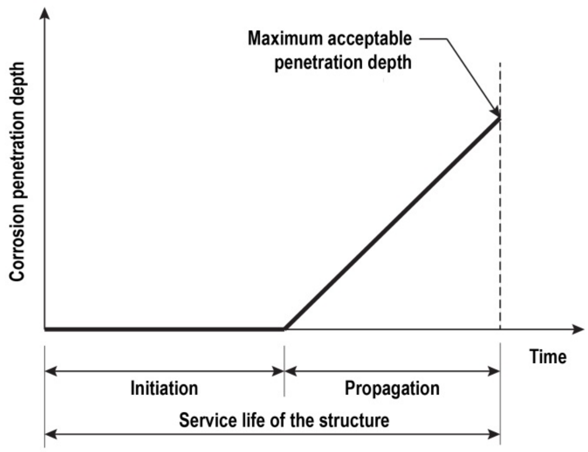
3.1.3 Reinforcement Corrosion

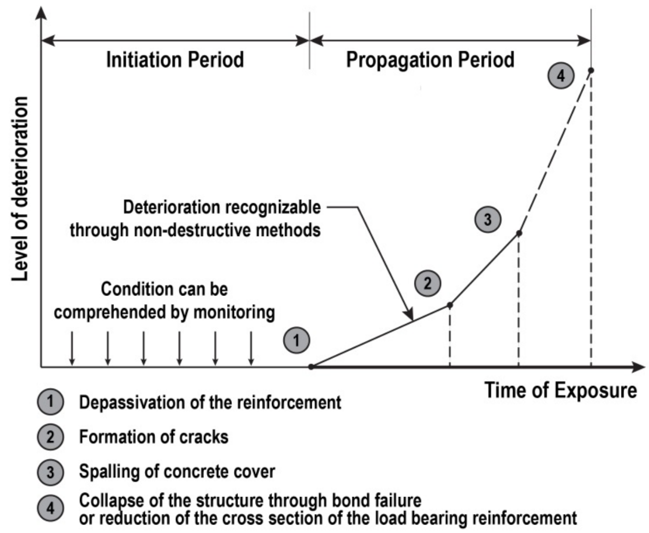
Reinforcement corrosion caused by chloride ion penetration occurs when chlorides in solution penetrate into the concrete and accumulate on the steel reinforcement surface. Once their concentration reaches a critical level, the protective passive layer may be locally destroyed. The critical chloride concentration or chloride threshold has been investigated extensively. The concentration of chlorides necessary to initiate reinforcement corrosion is subject to numerous influencing parameters ranging from concrete placement to the type of embedded reinforcement. The use of corrosion inhibiting admixtures has a direct effect on the critical chloride concentration. The dosage of calcium nitrite-based corrosion inhibiting admixtures (CNI) is also important when estimating the critical chloride concentration, as discussed in Section 3.2.3.

3.2 SERVICE LIFE OF CONCRETE STRUCTURES SUBJECT TO REINFORCEMENT CORROSION

The service life of concrete structures exposed to chloride-laden environments is composed of a corrosion initiation and propagation periods (Bertolini, Elsener, Pedeferri, Redaelli, & Polder, 2014), as shown in Figure 1a. The initiation period (T_i) is a function of the rate of carbon dioxide or chloride ingress, which is dependent on the environmental exposure conditions and the concrete transport properties. This period ends when the concentration of chlorides at the surface of the steel reinforcement exceeds a critical value. This concentration is called the “critical chloride content” or “chloride threshold”. Once the critical chloride concentration has been exceeded, the propagation period (T_p) of reinforcement corrosion is commenced. After the destruction of the passive film, corrosion will occur only if water and oxygen are present on the surface of the reinforcement. The corrosion rate determines the time it takes to reach the minimally acceptable limit state of the structure but it should be borne in mind that this rate can vary considerably depending on temperature and humidity.

