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# **Corrosion Durability of Reinforced Concrete Utilizing UHPC for ABC Applications**

**Final Report** 

March 2019

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Sponsored by Accelerated Bridge Construction University Transportation Center



UNIVERSITY TRANSPORTATION CENTER

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#### **CHAPTER 1: INTRODUCTION**

#### 1.1. Project Motivation

In the United States, ~30% of the ~600,000 highway bridges are labeled structurally deficient or functionally obsolete [1]. These bridges should be replaced or upgraded to sustain the transportation needs of the growing public and private sectors of the US economy. However, the required investment in the US transportation infrastructure was estimated to include an annual cost of \$9.4 billion over the next 20 years [1]. Approximately 65% of the bridges in the United State are reinforced concrete, of which 40% are classified as structurally deficient or functionally obsolete [2]. Twenty percent of these substandard concrete bridges are damaged due to chloride induced corrosion of the reinforcing steel [2]. Penetration of chlorides from marine exposures or application of deicing salts causes significant deterioration, requiring periodic maintenance and repair [3-4].

Corrosion of reinforcing steel is principally due to steel depassivation by chloride ions (or carbonation of the concrete pore water by carbon dioxide at higher and dryer elevations). In the absence of such aggressive chemical compounds, a passive film of iron oxides on the steel surface resulting from the alkaline concrete pore water protects steel from corrosion[5-6]. Reinforced concrete in marine environments, especially in marine tidal regions, is susceptible to chloride ingress to the cover depth of the steel reinforcing bars and then subsequent corrosion initiation. Electrochemical reactions continue after corrosion initiation, leading to the accumulation of expansive iron corrosion products that generate tensile stresses in the surrounding concrete of the corroding steel reinforcement. This results in concrete cracking and spalling, which exacerbates the progressive damage, thus affecting the durability of the structure. Other environmental and material factors can affect the rate of steel corrosion including among others, the concrete resistivity, the internal moisture content, concrete porosity, as well as the availability of oxygen [7-10].

Structural members damaged by corrosion are typically repaired by removal of delaminated concrete and application of repair patches or jackets. Concrete piles retrofitted with repair jackets, which ideally would limit the chloride and oxygen ingress to the embedded steel, have been documented to have continued steel corrosion. Application of cathodic protection/prevention systems for marine reinforced concrete structures have been utilized with success and protected concrete for decades [11-12]. But traditional repair methods have limitations due to the presence of incipient anodes after removal of the damaged concrete and replacement with chloride-free repair material. Due to the presence of vestigial chloride ions in the existing concrete surrounding the repair location, incipient anodes may develop there causing the halo effect thus allowing for anew corrosion damage adjacent to the repair area[13-15]. In this phenomenon, the steel within the newly repaired area as well as steel extended in the substrate concrete can serve as a cathode generating accelerated corrosion of the steel in the existing concrete surrounding the patch repair. This macrocell corrosion can be aggravated in chloride-induced corrosion of rebar embedded in concrete with large cathode areas. Investigations on concrete with different water-to-cement (w/c) ratio (0.40-0.70), cover depth and cathode-to-anode ratio showed that the rate of corrosion on the anode mainly depended on chloride content and oxygen supply[16].

Due to the escalating scale of corrosion induced damage, repair methods that have been implemented are either cost or labor-intensive. It is not uncommon for structures to have advanced levels of corrosion-induced damage where major repair and maintenance are required. However, the transportation infrastructure cannot be out of service without disruption to critical economic public, civil and commercial activities. This mandates the development of new techniques and materials for accelerated rehabilitation and recovery. The development and application of effective repair methods and durable materials aiming at extending the lifetime of both existing and new structures is a critical issue. The repair material should have proper properties required to form an adequate physical and chemical barrier against the diffusion of deleterious substances, such as chlorides and carbon dioxide. ACI Committee Report 54610 provides guidance on the selection, application, and methods for repairing concrete structures. A durable repair of corrosion damage requires all chloride-contaminated concrete be removed and replaced by low-shrinkage, low modulus, high-creep, high-tensile-strength patch material with the same coefficient of thermal expansion to minimize interface cracking[17]. The enhanced mechanical and low permeability characteristics of ultra-high-performance concrete (UHPC) have been widely reported and make UHPC a material of interest for structural applications including rehabilitation of concrete structures [18-20]. While UHPC does not have lowshrinkage, low modulus, or high creep, its high tensile strength and low permeability make it an option as a repair material. Application of UHPC can potentially result in both technical and economic advantages, including decreased reinforcement corrosion activity and an increase in service life. Because it is becoming more common to apply UHPC for rehabilitation to improve strength and ductility, improved protection against corrosion would serve as a complimentary benefit of this treatment.

#### 1.2. Research, Objectives, and Tasks

Experimental research on the corrosion durability of a proposed Accelerated Bridge Construction (ABC) solution to repair reinforced concrete elements in marine environments utilizing UHPC is presented in this report. It had been suggested that repair of concrete members with a UHPC shell will decrease or slow reinforcement corrosion by confining the concrete and providing a barrier layer with reduced permeability; however, it has been observed that corrosion cells may redevelop in steel encapsulated in the repair materials. The work here investigated corrosion durability properties of UHPC and its possible use to mitigate macrocell corrosion caused by the presence of incipient anodes in concrete repairs with dissimilar concrete materials. The objectives of the preliminary research were to:

1) identify the possible beneficial properties of UHPC related to corrosion;

2) investigate the extent of local macrocell coupling between incipient anodes and local cathodes in the UHPC repair material; and

3) identify the effect of the incipient chloride content.

### 1.3. Research Advisory Panel (RAP)

The project work has been presented in several forums including national and international organizations in the area of accelerated bridge construction and work has been disseminated in peer-reviewed conference proceedings

## 1.4. Report Overview

Chapter 1 provides introductory project information including research objectives, Chapter 2 provides research methodology, and Chapter 3 provides results and discussion on material and corrosion aspects. Chapter 4 discusses the effect of repair cold joints. Chapter 5 presents discussion on service life calculations. Chapter 6 presents research conclusions.

