Report Prepared by:

Kingsley Lau and Alberto A. Sagüés

CORROSION OF STEEL IN LOCALLY DEFICIENT CONCRETE Contract No. BD544-31 Final Report to Florida Department of Transportation

A. A. Sagüés Principal Investigator Department of Civil and Environmental Engineering

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This investigation confirmed prior noted trends of extensive preferential chloride intrusion at preexisting cracks in a majority of cases of substructure members in Florida bridges built with low permeability conventional concrete. Corrosion clearly associated with the presence of preexisting cracks was found in a 15-year old Howard Frankland Bridge built with Epoxy Coated Rebar. Local corrosion penetration at the affected locations approached 1 mm. Laboratory experiments suggests that increased oxygen access at a preexisting crack is not necessary for the development of expansive corrosion products. For bridges with low chloride diffusivity carbonation penetration into the surface of the concrete at elevations exposed to seawater splash was typically negligible in both sound and cracked concrete barring an isolated instance. In contrast, on-crack carbonation depth was large and even surpassed reinforcement depth at higher elevation locations with drier concrete. A model for projecting impact of preexisting cracking on spall damage was developed based on working assumptions. Model projections indicated that even though assuming that the incidence of damage is limited to a small region around the crack, if the crack orientation with respect to the rebar were adverse and chloride transport were greatly enhanced (as it could be expected in relatively wide cracks), corrosion damage from localized concrete deficiencies could significantly increase damage by as much as 300%. It is recommended that continuing monitoring takes place at known deficiencies in otherwise high quality concrete structures. Judicious application of the predictive model is recommended to explore cost effectiveness of alternative corrosion protection methods such as corrosion resistant rebar to mitigate the effect of local concrete deficiencies. The technical content of this investigation is closely supplemented by that of the companion FDOT project, "Corrosion Evaluation of Bridges with Epoxy-Coated Rebar", BD544-23.					
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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the State of Florida Department of Transportation.

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*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

EXECUTIVE SUMMARY

Note: The technical content of this investigation is closely supplemented by that of the companion FDOT project, "Corrosion Evaluation of Bridges with Epoxy-Coated Rebar", BD544-23. To best use available resources, field surveys and related analysis were conducted concurrently for both projects. For contractual reporting purposes the findings under both projects are reported in separate documents. However, for technical discussion and elaboration of conclusions it has been often necessary to refer to and reproduce here some of the material from the Final Report for Project BD544-23, cited accordingly. It is recommended that both documents be consulted for an integral view of the issues concerned.

Corrosion of reinforcing steel in FDOT's structural inventory occurs mainly in marine bridge substructure, due to concrete penetration of chloride ions that cause passivity breakdown at the rebar surface. With few exceptions chloride penetration in bridges built more than about 20 years ago was relatively fast, since the concrete used at the time was often much more permeable, and rebar cover thickness lower, than those currently specified. Present reinforced concrete FDOT guidelines Section 346 result in extremely slow chloride buildup at the rebar depth in sound concrete. This slow chloride buildup is due to thicker rebar cover and a greatly reduced bulk diffusivity of chloride ions through the concrete. However, some incidence of concrete deficiencies is unavoidable in actual structures even under strict quality control.

Notably, thin cracks (e.g. 0.1mm wide) from mechanical/thermal origin have been regularly observed in FDOT substructures with both cast in place and precast members, typically appearing one every few feet of waterline perimeter of substructure Although thin, those cracks usually reach down to the rebar depth providing a local but fast pathway for chloride ion penetration, which has been well documented for several modern FDOT bridges, especially at low elevations where the concrete is moist. Examination of the potential for and extent of corrosion at those locations was necessary and this investigation was conducted to that end.

The present investigation confirmed prior noted trends of extensive preferential chloride intrusion at preexisting cracks in a majority of cases of substructure members in Florida bridges built with low permeability conventional concrete. An exception noted from previous research was cylinder piles in bridges several decades old. Preferential chloride intrusion was not noted in bridges with relatively high permeability concrete as in those cases bulk diffusion was rapid enough to mask any faster transport through cracks. Corrosion clearly associated with the presence of preexisting cracks was found in only one structure, the Howard Frankland Bridge, 15 years old at the time of inspection and built with Epoxy Coated Rebar (ECR). Corrosion there was severe at multiple locations where preexisting cracks intersected rebar. Local corrosion penetration at the affected locations approached 1 mm. It is recommended that those and similar locations be carefully monitored in the future.