Corrosion-induced deterioration occurs when the corrosion process results in the accumulation of corrosion products that result in internal pressures in the concrete that lead to cracking or other limit states, as shown in Figure 1b. The rate of corrosion deterioration is dependent on numerous parameters that prevent it from being included in service life simulations. It is typically assumed that corrosion deterioration signs such as cracking of concrete or spalling of concrete cover could be visible after 5 to 10 years.





a) Tuutti's Model adapted from (Tuutti, 1982)

b) *fib* Model Code 34 Limit States adapted from (fib, 2006)

Figure 1 - Service Life of Structures Subject to Corrosion

3.2.1 Exposure Conditions

The exposure conditions applied in the service life simulations were based on actual conditions reported for Chicago, IL. Temperature and relative humidity historical data necessary for the service life prediction simulations were provided by the Tollway and derived from an analysis of existing bridge decks.[18] The value of average annual temperature of 50°F with amplitude of 34°F, and the average annual relative humidity of 69.4% was utilized. The number of deicing salt exposure days was considered to be the number of days in which the average temperature was lower than 32°F. However, deicing salt exposure can be significantly higher in different geographical locations depending on the maintenance practices of the Tollway. For the service life simulation, it was considered that the concrete bridge deck elements are exposed to a chloride solution with a concentration of 2800 mmol/l of NaCl for 55 days[18]. The chloride concentration was later revised to 1600 mmol/l in 2017[19].

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[18] Tourney Consulting Group: *Combined Sister Structure Report.* April 15, 2016.
[19] Tourney Consulting Group: *Combined Sister Structure Report*. August 9, 2017.

3.2.2 Service Life Models

For our study, we used both the Fick's Second Law and STADIUM® as the select service life predicting models. Modeling of chloride ingress using Fick's Second Law was conducted incorporating the concept of the formation factor (Weiss, Barrett, Qiao, & Todak, 2016; Snyder K. , 2001). Formation factor is derived from Archie's Law (Archie, 1942) and has been used to describe ionic diffusion processes in concrete. Formation factor is defined as the ratio of the electric resistivity of a bulk body (*ρ*) and the resistivity of the pore solution in the body (ρ_o), as shown in Eq. 1:

$$F \equiv \frac{\rho}{\rho_0} \cong \frac{1}{\varphi\beta} \tag{1}$$

where φ is the volume of concrete pores and β is the pore connectivity. In order to describe the transport properties of concrete, the Nernst-Einstein relationship is useful for correlating the formation factor to the diffusion of an ion through a diluted medium:

$$F = \frac{D}{D_0} \tag{2}$$

where *D* is bulk diffusion coefficient is and D_0 is self-diffusion coefficient. The self-diffusion of chloride ions in diluted solutions is approximately 2.0 x10^{-12} m²/s (Dean & Lange, 1998).

The relationship between concrete resistivity, pore solution resistivity and the formation factor is not simple but can be estimated from the relationship between the formation factor and the rapid chloride penetration test. It has been shown that the electrical charge (*Q*) in the AASHTO T 277 test is inversely related to the electrical resistivity of the concrete (Spragg, Yiwen, Snyder, Bentz, & Weiss, 2013), and can be described using Eq. 3:

$$Q = \frac{206{,}830\ Vms}{\rho_0 \cdot F} \tag{3}$$

Where *Q* is the electrical charge in Coulombs, ρ_0 is the pore solution resistivity in Ohms, and *F* is the formation factor. A constant of 206,830 Vms (Volt per meter per second) is used for the calculation of the electrical charge. This indicates that the results of the AASHTO T 277 test are dependent on the microstructure (or the formation factor, which is the inverse product of pore volume and connectivity) and the pore solution resistivity, in other words the chemistry of the binder system.

Estimation of the service life of concrete placements exposed to deicing salts can be derived from the use of the formation factor using Eq. 2. For simplicity, the transport of chlorides in concrete will be described with Fick's Second Law of diffusion as shown in Eq. 4:

$$C(x,t) = C_s * \left[1 - erf\left(\frac{x}{\sqrt{4Dt}}\right)\right] \quad (4)$$

Where C_s is the chloride surface concentration (expressed in percent of concrete weight), x is the concrete cover thickness (m), D is the chloride diffusion coefficient (m^2/s), t is time (s), and *erf* is the Gaussian error function. For simplicity, Eq. 3 assumes that the background chloride concentration in the concrete is negligible.