#### CHAPTER 2. METHODOLOGY

#### 2.1. Materials

The experiments included casting two concrete types (conventional concrete (NSC) and ultra-high-performance concrete (UHPC)). The concrete mix proportions are listed in Table 1. The conventional concrete comprised of Type II Portland cement, crushed limestone coarse aggregate (maximum size 20 mm), and sand fine aggregate. One batch denoted as Regular Mix also included fly ash and had a water-to-cement (w/c) ratio of 0.43. Additional batches denoted as Chloride Mix incorporating sodium chloride (0, 0.4, 4, 8% NaCl by weight of cement) were mixed with a higher w/c ratio (w/c $\sim$ 0.58) to further differentiate the material properties from the regular mix. The slump of the regular mix concrete was 127 mm, and the slump of the chloride mix concrete had slump in the range of 130-140 mm. The average 28-day compressive strength of the regular mix concrete was 49 MPa and ranged from 21-28 MPa for the chloride mixes. The UHPC used in this study was an available commercial product and was composed of a blended premix powder, water, superplasticizer, and 2% steel fibers by volume. The premix powder included cement, silica fume, ground quartz, and sand. The fibers were 13 mm long, with a tensile strength of 2800 MPa. Flow table test was performed according to ASTM C143720, to obtain the rheology of the UHPC. Static and dynamic flowability of UHPC was measured 200mm, and 250mm, respectively.

Constituents	Normal Strengtl	Ultra-High Performance Concrete (UHPC)	
	Regular Mix (NSC Repair)	Chloride Mix (NSC Substrate)	Premix+Admix (UHPC Repair)
Portland Cement (kg/m <sup>3</sup> )	297 <sup>1</sup>	312 <sup>1</sup>	712
Fine Aggregates (kg/m <sup>3</sup> )	757	808	1,020
Coarse Aggregates (kg/m <sup>3</sup> )	979	959	-
Ground Quartz (kg/m <sup>3</sup> )	-	-	211
Fly Ash $(kg/m^3)$	74	-	-
Silica Fume (kg/m <sup>3</sup> )	-	-	231
Air-Entraining Agent (mL)	325	-	-
Accelerator <sup>2</sup> (kg/m <sup>3</sup> )	-	-	30
Superplasticizer <sup>2</sup> (kg/m <sup>3</sup> )	-	-	30.7
Water (kg/m <sup>3</sup> )	127.5	2267	109
Sodium Chloride (g)	-	0, 47, 466, 933	-

Table 1. Concrete Mix (Cylinder and Slab Specimens)

1. Portland Cement Type II. 2. Proprietary materals

#### 2.2. Test Setup

#### 2.2.1. Concrete Material Parameters

In small scale testing, the various concrete mixes were cast in standard 76.2 mm or 102 mm diameter cylinder molds and cured in various environments including immersion in lime water or placed in 75% or 100% relative humidity (RH) chambers. Sample geometry and dimensions are summarized in **Table 2**. The small samples were de-molded 7 days after casting and placed in the relevant curing and exposure environments. All small-scale samples were kept in the laboratory where ambient temperature was typically 25°C.

The small NSC and UHPC samples were used to measure and compare the concrete resistivity, oxygen diffusivity and internal moisture of the two concrete types for each of the three exposure environments. For those experiments, the samples were instrumented and prepared to facilitate different test methodologies. The mass change was regularly recorded only for those samples where the bulk concrete was not instrumented or modified for testing (bulk resistivity samples). All testing was conducted on duplicate samples.

Table 2. Cast Concrete Cylinder Specimen Geometry
---

	Cylinder dimensions (diameter x height) (mm)
Oxygen Diffusivity	76x76
Bulk Resistivity	76x152
Internal Relative Humidity	102x204

Four-point resistance measurements utilizing a soil resistance meter were made to calculate the concrete bulk resistivity. The inner reference electrodes were activated titanium mesh and the outer counter electrodes were parallel stainless-steel plates. All concrete samples were surfaced dried with a towel prior to testing. All electrodes were separated by moist sponges in a test array that was mechanically confined with a clamp. Excess free moisture was avoided to prevent possible preferential charge through the outer surface of the concrete. The concrete bulk resistivity was calculated based on Equation 1,

Eq.(1)

where  $\rho$  is the resistivity of the concrete ( $\Omega$ .m), R is the resistance ( $\Omega$ ), A is the cross-section area of the samples (4600 mm<sup>2</sup>), and L is the length of the sample (0.152 m).

Samples used to measure the oxygen diffusivity of the concrete samples had a stainlesssteel disk (diameter of 50 mm) that was coated with an epoxy on the back face and activated titanium rod and mesh embedded inside the concrete to conduct cathodic potentiodynamic polarization scans. The polarization scans were made from the open-circuit potential (OCP) condition to  $-1.1V_{SCE}$  at a scan rate of 0.025mV/s. The limiting current density was calculated by least-squares fitting using Butler-Volmer equation including concentration polarization. The stainless-steel disk was used as the working electrode, the activated titanium rod was used as the reference electrode, and the activated titanium mesh was used as the counter electrode. Although the efficiency of oxygen reduction reactions on stainless steel is not the same as for plain carbon steel, the experiments aimed to differentiate oxygen transport parameters in the tested concrete types, mixes, and exposure conditions. For these experiments, all concrete surfaces except the top surface were coated with an epoxy. As a first approximation, the oxygen diffusivity  $D_{O2}$  was calculated following Equation 2,

$$i_{\rm L} = \frac{D_{\rm O2} \, n \, F \, C_{\rm B}}{\delta} \tag{Eq (2)}$$

where  $i_L$  is the measured limiting current density,  $D_{O2}$  is the oxygen diffusivity, n is the valence (n=4), F is Faraday's constant (F=96,500 coul/mol),  $C_B$  is the assumed oxygen bulk concentration at the concrete surface (assumed to be  $2.5 \times 10^{-7}$  mol/cm3), and  $\delta$  is the diffusion length assumed to be the length of the sample ( $\delta$ =7.6cm). Samples used to measure the internal relative humidity were prepared after 56 days of exposure (63 days after casting) following ASTM F217021. A 22-mm diameter, 102 mm deep hole was drilled at the top surface where a plastic sleeve was inserted and sealed to expose only the bottom surface of the cavity. For the testing initiated after 67 days after exposure (74 days after casting), a hygrometer probe was sealed inside the cavity to monitor the temperature and IRH during 3 day intervals for up to ~160 days of exposure (~167 days after casting).