Examination of corrosion products showed them to grow underneath the epoxy coating and containing chloride ions. The composition of the products was consistent with the makeup of chloride-substituted Akaganeite which can support significant local acidification in a crevice environment. Laboratory experiments showed that local expansive in-crevice corrosion products formed similarly in epoxy-coated rebar under anodic polarization in simulated concrete pore solution with chlorides regardless of the presence or absence of oxygen outside the crevice. The result suggests that increased oxygen access at a preexisting crack is not necessary for the development of expansive corrosion products.

There was no clear differentiation in surface concrete resistivity on cracked and sound locations in the field. Overall moisture content in the bulk concrete material likely has a greater effect on surface resistivity than crack presence. However, electrochemical impedance measurements indicated that substantial moisture penetration can take place at crack locations with consequent potential for locally accelerated corrosion.

For bridges with low chloride diffusivity carbonation penetration at elevations exposed to seawater splash was typically negligible in both sound and cracked concrete, except for one case in concrete with an external coating where on-crack carbonation extended beyond reinforcement depth. Deep on-crack carbonation was noted at high elevations in pile caps where concrete was drier. Although chloride penetration was small there, continuing monitoring of those locations is recommended. Except for one isolated case, carbonation penetration was also typically negligible both on and off-crack locations of the splash zone of high to medium chloride diffusivity bridges.

A model for projecting impact of preexisting cracking on spall damage was developed based on working assumptions. While recognizing that validation of the model will need to await development of data over a longer time period, the projections indicate that, as expected, relatively isolated cracking should only create topical concrete damage with reduced maintenance requirements. However, model projections indicated that even though assuming that the incidence of damage is limited to a small region around the crack, if the crack orientation with respect to the rebar were adverse and chloride transport were greatly enhanced (as it could be expected in relatively wide cracks), corrosion damage from localized concrete deficiencies could significantly increase maintenance costs. For certain service times the added projected damage was in the order of 300% greater than for sound concrete in some of the scenarios addressed.

It is recommended that continuing monitoring takes place at known deficiencies in otherwise high quality concrete structures. Judicious application of the predictive model developed in this project may aid in exploring the cost effectiveness of alternative corrosion protection methods, such as corrosion resistant rebar to prevent local damage for a given extent of existing or anticipated concrete deficiency.

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The assistance with field inspections of many collaborators at the FDOT State Materials Office and from Concorr Florida, Inc. is greatly acknowledged, as is that of numerous student participants in the University of South Florida College Of Engineering Research Experience for Undergraduates program and staff from the USF Nanomaterials and Nanomanufacturing Research Center. The assistance of Mr Ivan Lasa of the FDOT State Materials Office in providing corrosion condition information for the Group 1 bridges is particularly acknowledged.

1 INTRODUCTION

Note: The technical content of this investigation is closely supplemented by that of the companion FDOT project, "Corrosion Evaluation of Bridges with Epoxy-Coated Rebar", BD544-23. To best use available resources, field surveys and related analysis were conducted concurrently for both projects. For contractual reporting purposes the findings under both projects are reported in separate documents. However, for technical discussion and elaboration of conclusions it has been often necessary to refer to and reproduce here some of the material from the Final Report for Project BD544-23, cited accordingly. It is recommended that both documents be consulted for an integral view of the issues concerned.

1.1 Project Scope

Corrosion of reinforcing steel in FDOT's structural inventory occurs mainly in marine bridge substructure, due to concrete penetration of chloride ions that cause passivity breakdown at the rebar surface. With few exceptions chloride penetration in bridges built more than about 20 years ago was relatively fast, since the concrete used at the time was often much more permeable, and rebar cover thickness lower, than those currently specified. An FDOT-sponsored investigation completed in 2002 [1] showed that present reinforced concrete FDOT guidelines Section 346 result in extremely slow chloride buildup at the rebar depth in sound concrete. This slow chloride buildup is due to thicker rebar cover and a greatly reduced bulk diffusivity of chloride ions through the concrete. The latter reflects use of a high cement content, low water/cement ratio, and pozzolanic admixtures all of which dramatically decrease the effective connectivity of the pore network.