The rate of chloride ingress is determined by the transport properties of the concrete mixture. Predictions of concrete elements service life consider diffusion as the main transport mechanism. The effect of other transport mechanisms, i.e. capillary suction and convection, are commonly considered to be of secondary (lesser) importance. The diffusion coefficient is a function of the water-to-cementitious ratio (w/cm) and the type and quantity of cement replacement material.

STADIUM® prediction tool is recognized and specified in the Unified Facilities Guide Specifications (UFGS) for the design and construction of new maritime works for the U.S. Navy, U.S. Air Force, U.S. Army Corps of Engineers and NASA[20]. STADIUM® is a finite element model that can describe the transport of multiple ions (chlorides and sulfates) in concrete. Each STADIUM® test consists of measurements of concrete porosity, ionic conductivity, and moisture drying. For service life simulations, the material proportions of each concrete mixture and the chemical composition of cement and supplementary cementitious materials are implemented in the analysis.

3.2.3 Limit State Assumptions

Corrosion initiation occurs when the chloride concentration at the reinforcement depth equals or exceeds a threshold value (chloride threshold).

20 https://www.simcotechnologies.com/what-we-do/stadium-technology-portfolio/stadium-overview/

The concentration of chlorides necessary to initiate reinforcement corrosion is subject to numerous influencing parameters. Typically, the chloride threshold concentration (C_t) for plain (black) steel[21] is in the range of 0.04 to 0.06% by weight of concrete (Angsta, Elsener, K.Larsenac, & Vennesland, 2009); with a typically used value of 0.05% by weight of concrete.

The use of corrosion inhibiting admixtures has a direct effect on the critical chloride concentration. The dosage of calcium nitrite based corrosion inhibiting admixtures is also important when estimating the critical chloride concentration. Table 7 shows the effect of different CNI dosages on the critical chloride concentration for black steel embedded in concrete[22].

Table 7 – Effect of Corrosion Inhibitors in Concrete on Chloride Threshold Values

CNI Dosage (gal/yd^3)	Threshold, C_t (% mass of concrete)	Threshold, C_t (ppm)
0	0.05	500
2	0.15	1500
3	0.24	2400
4	0.32	3200
5	0.37	3700
6	0.40	4000

3.3 TESTING OF CONCRETE SAMPLES IN THE LABORATORY

Testing on samples fabricated in the laboratory was conducted to evaluate the effect of different concrete resistivity test methods and curing conditions. Testing of the surface resistivity per AASHTO T 358[23], bulk conductivity in accordance with ASTM C1760[24] and bulk resistivity in accordance with a RILEM Technical Recommendation[25] were performed on two sets of concrete specimens. The first set was a composed of concrete samples fabricated with each of the concrete mixtures described in Section 2.1.

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21 ASTM A615/A615M-16 *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*, ASTM International, West Conshohocken, PA, 2016

22 http://www.life-365.org/download/Life365_v2.2.1_Users_Manual.pdf

23 AASHTO T 358, *Standard Method of Test for Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration*, 2015

24 ASTM C1760-12 *Standard Test Method for Bulk Electrical Conductivity of Hardened Concrete, ASTM International, West Conshohocken*, PA, 2012

25 Polder, R. B. (2001). *Test methods for on site measurement of resistivity of concrete*—a RILEM TC-154 technical recommendation. Construction and building materials, 15(2), 125-131

Testing of the electrical resistivity was conducted as follows:

- Surface resistivity testing per AASHTO T 358 was performed at an age of 27 days before concrete samples were sectioned from concrete cylinders and conditioned in accordance with AASHTO T 277. Upon removal form the saturated limewater at 100°F, the samples were stored in sealed containers with saturated limewater in standard laboratory conditions at 73°F/50% RH. Testing was conducted after 1 hour in order to mitigate the effect of elevated temperature on the test results. The duration of the test was approximately 5 minutes per sample.
- After conditioning and placing the samples in cells per AASHTO T 277, bulk resistivity measurements were performed in accordance with RILEM TEM at an age of 28 days. Testing was conducted with a handheld LCR meter at a frequency of 120 Hz to avoid electrode polarization. Calculations of the cell constant (sample area divided by sample length) were performed and used to correct the measurements. The duration of the test was one minute per sample. Testing of RILEM TEM was performed before ASTM C1760 was conducted.
- Bulk conductivity per ASTM C1760 was determined on 4 in. nominal diameter and 2 in. nominal thickness samples. The duration of the test was one minute per sample.
- After the three electrical tests were performed, AASHTO T 277 was conducted. The duration of the test was 6 hours.