#### 2.2.2. Corrosion Macrocell Testing

Sixteen reinforced concrete prisms were cast and divided into two groups according to the type of repair concrete material (NSC or UHPC). All testing was conducted on duplicate samples. Each prism contained a concrete portion made from the Chloride Mix to represent the substrate concrete and a concrete portion made from either the Regular Mix or UHPC to represent repair concrete. The interior side of the initial cast concrete section surface was roughened with a mechanical grinder; and prior to casting of the final concrete section, this interface surface was kept moist to facilitate bond at the cold joint. The details of the test specimen are shown in **Figure 1**.

Each concrete prism had geometry 610x305x76 mm and contained eight equidistant 9.5 mm-diameter deformed steel rebar with 25 mm clear cover from the top, bottom and side surfaces. Three of these bars were placed in the substrate concrete and five bars were placed in the repair concrete. To eliminate steel corrosion on the bar sections extending out of the concrete, the outer 508 mm sections of the rebar were coated with epoxy. Activated titanium reference electrodes were placed in between each bar and two discrete activated titanium mesh were embedded on the near surface of the prism. Each bar was electrically coupled via electrical switches to allow macrocell current measurements from steel electrodes in the substrate concrete and repair concrete with a current meter. The electrical switches also allowed for changes in sample configuration cathode and anode size. Cathode-to-anode ratio (5:3, 5:2, 5:2, 5:1, 4:3, 4:2, 4:1, 3:3, 3:2, 3:1, 2:3, 2:2, 2:1, 1:3, 1:2, 1:3)) were varied by systematically decoupling rebar electrodes. Cathode-to-anode ratio 1 and 2 had multiple configurations.



Figure 1. Test Slab Geometry

All concrete samples were kept in the laboratory where the ambient temperature was typically 25°C. The first phase of measurements was made for ~250 days after casting and the second phase of measurements was made for an additional ~250 days. In the second phase, a pond was made out of acrylic molding attached along the periphery of the top surface of the concrete test prisms. Tap water was periodically added so that the top surface of the sample remained wet.

#### 2.2.3. OCP, LPR, and Electrical Resistance

The open-circuit potential (OCP) of the individual bars was periodically measured by decoupling the rebar via the external electrical switch. A copper/copper-sulfate reference electrode (CSE) was centered on the concrete surface immediately above the rebar. The concrete test samples contained 5 rebars in the repair concrete and 3 rebars in the substrate concrete. OCP was made for individual bars. The OCP was measured after 2-3 hours decoupling. Although the measured potentials may not necessary reflect terminal rest potentials, the measured values did differentiate relative passive and active conditions.

Linear polarization resistance (LPR) measurements were periodically made after approximately 24-hour depolarization of the coupled electrodes. The corrosion current density was calculated from Equation (3):

$$I_{corr} = B/Rp$$
 Eq (3)

where B was assumed to be 26mV, Rp was the measured polarization resistance and A is the surface area of rebar. Although  $i_{corr}$  was calculated for individual bars, the replicate readings are shown as the average of group measurements within the repair or substrate concrete sections.

The resistance between every two working electrodes was measured using two-point resistance measurements prior to and after ponding the samples to see the effect of concrete wetness on the resistivity. The measurements were made for five configurations as shown in **Figure 2** including resistance measurements in the substrate concrete, repair concrete, and the interface.



Figure 2. Configuration for Resistance Measurements

#### 2.2.4. Cold Joint

#### 2.2.4.1. Test Setup

Normal strength concrete and UHPC mix designs used in this phase were as shown in **Table 3**. Twelve test specimen were cast in 75x120 mm plastic molds. Eight of those samples were made by partial casting of NSC and UHPC creating a lift of the dissimilar concretes in the geometry of two half cylinders. The base NSC portion was made by cutting an initial full cylinder concrete casting into a half cylinder. After conditioning (in either 0%, 75%, 100% RH or soaked), the smooth cut surface was roughened by cutting parallel grooves (3 mm deep with 1 cm spacing) where upon the specimen was reset in a cylinder mold and the void space was cast with UHPC. The remaining specimens consisted of full NSC or UHPC. The specimens were demolded after seven days of curing within the plastic mold and placed in the various conditioning environments for 190 days.

Constituents	Normal Strength Concrete (NSC)	Ultra-High Performance Concrete (UHPC)
	Regular Mix (NSC Repair)	Premix+Admix (UHPC Repair)
Portland Cement (kg/m <sup>3</sup> )	520	712
Fine Aggregates (kg/m <sup>3</sup> )	1,007	1,020
Coarse Aggregates (kg/m <sup>3</sup> )	545	-
Ground Quartz (kg/m <sup>3</sup> )	-	211
Silica Fume (kg/m <sup>3</sup> )	-	231
Air-Entraining Agent (mL)	6	-
Accelerator <sup>2</sup> (kg/m <sup>3</sup> )	-	30
Superplasticizer <sup>2</sup> (kg/m <sup>3</sup> )	-	30.7
Water $(kg/m^3)$	198	109

Table 3. Concrete Mix (Cold Joint)

A modified Rapid Migration Test was used to assess chloride penetration through the cold joint. The test setup is shown in **Figure 3**. The flat portions of the cylindrical specimen were uniformly ground to create smooth surfaces. An acrylic tube used as a ponding dam was attached to the top surface. An epoxy sealer was used around the base of the ponding dam and extended on the outer surface of the cylinder to minimize salt solution leakage and weeping on the surface of the specimen. The pond was filled with a 10% NaCl solution. The bottom portion of the specimen was set into a shallow solution of lime water. Titanium mesh electrodes were placed at each end and 25 Volts was applied by a DC power supply for 21 days to allow electrical migration of chloride ions to flow through the concrete.