Greatly reduced bulk chloride transport efficiently controls corrosion over nearly all of the exposed substructure, where concrete is free of cracks or other local deficiencies. However, some incidence of those deficiencies is unavoidable in actual structures even under strict quality control. Notably, thin cracks (e.g. 0.1mm wide) from mechanical/thermal origin have been regularly observed in FDOT substructures with both cast in place and precast members, typically appearing one every few feet of waterline perimeter of substructure [1]. Although thin, those cracks usually reach down to the rebar depth providing a local but fast pathway for chloride ion penetration. Fast chloride penetration at cracks has been well documented for several modern FDOT bridges, especially at low elevations where the concrete is moist [1]. Lift lines, where consecutive concrete lifts failed to fully join (e.g in footers of the McArthur Causeway bridge) and cold joints in general offer similar potentially fast pathways for chloride ingress in otherwise highly impermeable concrete. Additional constructions deficiencies such as local poor consolidation, and post concreting damage as in pile driving distress, uneven post tensioning and watercraft impact may offer similar locally fast chloride ingress.

If the local deficiency is narrow (as in a crack) and not aligned with rebar, only a very small fraction of the rebar surface area is expected to be immediately affected by the locally incoming chloride. However, once initiated corrosion may propagate into the adjacent steel surface because of local acidification associated with the anodic reaction, and further intensified by macrocell coupling with cathodic reactions on the surrounding passive steel. Both experimental and modeling studies indicate the possibility of developing local corrosion rates that may be e.g. one order of magnitude greater than those experienced under more uniform corrosion conditions [2]. Such deterioration could lead not only to spalling of the concrete cover but also to severe enough rebar cross section loss to compromise overall mechanical strength.

Mitigating factors may be present as well, including self-healing of thin cracks by carbonate precipitation and other accumulations, formation of solid corrosion products that seal the crack zone, reduced macrocell coupling due to elevated electric resistance of the high quality concrete, low moisture retention except at the very lowest elevations, and greater resistance to spalling when only a small portion of the rebar is affected [3]. In the case of bridges built using epoxy-coated rebar (ECR), additional mitigation would apply if the crack or deficiency reaches the rebar surface at a place where the coating is free of breaks. Macrocell action would also be significantly lowered if the rest of the coated bar surface is mostly free of flaws.

At present there is insufficient information to determine whether aggravating or mitigating factors for corrosion at local concrete deficiencies are predominant in FDOT bridges. If the former predominate, it will be essential to develop strategies to address repair of existing structures when local corrosion damage appears, and also design guidelines to minimize or avoid local deterioration in new structures. Those guidelines may involve costly special concreting practices to strictly avoid minor deficiencies, and/or extensive use of corrosion resistant reinforcement in high risk areas. Judicious continuation of the present construction approach, with considerable lower expense, would be in order if mitigating factors are found to prevail. Therefore, significant benefit in deciding on appropriate design and rehabilitation guidelines can be derived from assessing the likely extent of corrosion damage associated with locally deficient concrete in FDOT aggressive service conditions.

This research was conducted concurrently with FDOT project BD544-23. For contractual reporting purposes the findings under BD544-23 are reported in a separate document [4]. However, for technical discussion of present findings it has been often necessary to refer to and reproduce here some of the results from BD544-31.

1.2 Objectives

The objective of this investigation is to assess the present and future extent of corrosion at localized concrete deficiencies in FDOT marine bridges.

The approach to achieve those objectives is to (1) review the available relevant evidence in the technical literature; (2) conduct field inspections of existing bridges and extract concrete and steel samples; (3) evaluate field samples in the laboratory including experiments with simulated systems for comparison as needed; (4) formulate and operate a model to predict corrosion damage in existing bridges and to evaluate corrosion prognosis in new structures

1.3 Approach

Tasks keyed to each approach item are listed below.

<u>Task 1 - Literature Review</u>. Reports from prior and current FDOT evaluation of structures with local concrete deficiencies were evaluated, along with those from other sources in the technical literature.

<u>Task 2 - Field survey and sample extraction</u>. Thirteen FDOT bridges were evaluated, selected primarily from those already evaluated in the previous investigation [1] which represent new construction methods and for which detailed documentation on cracking/lift lines already exists. Major bridges in that group include the Sunshine Skyway bridge and the Howard Frankland Bridge. Sample extraction of structures was made concurrent with field activities being conducted under FDOT contract BD544-23 "Corrosion Evaluation of Bridges with Epoxy-Coated Rebars" [4]. Surveys and extractions were conducted in cooperation with personnel of the Corrosion Laboratory, FDOT State Materials Office.