Table 8 shows a summary of the concrete mixtures subject to AASHTO T 277 testing and the alternative electrical testing methods.

Table 8 – Transport Property Testing of Concrete Samples at Standard Curing Conditions

Mixture ID	AASHTO T 277 – ASTM C1202 (Coulombs)	ASTM C1760 (Ωm)	RILEM TEM (Ωm)	AASHTO T 358 (Ωm)
1	323	437	389	462
2	423	380	335	435
3	479	339	301	406
4	317	434	386	455

The second set of concrete samples was made in order to investigate the effect of curing conditions on the concrete transport properties. All concrete samples were fabricated using the material proportions of Mixture 1. Upon fabrication, concrete samples were cured under two different conditions: sealed and stripped from the molds. The sealed samples were kept in the molds and stored in Ziploc plastic bags to prevent leaching. Subsequently, samples in both sealed and stripped conditions were stored in three different curing conditions: (1) immersed in limewater at 73°F for 28 days, (2) immersed in limewater at 73°F for 7 days and then stored in limewater at 100°F for 21 days as per ASTM C1202 Section 8.2., and (3) immersed in limewater at 120°F for 14 days to increase leaching. Results of the transport properties of these samples are shown in Table 9.

Table 9 - Transport Properties of Concrete Samples Cured at Various Temperature Conditions

Mixture ID	AASHTO T 277 – ASTM C1202 (Coulombs)	ASTM C1760 (Ωm)	RILEM TEM (Ωm)	AASHTO T 358 (Ωm)
1 - Sealed 73°F	633	237	222	437
1 - Stripped 73°F	717	235	224	438
1 - Sealed 100°F	338	441	411	826
1 - Stripped 100°F	389	456	432	850
1 - Sealed 120°F	279	728	550	766
1 - Stripped 120°F	305	704	506	912

Results obtained for chloride ion penetrability and concrete bulk resistivity of Mixture 1 samples cured at 100°F correspond well with results of Mixture 1 samples tested at standard curing conditions. However, surface resistivity results of samples cured in sealed and stripped condition at 100°F were found to be two times higher than those cured at standard conditions. In addition to the effect of curing at elevated temperatures, an increase in surface resistivity values can be attributed to self-desiccation of the sample when cured in sealed conditions, and to leaching of alkali ions when cured in a stripped condition. Both of these factors were likely diminished in samples that were vacuum-saturated during the preparations for AASHTO T 277.

3.4 FORMATION FACTOR AND SERVICE LIFE ESTIMATION

In order to calculate the formation factor, the pore solution resistivity was estimated using the NIST "Estimation of Pore Solution Conductivity" calculator[26]. Alkali contents obtained from the chemical analysis of all cementitious materials except the silica were used as calculator input. For silica fume, default values available in the calculator were utilized. Calculated resistivity of the pore solution is shown in Table 10.

[26] https://www.nist.gov/el/materials-and-structural-systems-division-73100/inorganic-materials-group-73103/estimation-pore

Table 10 – Calculated Pore Solution Conductivity

Mixture ID	Curing Condition	Pore Solution Resistivity, Ωm
1	Saturated	0.071
1	Sealed	0.058
2	Saturated	0.156
3	Saturated	0.068
4	Saturated	0.072

The calculated formation factor, as shown in Table 11, obtained from concrete bulk resistivity measurement results meets the proposed special provisions of the Tollway for HPC Extra concrete placements for all considered mixtures but Mixture 2. For Mixture 2, significantly higher value of the pore solution conductivity (or significantly lower value of the pore solution resistivity) was estimated, primarily due to the chemistry of the mixture constituents as well as a result of higher water content compared to the rest of the mixtures. Since an estimated value of degree of hydration (70%) was used, a lower value of the formation factor was obtained.