After that time, the specimens were subjected to a splitting tensile strength test. The exposed surfaces were sprayed with 0.1N silver nitrate solution to visually identify chloride ion presence.

#### CHAPTER 3. RESULTS AND DISCUSSION

#### 3.1. Bulk Concrete Material Characteristics

The small-scale test samples were made to identify UHPC material properties relevant to corrosion durability particularly in various environmental moisture exposure conditions such as immersion conditions, 100% RH, and 75% RH. The research was not intended to have direct comparisons to current commercially available repair materials but rather give indication on the performance of UHPC used for repairs in poor quality concretes as may be present in older structures where corrosion may be prevalent.

**Figure 4** shows the results of concrete mass change for NSC and UHPC exposed in immersed conditions, 100% RH, and 75% RH. As expected, there is a small mass loss during hydration of the conventional concrete in ambient 75% RH conditions and some mass increase during hydration when exposed to high moisture conditions such as in immersion or 100% RH conditions. In the high moisture conditions, excess available moisture is retained in the developed concrete macropores. Only minor to no increase in mass was observed for UHPC in all of the tested moisture exposure conditions. This may be due self-dessication of concrete due to the high cement content and low concrete porosity



Figure 4. Mass Change for NSC and UHPC in Moisture Exposure Environments.

**Figure 5** shows the calculated bulk resistivity of the concretes in the moisture exposure environments. For conventional concrete and UHPC, the increase in bulk resistivity regardless of expected internal moisture presence (due to the various moisture exposure environments) is indicative of early cement hydration. In the conventional concrete, the lower bulk resistivity developed in the moist exposure conditions is due to filling of pore spaces with excess moisture (as supported by the increase in mass with time in those samples). UHPC showed bulk resistivity up to an order of magnitude larger than the tested conventional concrete, consistent with its higher cement factor and relatively low internal moisture content due to its low permeability.



Figure 5. Bulk Concrete Resistivity for NSC and UHPC in Moisture Exposure Environments.

**Figure 6** shows the internal relative humidity (IRH) for conventional concrete and UHPC. As expected, the IRH was higher for the moist exposure conditions than at 75% RH. Also, UHPC generally showed lower internal relative humidity consistent with low internal moisture content that was also described by the low mass gain and high bulk resistivity for samples exposed in both 75% RH and higher moisture conditions. Therefore, the results verify high quality, low permeability characteristics of UHPC in both ambient and high moisture exposure environments.



Figure 6. Internal Relative Humidity and Temperature of NSC and UHPC in Moisture Exposure Environments.

Characterization of oxygen transport through UHPC is important to identify if the highquality concrete may mitigate corrosion by slowing the rate of oxygen reduction. The limiting current was calculated by least square fitting of the cathodic polarization scans. An example of that procedure is shown in **Figure 7** for UHPC and conventional concrete conditioned in 100%RH. The findings for the small-scale testing indicate low permeability for UHPC in moist exposure conditions. Per earlier discussion, the UHPC would then be expected to have low porosity and low internal moisture content where one could pose that the reduced moisture presence may enhance gas transport. However, the larger cement factor in UHPC would provide a denser material. Indeed, the calculated approximate oxygen diffusivity (**Figure 8**) for UHPC was much lower than the conventional concrete, and lower diffusivity was observed in UHPC exposed in immersed conditions than in ambient humidity conditions. Therefore, development of corrosion cells is expected to be mitigated in part due to low gas permeability.



Figure 7. Example of Cathodic Polarization Curve Fitting Results.



Figure 8. Apparent Oxygen Diffusivity for NSC and UHPC in Moisture Exposure Environments.

### 3.2. OCP, LPR, and Concrete Resistivity

Prior to coupling of the rebar electrodes embedded in the representative repair and substrate concrete mixes, the open-circuit potential of the individual bars was measured by centering a copper/copper-sulfate reference electrode on the concrete surface immediately above the rebar. As shown in **Table 4**, noble potentials indicative of passive conditions was measured for steel embedded in the repair concrete as well as for steel in the substrate concrete with chloride contents 0 and 0.4wt%. The steel embedded in concrete with higher chloride concentrations had potentials indicative of active corrosion conditions.

The repair NSC and UHPC were chloride free, and the OCP generally reflected passive corrosion conditions before and after wetting although some measurements were more electronegative. For rebar embedded in the substrate NSC concrete admixed with low (0.4%) or no chloride contamination, the OCP also reflected passive conditions prior to wetting. However, after wetting, OCP values for the 0 and 0.4% chloride case were between -263 and -538 mVCSE. This may be related to possible concrete pore water carbonation of the pore quality NSC that may have occurred prior to wetting. As expected, with chloride contamination (4% and 8%), active potential was observed before and after wetting.