<u>Task 3 - Laboratory evaluation</u>. Selected concrete samples were analyzed to obtain chloride penetration profiles and assess preferential ingress along deficiencies. This activity was conducted only to the extent needed to confirm already available information from project BA502 or elucidate uncertain situations. Laboratory work focused on determining rebar condition at distressed versus nearby sound concrete, aimed to establish differences in corrosion modalities and quantitative corrosion rate, to be used as input for the predictive model developed under Task 4. In addition, laboratory tests were conducted to assess whether the formation of corrosion products inside crevices in epoxy-coated rebar depended on the extent of oxygen access there. Those experiments sought to establish whether enhanced oxygen access at a crack would result in different corrosion products than in the case of corrosion under otherwise sound concrete.

<u>Task 4 - Predictive modeling</u>. The information from the previous tasks was used to prepare a predictive model of the extent of local corrosion to be expected in a generic FDOT structure having extreme levels of local deficiency incidence. The model inputs included relevant concrete and exposure properties (e.g. concrete class, environmental chloride, elevation), severity of local distress Model output consists of damage) as function of time in service.

The model uses the statistical approach developed under FDOT project BB-259 [5], in which the distribution of damage over the entire affected portion of the structure is considered. A generalized treatment of the problem is found in Reference [6]. The model also incorporates features developed in the companion FDOT project BD544-23 [4].

Model results were parameterized based on likely input ranges to establish scenarios that can bracket the range of observed conditions and anticipate the future FDOT structural inventory, and the model calibrated to the extent possible to best match the observed range of localized damage as exemplified by previous work [5,6]. The relevance of these findings to the need for possible methods of controlling that risk was discussed.

2 REVIEW OF RESULTS FROM RECENT INVESTIGATIONS

Recent FDOT investigations have included surveying of marine bridges where concrete cracking had been observed. A summary of pertinent results to this investigation are included here.

In a 2002 investigation [1], corrosion assessment of 10 bridges with concrete cracking including the Sunshine Skyway Bridge, Howard Frankland Bridge, Courtney Campbell Causeway, McArthur Causeway, Dames Point Bridge, Bay Side Bridge, Safety Harbor Bridge, New Smyrna Bridge, Boca Ciega Bay Bridge, and New Pass Bridge was made. Many stress cracks in footers of piles reached down to the waterline. Shrinkage cracks were also noted. Incidences in the order of one stress crack every several meters of waterline perimeter were not uncommon. Shrinkage cracks were normally superficial and stress cracks often penetrated past reinforcement depths. Some of the stress cracks tended to show efflorescence. Of the bridges in the investigation, the median surface stress crack width was ~0.15 mm. For most of the bridges in the investigation, examination of the chloride profiles showed significantly higher chloride concentration in cracked core samples than core samples on sound concrete. The observed enhanced chloride penetration at cracks is notable in that many of the cracks were narrower than the value of 0.3 mm sometimes guoted as the minimum for concern. A general tendency toward lower values of chloride ion diffusivity, D, at higher elevations was observed. Median D values ranged from 1.6x10⁻⁹ cm²/s for Sunshine Skyway Bridge to 2x10⁻⁸ cm²/s for New Smyrna Bridge, consistent with bridge age and concrete mixture proportions Median chloride surface concentration, Cs, was 12-24 kg/m³.

In a 2005 investigation [7,8], corrosion assessment of 4 bridges utilizing spin-cast cylinder pile construction including the old Hathaway Bridge (no longer in service), Pensacola Bay Bridge, Brooks Bridge, and the new St. George Island Bridge was made. The cylinder piles in the structures had many instances of fine (<0.3 mm width) longitudinal cracks that often penetrated deep into the concrete. It was noted that the thickness of the cracks fluctuated along the length of extracted concrete core samples. Examination of the chloride profiles showed some limited indications of enhanced chloride penetration but much less developed than examples of enhanced chloride mentioned above. The range of surface chloride concentrations were similar to those encountered in other Florida marine bridges built with conventional concrete. The observed chloride diffusivities were small ($\sim 10^{-9}$ cm²/s) which caused chloride penetration to be areatly reduced in the concrete bulk. Even though thin cracks were frequently observed, inward chloride penetration appeared to be ruled mostly by bulk parameters. The steel corrosion survey failed to show any examples of clear association between prior presence of cracks and preferential corrosion initiation even in bridges in service for over 40 years.