Table 11 - Formation factor of concrete mixtures (saturated curing conditions)

Mixture ID	Formation Factor, (-)		
	ASTM C1760	RILEM TEM	AASHTO T 358
1	5956	5464	6482
2	2532	2154	2797
3	5171	4415	5952
4	5813	5384	6338

Table 12 shows the results of the calculated formation factor of concrete samples cured in sealed and stripped conditions while immersed in limewater at different temperatures.

Table 12 - Formation Factor of Concrete Samples Cured at Different Conditions

Mixture ID	Formation Factor, (-)		
	ASTM C1760	RILEM TEM	AASHTO T 358
1 - Sealed 73°F	4066	3807	7506
1 - Stripped 73°F	3297	3149	6151
1 - Sealed 100°F	7569	7047	14176
1 - Stripped 100°F	6404	6060	11927
1 - Sealed 120°F	12489	9442	13142
1 - Stripped 120°F	9872	7100	12799

3.5 SERVICE LIFE PREDICTIONS

3.5.1 Formation Factor Simulations

Based on the formation factor value, the chloride diffusion coefficients can be calculated based on Eq. 2. The self-diffusion coefficient for chlorides in dilute solutions was assumed to be 2.0 $x10^{-12}$ m²/s. Calculated chloride diffusion coefficients are shown in Table 13. Results show that the chloride diffusion coefficient based on bulk electrical resistivity test methods are similar, whereas those calculated form surface resistivity values are significantly lower. As discussed, this is attributed the alkali leaching effect during sample storage.

The estimated service life defined as the time to corrosion initiation plus a fixed corrosion propagation period with duration of 20 years was determined from each of the select test methods. The following input parameters were used:

- Surface chloride concentration of 0.57 % wt. of concrete (as discussed in Section 3.2.1);
- Concrete cover thickness of 0.05 m, corresponding to 2 in. of concrete cover;
- Diffusion coefficient as described in Table 13;
- Chloride threshold for epoxy-coated bars of 0.09 % wt. of concrete (900 ppm)[19];
- Corrosion propagation period of 20 years.

Table 13 – Estimated Chloride Diffusion Coefficient and Predicted Service Life of Concrete Samples Fabricated with Different Concrete Mixtures

Mixture ID	Estimated Chloride Diffusion Coefficient, $x10^{-12}$ m²/s			Predicted Service Life, Years		
	ASTM C1760	RILEM TEM	AASHTO T 358	ASTM C1760	RILEM TEM	AASHTO T 358
1	0.34	0.37	0.31	87	80	95
2	0.79	0.93	0.72	37	31	41
3	0.39	0.45	0.34	76	65	87
4	0.34	0.37	0.32	85	79	93

Results show that the predicted service life of concrete bridge decks constructed with HPC concrete mixtures 1, 3 and 4 evaluated in this research project ranges from 76 to 87 years. This value does not consider the estimated service life from AASHTO T 358 measurements. The use of Mixture 2 showed lower estimated service life compared to the remainder of studied mixtures due to significantly lower formation factor of this mixture.

The calculated diffusion coefficient and estimated service life of concrete cured under different conditions is shown in Table 14. Calculated results clearly show that the effect of the curing conditions on concrete transport properties can lead to significant variations of the estimated chloride diffusion coefficient and the predicted service life.