Tuble 1. Open chedit i Stellin Stub Concrete Specifich (m (CSE)							
Steel in Repair Concrete Steel in NSC Substrate							
Repair Material	Initial	Dry	Wet	Cl- Content	Initial	Dry	Wet
NSC	-132,	-115,	-97,	00/	-118,	-108,	-431,
NSC	-132	-108	-120	0%	-131	-105	-263
NCO	-113,	-106,	-113,	0.40/	-118,	-142,	-472,
NSC	-128	-116	-145	0.4%	-184	-135	-430
NSC	NGC 40/ NGC -131, -116,	-116,	-142,	40/	-466,	-475,	-576,
	-126	-114	-135	4%	- 491	-396	-632
NSC	-127,	-129,	-246,	<b>Q</b> 0/	-508,	-690,	-690,
NSC	-148	-115	-173	0%	-505	-509	-670
UHPC	-109,	-101,	-217,	00/	-116,	-103,	-439,
	-75	-76	-217	0%	-136	-113	-538
LINDC	-160,	-111,	-330,	0.404	-172,	-144,	-351,
UHPC	-150	-82	-240	0.4%	-170	-148	-475
UHPC	-335,	-40,	-221,	40/	-466,	-458,	-717,
	-100	-138	-380	4%	-487	-480	-715
UHPC	-244,	-122,	-221,	00/	-493,	-489,	-684,
	-181	-49	-252	8%	-530	-513	-706
	Steel Repair Material NSC NSC NSC NSC UHPC UHPC UHPC UHPC	Steel in Repair         Repair       Initial         Material       Initial         NSC       -132, -132         NSC       -113, -128         NSC       -131, -126         NSC       -127, -148         UHPC       -109, -75         UHPC       -160, -150         UHPC       -335, -100         UHPC       -244, -181	$\begin{array}{c c c c c c c } \hline Steel & in Repair Concret Repair Material Initial Dry Initial Dry Initial Dry Initial Dry Initial NSC -132, -115, -132, -108NSC -132, -108NSC -128, -106, -128, -116NSC -128, -116NSC -126, -114NSC -126, -114NSC -127, -129, -148, -115UHPC -109, -101, -75, -76UHPC -160, -111, -150, -82UHPC -335, -40, -100, -138UHPC -244, -122, -181, -49$	$\begin{array}{c c c c c c c } Steel & In Repair Concrete \\ \hline Repair Material & Initial & Dry & Wet \\ \hline Material & Initial & Dry & Wet \\ \hline MSC & -132 & -115 & -97, \\ -132 & -108 & -120 \\ \hline MSC & -113 & -106 & -113, \\ -128 & -116 & -145 \\ \hline MSC & -128 & -116 & -142, \\ -126 & -114 & -135 \\ \hline MSC & -127 & -129 & -246, \\ -148 & -115 & -173 \\ \hline MPC & -109 & -101 & -217, \\ -75 & -76 & -217 \\ \hline UHPC & -160 & -111 & -330, \\ -150 & -82 & -240 \\ \hline UHPC & -335 & -40 & -221, \\ -100 & -138 & -380 \\ \hline UHPC & -244 & -122 & -221, \\ -181 & -49 & -252 \\ \end{array}$	$\begin{array}{c c c c c c c c c c c c c c c c c c c $	$\begin{array}{c c c c c c c c c c c c c c c c c c c $	$\begin{array}{c c c c c c c c c c c c c c c c c c c $

Table 4. Open Circuit Potential of Steel in Slab Concrete Specimen (mV<sub>CSE</sub>)

Consistent with OCP measurements, low corrosion currents were measured in the chloride-free repair UHPC and NSC concrete before and after wetting (see **Figure 9**). As expected, high corrosion current densities were measured for the substrate NSC concrete contaminated with 4 and 8% chlorides; these corrosion current densities showed further increase after concrete wetting due to the expected lower solution resistance of wet concrete. The unexpected activation after wetting of the substrate NSC concrete with 0 and 0.4% chloride in all test samples showed an increase in corrosion current density relative to corrosion current densities measured for those electrodes when passive in the dry condition.



Figure 9. Corrosion Current Density

The results of measured two-point resistance of the samples is shown in **Figure 10.** The resistance of the samples regardless of the location on the concrete samples decreased after wetting; however, this reduction was significantly less in UHPC consistent with the low permeability of the material. The changes in the concrete bulk resistance was consistent to the trends observed in the macrocell measurements and further illustrate the positive use of UHPC.



Figure 10. Concrete Resistance

Comparison of the potentials for steel in the substrate and repair concrete from **Table 4**, for dry and wet concretes, show that there is generally low driving voltage when chloride level are low (0 and 0.4% CL-) for either repair application of NSC and UHPC. Therefore low macrocells were expected regardless of the resistance of the concrete materials. In presence of high chloride content (4 and 8% CL-), the driving voltage, regardless of repair concrete material was high (as much as ~500 mV). High macrocell currents can develop for NSC due to this driving voltage and the relatively low concrete resistance (**Figure 11**). On the other hand, despite similar driving voltage, low macrocell current can develop with UHPC repair due to its high concrete resistance.

#### 3.3. Corrosion Macrocell

After coupling all rebar electrodes together, the macrocell current between the electrodes in the repair and substrate concrete was measured. Low macrocell currents were measured for NSC in the dry condition where passive corrosion conditions were present for rebar in both substrate and repair concrete (0 and 0.4% wt Cl in substrate concrete). After wetting, the OCP values showed unexpected activation of the rebar in the substrate concrete for these test samples. The developed macrocell current, correspondingly, showed an increase. When rebar in the substrate concrete in the presence of higher chloride content (4 and 8% wt Cl-) had active corrosion, large macrocells developed. These values showed a significant increase after wetting (see Figure 11). As will be described later, macrocell current can be enhanced with larger cathode-to-anode ratios. In contrast, low macrocell current developed between the rebar electrodes embedded in the UHPC repair and substrate concrete regardless of the level of anodic activity in any of the chloride content in the substrate concrete. Wetting of the concrete did indicate overall increase but current levels still remained significantly smaller than comparative samples cast with repair NSC. The results indicate that regardless of corrosion activity, there is a mitigating effect by the placement of UHPC. Galvanic coupling of rebar electrodes between the repair and substrate concrete is reduced due to the high electrical resistivity provided by the UHPC and enhancement due to moisture is mitigated by the low permeability characteristics of the material.



Figure 11. Example of Macrocell Current Data of Steel Embedded in Substrate and Repair Concrete.

Although some extent of corrosion mitigation was apparent with the placement of UHPC as repair material, there are still concerns about the effect of incipient anodes when low level

vestigial chlorides remain adjacent to the repair concrete. Any level of cathodic prevention prior to concrete repair would be loss and would allow a new corrosion leading to the halo effect. This effect would be exacerbated in conditions where the area effect can be magnified. In part to address this concern, the test set-up was configured to allow for various cathode-to-anode (C/A) ratio. **Figure 12 and 13** shows a compilation of test results of C/A ratio for rebar embedded in UHPC or conventional concrete with substrate concrete containing 0, 0.4, 4, and 8% wt chlorides before and after concrete wetting.