3 FIELD CORROSION ASSESSMENT OF FDOT MARINE BRIDGES WITH CONCRETE CRACKING

Note: Examination of the bridges surveyed in this investigation was concurrent with field activities conducted under FDOT contract BD544-23. As such, bridge group nomenclature will be kept consistent with that in the companion report [4], from which portions are reproduced in the following for completeness and clarity. The organization of the material in the present report is different from that in [4], in that here chloride penetration in cracked concrete is discussed separately in Section 4, as it is central to the objective of the present project. This Note also applies to subsequent Sections of this Report as to reproduction of excerpts from [9] as needed for completeness and clarity.

3.1 Bridge Survey for Current Investigation

3.1.a Bridges Investigated

Table 1 lists the structures of all groups, construction information, reinforcement type, and the bridge identifications used in the rest of the report. Table 2 includes bridges from earlier investigations [1,7]. Group 1 bridges as well as CH2 are included for completeness in conjunction with the companion report but will not be further discussed further for this investigation. The VA1/2 and SNK bridges have capped drilled shafts supporting columns. The CHO bridge has reinforced concrete columns with connecting struts, supported by capped prestressed piles. The SSK substructure consists of reinforced concrete columns with footers and struts in the low approach spans and elliptical posttensioned columns for the high approaches. The PER substructure consists of reinforced concrete piles for the low approach and reinforced concrete columns on footers for the main span. The HFB substructure consists of reinforced concrete columns on footers. Substructure in marine service for NWR, ITA, and ITB include the bascule and rest piers. IT2 and IT3 consist of reinforced concrete columns with footers. The substructure on these bridges was painted with a texture coating above the high tide level. Information on the cementitious content of the concrete and use of pozzolanic cement replacement is given in Table 1. Additional details are provided in the companion report for BD544-23 [4].

3.1.b Field Investigation Methodology

Field bridge evaluation included visual surveying for concrete deficiencies such as cracking, concrete core sampling, and testing of probable corrosion status. A general visual survey was made along the entire span of bridge. Locations from several substructure component types (columns, footers, struts) were selected for examination, focusing on cracks on concrete sections at low elevations exposed to sea splash. Typically, locations with the most severe cracking were selected for examination. For SSK, higher elevation locations (~7-8 m above high tide) where wide cracks were observed were also examined. Elevations are reported as distance above the high tide level (AHT). Concrete clear cover was noted and checks for concrete delamination were made by hammer sounding. When cracks were observed, pairs of cores were collected unless otherwise indicated along the same elevation typically ~15 cm apart on center with one core centered on crack. Half-cell potentials were measured with a copper/copper-sulfate reference electrode (CSE) along the elevation of the substructure component. Concrete surface resistance was measured using a Wenner array probe with an inter-probe spacing of 5 cm, chosen as a compromise between sampling size and possible interference from embedded rebar. Concrete porosity was measured following ASTM C642-97. Steel reinforcement was extracted and examined for evidence of active corrosion. Further details are given in the companion report for BD544-23 [4].

3.2 Field Investigation Results

The following subsections describe the observations for each bridge examined. Refer to Appendix I for graphic presentation detailing concrete crack findings and cores extracted for each structure, and to Appendix II for concrete surface resistivity field data.

3.2.a Group 2 Bridges (High D_{CI}- Bridges)

<u>Vaca Cut (VA1/2) and Snake Creek (SNK)</u>. For simplicity, the side-by-side bridges VA1 and VA2 are treated as one bridge in the following¹ unless indicated otherwise. The substructure from VA1/2 and SNK has only reinforced concrete drilled shafts in contact with the seawater. Concrete deterioration was generally inconspicuous with the exception of vertical cracks (0.08-0.3 mm wide) on one shaft each in VA1 and VA2 and one in SNK, out of a combined total of 26 shafts in water for those bridges. The drilled shaft containing the largest crack in VA1/2 (0.3 mm width, VA1 (Figure 1), 70 cm AHT, ~13 cm deep had also internal cracks (diagonal and transverse), leading from reinforcement depth, that had not yet propagated to the concrete surface. The crack at SNK was 0.08 mm thick, ~30 cm high from 4 cm below high tide line to 26 cm AHT, and ~18 cm deep. The reexamined drilled shafts did not have any discernable deterioration. Concrete delamination could not be detected by hammer sounding on the any of the concrete sections (sound or cracked) from either bridge likely due to the large concrete cover (~13-15 cm).