Table 14 - Estimated Chloride Diffusion Coefficient and Predicted Service Life of Concrete Samples Cured under Different Conditions

Mixture ID	Estimated Chloride Diffusion Coefficient, x10^{-12} m²/s			Predicted Service Life, Years		
	ASTM C1760	RILEM TEM	AASHTO T 358	ASTM C1760	RILEM TEM	AASHTO T 358
1 - Sealed 73°F	0.49	0.53	0.27	59	56	110
1 - Stripped 73°F	0.61	0.64	0.33	48	46	90
1 - Sealed 100°F	0.26	0.28	0.14	111	103	207
1 - Stripped 100°F	0.31	0.33	0.17	94	89	174
1 - Sealed 120°F	0.16	0.21	0.15	183	138	192
1 - Stripped 120°F	0.20	0.28	0.16	144	104	187

3.5.2 STADIUM Simulations

Service life predictions were performed for bridge deck elements using Mixture 1 STADIUM® test results. Simulations were carried out using the current version of the model, i.e. STADIUM® v 2.997. The input for the STADIUM® simulations is shown in Table 15.

Table 15 – STADIUM® Modeling Scenarios

Element Thickness (in.)	Exposure Type	Reinforcement	Chloride threshold (ppm)	Design Cover (in.)	Cover Tolerance (in.)	STADUM® Input Cover (in.)
8	Bridge Deck	ASTM A775	1600	2.25	0.25	2.0 and 2.25
		ASTM A775 + 3 gal/yd^3 CNI	2400			

Service life of the bridge deck concrete elements is typically defined as the sum of corrosion initiation and propagation periods. The determination of the structure service life is dependent on the following components and assumptions:

- Concrete mixture proportioning. Details of modeled Mixture 1 are shown Table 1.
- Chemical composition of portland cement, fly ash and slag cement.
- Epoxy-coated steel reinforcement (ASTM A775) in one mat with a concrete cover of 2 inches. The chloride threshold for the system is defined as 1600 ppm to account for background chlorides. The scenario including 3 gal/yd^3 of a corrosion inhibiting admixture considered a chloride threshold value of 2400 ppm, as shown in Table 7.
- IDC and MTC determined by STADIUM® testing, results of which are attached to this report.
- Aging coefficient of 0.65 was used. This assumption is conservative since it is expected that the resistance of concrete to transport of chlorides improve as the concrete ages.
- A concrete cover to reinforcing steel of 2 inches, and variations in the concrete cover of 0.25 in accordance to ACI 117[27] for elements with a thickness of up to 4 inches. Variations in concrete cover are crucial for service life predictions. Service life predictions presented in this report are only valid for these two cover thicknesses (intermediate values can be interpolated).
- Chloride ingress is strongly dependent on environmental conditions; a simulated chloride solution with a concentration of 1600 mmol/l was used for the modeling.
- The concrete shape that was modeled was a cast-in-place concrete bridge deck which had the top surface exposed to deicing salts.
- Modeling was performed for a period of up to 100 years.
- Default values available for environmental conditions were based on the previously defined exposure conditions.

Table 16 shows the summary of input values and service life estimations of via STADIUM®. The individual STADIUM® reports are provided in the Appendix.

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[27] ACI 17.1R-14 *Guide for Tolerance Compatibility in Concrete Construction*. American Concrete Institute, 2014.

Table 16 – Predicted Service Life according to STADIUM®

Mixture	Ion Migration Coefficient (x10-11 m^2/s)	Saturation at 50% R.H.	Ageing Coefficient, a	Moisture Transport coefficient ($x10^{-22}$ m^2)	ASTM C642 Porosity, %	Initiation period t_i, years	Service life $t_i + t_p$, years
#1	4.80	0.25	0.65	30.53	11.7	~36	~56
#1 + 3 gal/yd^3 CNI	4.80	0.25	0.65	30.53	11.7	~78	~98

Note: A propagation period t_p of 20 years is considered.

Results presented in Table 16 show that in a period of 100 years, the initiation of reinforcement corrosion in the bridge deck elements is predicted to occur at approximately 36 years, as shown in Figure 2. Figure 3 shows the results of the STADIUM® simulation for concrete mixtures including 3 gal/yd^3 of corrosion inhibiting admixture. Our modeling showed that if 3 gal/yd^3 of CNI are used, the initiation period more than doubles.