Consistent with the earlier OCP and LPR results, low macrocell currents developed in samples with conventional and UHPC repair concretes when the vestigial chloride content in the substrate concrete was low. At higher vestigial chloride contents in the substrate concrete, the macrocell current was enhanced at higher C/A for samples utilizing conventional concrete for the repair concrete. This is consistent with the fast deterioration of conventional patch repairs in marine bridges. In contrast, as mentioned earlier, macrocell current was much reduced in samples repaired with UHPC even with higher vestigial chloride presence in the substrate concrete. In conditions with 0, 0.4, and 4wt% chlorides, the macrocell was negligible, although some cases showed that the steel in the UHPC repair concrete was net anodic. In the condition with 8wt% chlorides in the substrate concrete, some enhanced macrocell developed with increased C/A.

Similar trends in the wet concrete was observed as in the dry concrete; however, as described earlier, macrocell currents were overall higher for all conditions including with the use of UHPC as the repair material. Low macrocell currents developed in samples with conventional and UHPC repair concrete when the vestigial chloride content in the substrate concrete was low. For both NSC and UHPC the macrocell currents increased with higher C/A. In comparison to results for concrete in dry condition, the macrocells were elevated for all concentrations. Identifying material characteristics, corrosion parameters, and environmental conditions would be important to specify practical threshold vestigial chloride content before corrosion damage can propagate in concrete. The results from the laboratory testing would indicate that substrate concrete containing significant chloride content should be removed prior to application of any repair materials including UHPC.



Figure 12. Effect of Cathode-to-Anode (C/A) Ratio for Rebar Embedded in NSC or UHPC Repair Concrete in Dry Condition.



Figure 13. Effect of Cathode-to-Anode (C/A) Ratio for Rebar Embedded in NSC or UHPC Repair Concrete in Wet Condition.

#### CHAPTER 4. COLD JOINTS

In repair applications, UHPC would be cast alongside the substrate concrete and the adhesion between UHPC and substrate is of crucial importance. The other challenging issue pertaining to the cold joint between UHPC and NSC could be the ingress of aggressive ions such as chlorides particularly in marine environments. This can jeopardize the durability of such structures. A component of the research examined the extent to which enhanced chloride transport may occur at the cold joint. The effectiveness of the bond at the concrete interface (with various levels of moisture availability at the time of UHPC repair) to minimize chloride penetration was examined. To this effect, the substrate concrete was conditioned to moisture content (5%, 75%, 100% RH, and wet) prior to and after UHPC repair concrete casting. Chloride penetration was accelerated by an impressed current source, and chloride content penetration was assessed by silver nitrate solution sprayed on the cold joint interface.

Error! Reference source not found. shows test results in terms of total charge passed (TCP) in coulombs for the NSC, UHPC, and composite NSC/UHPC specimens. The plain UHPC specimen exhibited the lowest TCP values (i.e. 8 and 21 coulombs at 7 days and 21 days, respectively), while the plain NSC specimen exhibited TCP value more than 1000 coulombs after 21 days. All the composite specimens, except the composite samples where substrate was conditioned at 0% RH before UHPC placement, exhibited TCP lower but within the same order of magnitude as the plain NSC specimens. This was likely attributed ionic current passing through the NSC portion of the concrete specimen. The high TCP value for of the composite samples where the substrate was conditioned at 0% RH before UHPC placement can be part contributed to the capillary absorption of the concrete substrate leading to quick absorption of chloride solution.



Figure 14. Cumulative Charge Data for Modified Rapid Migration Test

After splitting the samples under load, both surfaces were sprayed with 1M AgNO<sub>3</sub> solutions. The photos after spraying AgNO<sub>3</sub> indicator are shown in Error! Reference source not found. Chloride penetration could be readily observed by the differentiation in surface coloration on the NSC portion of the specimen. Bulk chloride penetration can generally be seen at the upper portion of the specimen and was larger for specimens with higher moisture levels. It was apparent from the color indicator that the large TCP for the composite specimens conditioned at 0%RH occurred largely along the edge of the specimen as moisture from the test solution had preferential transport there.



Figure 15. AgNO<sub>3</sub> Spray Indicator for Chloride Penetration.

#### CHAPTER 5. SERVICE PROJECTIONS

#### 5.1. Background

For conventional concrete repair, damaged concrete elements are patched, overlaid, or encased in new repair materials; but, there can be concrete with some level of chloridecontamination left in place in regions of the element adjacent to the repair. Thus, degradation of the repair in time can occur due to newly-formed corrosion relating to sites with chloride levels exceeding critical threshold concentrations,  $C_T$ . Research has shown that  $C_T$  is in part influenced by the level of steel polarization such as in cathodic prevention [15]. The primary transport mode of chloride ions from the surface of the uncracked concrete to the reinforcement is diffusion, described by Fick's Law. In concrete, chloride diffusion can be assumed to be a semi-infinite and unidirectional process, and that the process corresponds to Fick's second diffusion law [16]. With assumed spatially and temporally constant chloride surface concentration (Cs) and chloride ion diffusivity (D), an expression such as in Equation (4) can be useful to describe chloride penetration at least within a region with similar exposure conditions.

$$C(x,t) = Cs\left(1 - erf\left(\frac{x}{2\sqrt{Dt}}\right)\right)$$
 Eq (4)

where Cs = surface chloride concentration (kg/m3), t = time of exposure to Cs (s), D = chloride diffusion coefficient (m<sup>2</sup>/s).

In marine environments, moisture availability and exposure to chlorides will vary along the vertical elevation. D and Cs will vary in the immersed, tidal, splash, and atmospheric exposure regions. Even for the same concrete mix used for a substructure element, D and Cs can vary due to moisture content. In the tidal and splash regions Cs reach relatively constant high values early in the life of substructures in estuarial waters due to chloride accumulation in wet and dry cycles by a wave and tidal action [21]. A suggested chloride threshold level to initiate corrosion has been cited at 1.2 kg/m3 of acid-soluble chloride [22-24]. Formation of expansive corrosion products generate tensile stresses in the surrounding concrete of the corroding steel reinforcement and thus eventually causes the concrete to delaminate and spall.

The interface of repair and substrate concrete delineates zones of differential concrete bulk material characteristics and pore water chemistry. After removal and replacement of the chloride-laden damaged concrete and surface cleaning of the rebar within the repair patch, the steel potential within the patch and thus polarization of adjacent steel (including beneficial cathodic polarization due to the initial reactions at the initial anode) would change. This can result in new corrosion sites forming in the surrounding chloride-contaminated concrete. In this phenomenon, known as "halo effect," "ring anode or "patch-accelerated corrosion," the steel bar within the repair concrete will transition from anode to net cathode while the steel in the existing concrete adjacent to the patch serves as an anode [25-26]. Macrocell corrosion is enhanced (sometimes greatly) in chloride-induced corrosion of steel bars embedded in concrete with a small anode and a large cathode [27]. Macrocell corrosion can happen when the actively corroding bar is coupled to a passive bar, either due to its different composition or environment. The latter situation can occur when part of a steel rebar in chloride-contaminated concrete is coupled to the steel rebar in chloride-free concrete (repair condition) [28]. This phenomenon can become more aggressive when the repair area is exposed to the wet environments where concrete resistivity would be lower [29].

#### 5.2. Service Calculations

In the practical application of repair concrete in marine concrete substructures, the placement and geometry of the repair patch can significantly affect the durability of the structural element. Macrocell behavior can be completed due to numerous material and environmental conditions that can include concrete resistivity, oxygen availability, spatial geometric non-homogeneity, etc. Modeling efforts of macrocell development can be complex requiring many system variables. The work here does not pose to account for the many variables and instead seeks to provide a rough estimation of macrocell behavior with the application of UHPC relative to NSC based on information obtained from comparative laboratory testing of those materials. In part to assess application of UHPC repair, a simplified approach to calculate service projections was followed based on chloride diffusion and macrocell coupling. Estimation of chloride diffusion coefficient (D) and surface chloride concentration (Cs) from data sets of Florida bridges [23,30] was used for simple damage projection calculations.

For the calculations, simplifying assumptions and definitions were made. Corrosion of the steel was accounted only by macrocell currents which would underestimate actual corrosion rates. As such, the calculations output can only be used qualitatively in its comparison of the repair materials. The macrocell currents were obtained from the experimental work described above with the material and geometric constraints of the small-scale test samples. Acceleration of corrosion rates due to macrocell coupling was only accounted by the experimental data for a cathode-to-anode ratio of 5:1. However, this may not necessarily reflect larger extended geometries and conditions where cathodic regions may also be extended in the substrate concrete. Also, the calculations only account for a development of a small anodic region at the regions adjoining the repair patch (as these locations would have the highest chloride levels and loss of cathodic prevention (i.e. halo effect) due to repair). Corrosion at other locations including within the repair concrete was discounted. For simplicity, the chloride ion diffusivities were discretized and assumed to be only related to the quality of the concrete mix and did not account for internal moisture content.

For these calculations, a discrete anode length was assumed to develop. For simplicity, this length was assumed to be 1 cm. The bar diameter used in the calculation was 1.2 cm with a clear cover of 7.6 cm.

From the experimental work described above, macrocell currents developed between local anodes in the substrate concrete and the steel in the repair material. These macrocell currents were greater at higher chloride concentrations in the substrate concrete and furthermore enhanced in wet conditions. For the simple damage projection calculations, the macrocell current magnitudes were discretized based on the chloride levels used for testing (0, 0.4, 4, and 8% wt Cl-) as shown in **Figure 16.** Based on the chloride levels calculated by simple diffusion transport at time t, the macrocell current corresponding to the next higher discretized chloride step was selected. This was thought to be a conservative approach given the rather coarse level discretization. The cumulative mass loss (based on Faradays Law) was calculated using the given macrocell current for the time period required for chlorides to diffuse to the next higher chloride level step. The process was repeated (summing up the total mass loss) with each successive step of chloride level concentration (and the corresponding macrocell current). If the chloride concentration at the bar cover depth already exceeded lower steps, the next higher macrocell level was assumed. If chloride levels exceed 8%, the macrocell current at 8% was retained.



Figure 16. Macrocell Current Densities

In the first set, the comparative performance of repair with UHPC and NSC was made for various cases shown in **Figure 17**. For this set, three exposure zones were defined; Damage zone, Tidal zone, and Upper splash zones.

**Table 5** lists the defined diffusion parameters approximated from data for Florida bridges [23,30]. Chloride ion diffusivity was in the order of  $10^{-12}$  or  $10^{-11}$  m<sup>2</sup>/s. The damage zone refers to a section that had initial spalling and delamination due to chloride induced corrosion. Moreover, durability projections were made for cases with partial concrete repair and complete concrete repair. Assumptions for a partial repair were that after removal of degraded concrete, there was some level of sound base concrete that maintained elevated chloride levels based on concentrations calculated from **Equation (4)** for the time for initial corrosion initiation given the parameters for the respective exposure zones. It was assumed that the partial repair was made with adjoining concrete with diffusion characteristics of the tidal zone. Likewise, assumptions for a complete repair were that concrete in the tidal regions was fully removed and adjoining concrete had diffusion characteristics of the upper splash zone.



Figure 17. Calculation Approach to Compare Repair Service Time

Zana	Base C	Concrete 1 Base Concrete 2 (C2)		
Zone	D (m <sup>2</sup> /s)	Cs (kg/m <sup>3</sup> )	D (m <sup>2</sup> /s)	Cs (kg/m <sup>3</sup> )
Damage	$1 \times 10^{-12}$	25	$1 \times 10^{-11}$	25
Tidal	$1 \times 10^{-12}$	25	$1 \times 10^{-11}$	25
Upper Splash	$1 \times 10^{-12}$	15	1x10 <sup>-11</sup>	15

Table 5. Exposure Zones



Figure 18. Calculation Approach to Assess Repair Application.

In the second set, calculations were made to identify concrete and environmental conditions that can provide extended repair service life for the UHPC and NSC repairs. The base concrete quality was considered in these calculations quantitatively by chloride ion diffusivity (**Figure 18** and **Table 6**). Five conditions were considered and are shown in **Table 6**. Chloride exposure levels (for example in different natural water bodies or by elevation) were considered with discrete chloride surface concentrations also listed in the table.