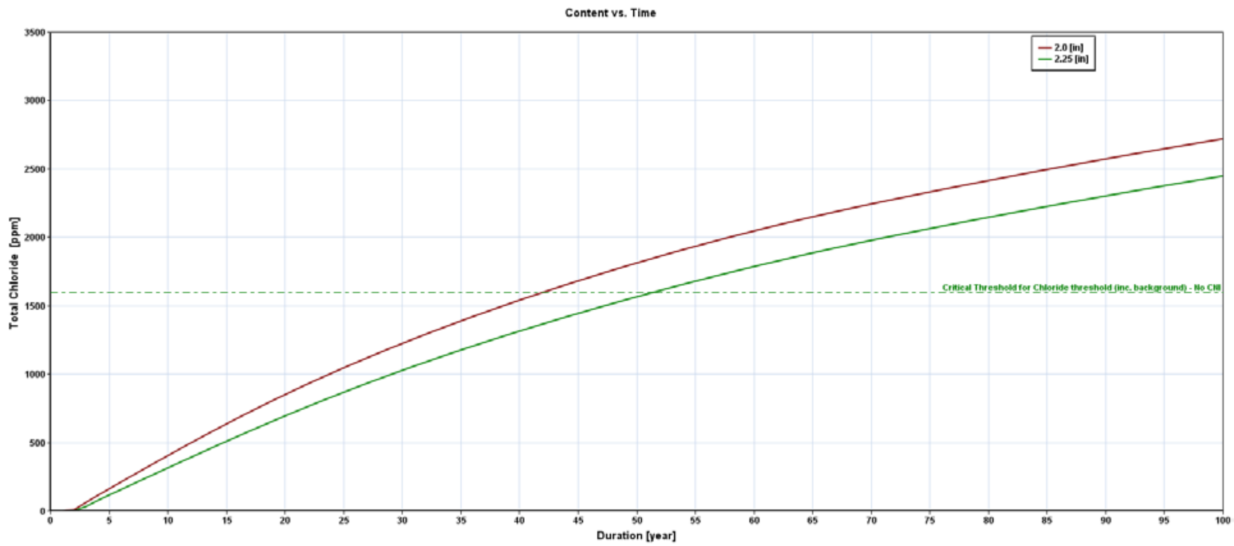

Figure 2 - STADIUM(R) Predicted Service Life: Mixture #1 - No CNI

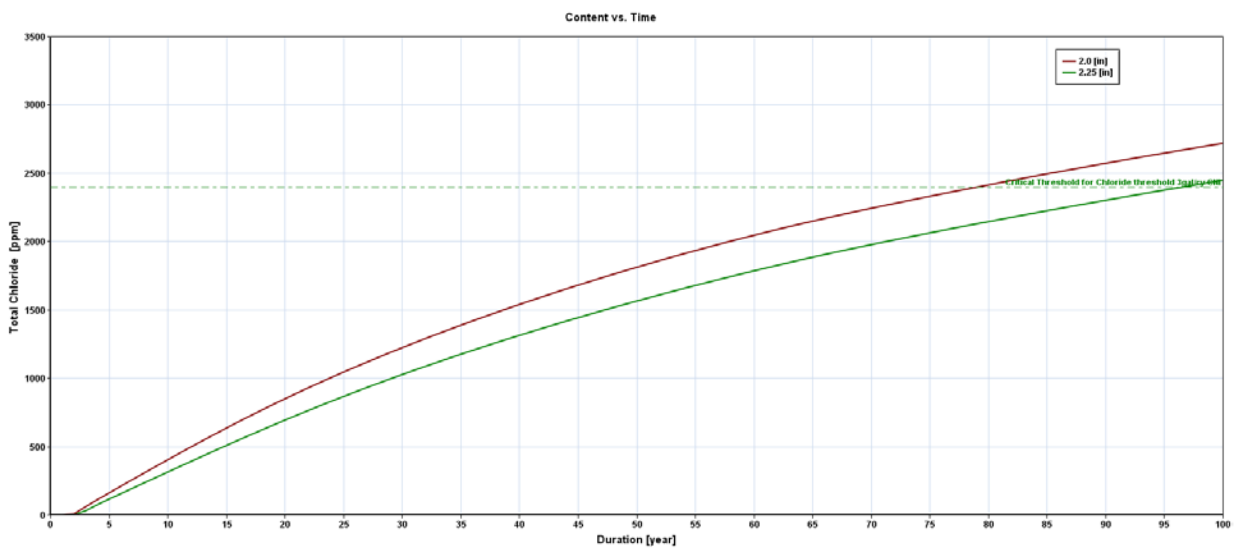

Figure 3 - STADIUM(R) Predicted Service Life: Mixture #1 - 3 gal/yd^3 CNI

3.5.3 Service life modeling limitations

Due to the complex nature of deterioration processes in concrete structures, limitations on the possible analyzed scenarios and transport mechanisms considered to take place are common. The parameters considered in this report are limited to the performance of the tested samples under standard test methods, present application of deicing salts, and efficient maintenance practices over the required service life of the structure.

The service life predictions presented in this report do not account for the effect of the following:

- Cracking of concrete due to fabrication, construction practices, overloading, and/or structural displacements.
- Quality or frequency of maintenance on joint sealers, membrane coatings or barriers.
- Future deicing salt application routines by Illinois Tollway.

Current service life prediction tools assume that the concrete cover over the reinforcing steel is uniform and not cracked. This key assumption presents a significant limitation to the current state of service life modeling because crack-free concrete cannot be guaranteed. Excessive cracking of the concrete can significantly decrease the efficiency of the cover in terms of preventing the accumulation of chlorides at the reinforcing steel. No commonly accepted method exists for evaluating the effects of repaired and/or unrepaired cracks. In most cases, measures are implemented in the design to minimize cracking. Finally, service life modeling tools provide a prediction of the time until corrosion-related repairs may be required.

Due to the dependency of the service life model on the frequency of application and concentration of de-icing salts, it is recommended that the structure is evaluated after five to ten years of service.

4. SUMMARY

A research program was carried out by CTLGroup with the primary goal of improving the current Tollway High-Performance Concrete (HPC) specification and subsequently increasing the service life of newly constructed bridge decks in the Tollway infrastructure network.

First, a laboratory study was conducted. Four HPC concrete mixtures were designed and their performance was evaluated. All considered mixtures were developed to meet the requirements of the current Tollway HPC specification with focus on long-term durability and cracking mitigation, while simultaneously aiming at reducing chloride penetrability. Various blends of cementitious materials were incorporated to explore different options and mixture design strategies. The results showed that the mixtures met the current specification in almost all aspects and at the same time the chloride penetrability was significantly reduced. Therefore, our study confirmed that it is possible to further improve transport properties of HPC mixtures used on Tollway project without compromising their long-term durability performance and cracking resistance.

Second, with the recent interest in using electrical transport test methods to evaluate concrete and their relationship to concrete durability, CTLGroup has performed an evaluation of three different test methods that are commonly used to characterize transport properties of hardened concrete. In addition to the chloride penetration resistance per AASHTO T 277 (RCP test), testing of the surface resistivity per AASHTO T 358, bulk conductivity per ASTM C1760 and bulk resistivity per RILEM TEM have been conducted. A comparison between the selected electrical resistivity methods and the electrical charge determined per AASHTO T 277 has been performed. Results show that the bulk conductivity and bulk resistivity values were corresponded well to each other whilst the surface resistivity measurements resulted in significantly higher values. It was concluded that the leaching of alkali ions during submerged condition in saturated limewater storage and self-desiccation of samples in a sealed condition may be responsible for the increased surface resistivity results.

Third, electrical resistivity test methods were used to estimate the formation factor of concrete. Calculated formation factors obtained from testing presented in this report corresponded well with values presented in technical literature on the matter. It was also found that the estimated formation factor obtained from surface resistivity measurements did not correspond well with obtained results from AASHTO T 277 testing. This confirms that leaching should be avoided when using surface resistivity as a tool for estimating the concrete resistance to chloride penetration.

Lastly, service life modeling was performed using STADIUM® simulations, various test scenarios, and assumptions pertinent to the Chicago, IL area and current Tollway design requirements. It was shown that mixtures considered in the laboratory part of this study can achieve service life of more than 50 years. If corrosion inhibitor is used, a significant increase in service life exceeding 75 years can be expected.

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