Base Concrete Quality (Chloride Diffusivity
$m^2/s$ )
$1 \times 10^{-12}$
$5.5 \times 10^{-12}$
$1 \times 10^{-11}$
$5 \times 10^{-11}$
$8 \times 10^{-11}$
Chloride Surface Concentration (kg/m <sup>3</sup> )
35
25
15
10

Table 6. Concrete and Environmental Conditions

#### 5.3. Calculation Results

#### 5.3.1. Comparison of Repair Service Time

**Figure 19** shows the calculation output results for conjectural cases for 2 concrete mixes (Base Concrete 1 and 2) characterized by its chloride ion diffusivity:  $1 \times 10^{-12} \text{ m}^2/\text{s}$  and  $1 \times 10^{-11} \text{ m}^2/\text{s}$ , respectively. For those concretes, a hypothetical repair was made by replacing damaged concrete either within the tidal zone (partial repair) or in the upper splash zone (complete repair) with UHPC or with NSC.

Generally, the influence of the substrate concrete quality was readily evident. For the conjectural cases, the calculated chloride levels in the zones adjacent to the repair concrete at the time of repair already exceeded 0.4%. Therefore, following the calculation protocol, the time for diffusion of chlorides to 4% was used for initial calculations for initial macrocell corrosion. Base Concrete 2 with  $D=1x10^{-11}m^2/s$  reached that initial chloride step 10 times faster than Base Concrete 1 with  $D=10x10^{-12} m^2/s$ . In a similar way, the extent of the repair material (i.e. partial or complete repair) was evident by the diffusion of chlorides where longer service time to reach 4% chloride was evident for the complete repair compared to the partial repair. Mass loss was faster for the calculation conditions assuming the concrete was in wet condition due to the enhanced macrocell currents there.



Figure 19. Calculation Results for Repair Service Time

**Figure 20** shows a compilation of calculations for repair service time for 1% corrosion damage. It was evident that extended service time in comparison to conventional patch repairs can be afforded by application of UHPC repair based on the assumptions of the calculation approach. The effect of substrate concrete quality, moisture level, and repair placement can also be summarized in the figure.



Figure 20. Calculation Results for Comparative Time to 1% Damage.

#### 5.3.2. Assessment of Repair Application

**Figure 21** shows calculation output results for conjectural cases for 5 different concretes characterized by chloride ion diffusivity ranging from  $1 \times 10^{-12}$  m<sup>2</sup>/s to  $8 \times 10^{-11}$  m<sup>2</sup>/s for 1% corrosion of rebar after repair with UHPC or NSC. The service life to 1% corrosion was calculated for various chloride exposure environments characterized by Cs. Only macrocell currents from laboratory testing in the wet condition were considered here as similar trends but with longer service time were calculated for dry conditions. Generally, the influence of low macrocell currents measured for UHPC samples compared to NSC samples is readily apparent in the calculation results. Much longer time to damage after repair was calculated for the UHPC repair than NSC concrete repair.



Figure 21. Output Results for Conjectural Cases

The simplifying assumptions in the calculation approach would not necessarily provide accurate service times but the results due provide information to quantitatively compare the effect of repair with UHPC compared to NSC for the given assumptions based on empirical data on macrocell development and mass diffusion. Even though the macrocell current step (used in the calculation for a given chloride level) used current values from laboratory testing at higher chloride levels, the enhancement of the macrocell in the lab sample may not necessarily reflect actual structures due to the fact that extended cathodes in poor quality substrate concrete where coupling could be enhanced. However, comparative conjectural cases with NSC concrete have similar limitations.

For better utilization of these simplified calculation approaches, consideration of microcell corrosion and then enhancement due to macrocell coupling can be considered. Also the effect of concrete resistance and oxygen availability were not explicitly considered. Beyond the influence on the developed macrocell determined empirically. Another major simplifying assumption that would not be realistic for damage projections for extended times was that corrosion at other locations were discounted and only corrosion at the repair boundary was considered. Past experiences with concrete repair with encapsulation materials including high performance concretes and composite materials have shown mixed results. These experiences must be considered for any repair protocols. Nevertheless, even with these precautions, lab testing and service life calculation with grossly simplified system geometry and simplified treatment of macrocell development, do show trends of benefit provided by UHPC.

### CHAPTER 6. CONCLUSIONS

In this study, the extent of macrocell development between the dissimilar concrete materials was examined to identify possible benefits and challenges of utilizing UHPC as a repair material for reinforced concrete marine bridges. The following conclusions were drawn for corrosion protection based on the results from small-scale cast concretes and durability projections for idealized test cases using a simple approach to calculate service life of the repair, based on simple diffusion behavior

- Only minor to no increase in mass was observed for UHPC in all of the tested moisture exposure conditions possibly due to self-dessication of concrete.
- UHPC showed bulk resistivity up to an order of magnitude larger than the tested conventional concrete, consistent with its higher cement factor and relatively low internal moisture content due to its low permeability.
- The calculated approximate oxygen diffusivity for UHPC was much lower than the conventional concrete, and lower diffusivity was observed in UHPC exposed in immersed conditions than in ambient humidity conditions. Therefore, development of corrosion cells is expected to be mitigated due to low gas permeability.
- UHPC generally showed lower internal relative humidity consistent with low internal moisture content.
- Low macrocell currents developed in samples with conventional and UHPC repair concretes when the vestigial chloride content in the substrate concrete was low.
- Even though higher current was observed in wet condition compared to dry condition, the increase in samples repaired with UHPC this value was significantly lower than the comparative one with NSC.
- At higher chloride contents in the substrate concrete, the macrocell current was enhanced at higher C/A for samples utilizing conventional concrete for the repair concrete and this trend was more highlighted after wetting.
- In context to the local coupling of local anodes to cathodes in repair material, the measured macrocell current was much reduced in samples repaired with UHPC even with higher chloride presence in the substrate concrete than the conventional concrete even after wetting the samples.
- The influence of low macrocell currents measured for UHPC samples compared to NSC samples was readily apparent in the calculation results. Much longer time to damage after repair was calculated for the UHPC repair than NSC concrete

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