In VA1, two shafts were cored in five locations fully exposing reinforcement in three cores. Two of those cores were an on-crack and off-crack pair at the largest vertical crack, 46 cm AHT. The reinforcement, both on- and offthe main crack showed extensive corrosion despite the use of fusion-bonded

¹ This follows a recent FDOT change in numbering, to designate bridges 900124 (VA1, southbound) and 900126 (VA2, northbound) under the single number 900126.

epoxy coating. Lesser but still significant corrosion distress was observed on samples from a core at 165 cm AHT. There, the distress was limited to rusty appearance of exposed steel under epoxy coating breaks. Epoxy coated steel reinforcement was fully exposed at four core locations in SNK, including a low elevation on- and off-crack pair that showed significant corrosion but not as severe as in VA1/2. Half-cell potential mapping of the epoxy coated reinforcement generally showed potential values (more negative than -300mV CSE), traditionally indicative of corrosion activity for plain rebar in atmospherically exposed concrete. Although potential may not be a reliable indicator of active corrosion of coated steel in marine concrete, it is noted that all the rebar which had showed visual signs of corrosion were similarly negative. Concrete resistivity (Figure 2a) reached <5 k Ω -cm, indicative of highly permeable concrete. A general trend of lower resistivity at low elevations was consistent with expectations of near water saturation there. The large aggregate consisted of limestone. The volumetric porosity of the concrete from both bridges was high, $\sim 20\%$ (Figure 3), consistent with the high permeability observed

William Marler (CHO). Two of 20 footers from the ten high elevation piers had cracks wider than 1.0 mm; ten of the footers had cracks larger than 0.2 mm. Minor concrete cracking was typical on all of the footers. Vertical and map-type cracking was observed on one of 20 columns from the high elevation piers. Rust bleedout was observed on two columns and one strut. The origin of that bleedout was not confirmed as samples of reinforcement were not obtained, but corrosion of reinforcement may be possible. Core samples were extracted from two footers each with one wide vertical crack (1.0 and 0.63 mm wide respectively). Approximately 0.42 m² and 0.1 m², respectively, of concrete around the crack locations seemed to be delaminated as determined by hammer sounding. Extensive corrosion of the reinforcing steel was observed on both footers at elevations 8.9 to 17.8 cm above high tide level, both where the crack intersected steel and in adjacent sound concrete locations. The concrete cover to vertical bar ranged from 7.1-10.4 cm; nominal design cover was 10.2 cm. Highly negative half-cell steel potentials were measured, -400 to -552 mV CSE. In plain steel rebar such potentials would likely be reflective of the observed ongoing corrosion, but it is cautioned that potential readings in epoxy coated rebar, especially in wet concrete, may not be always reliable indicators of corrosion condition. Concrete surface resistivity measurements on the columns ranged from 46 to 128 k Ω -cm on the columns at elevations 0.6 to 2.4 m AHT. Concrete resistivity on the footer ranged from 16 to 63 k Ω -cm at elevations 0 to 0.55 m AHT. The large aggregate consisted of river rock. The volumetric porosity of the concrete was ~14%.

3.2.b Group 3 Bridges (Low D_{CI}- Bridges)

<u>Sunshine Skyway (SSK)</u>. The low approach span substructure consists of 512 reinforced concrete columns with footers and struts exposed to direct sea splash, and 256 cap beams at ~7 m AHT. The high approaches have elliptical

post-tensioned columns. The average clear cover of the outer mat steel ranged from 9-11 cm in the various substructural components.

At low elevations, hairline cracks (<0.03 mm) were commonly observed on the concrete footers (<60 cm AHT) and columns (<200 cm AHT). Larger vertical cracks (~0.3 mm, (Figure 1)) with efflorescence were found on the elliptical posttensioned columns. No concrete delamination was observed or detected by hammer sounding on any cracked or sound sections. A total of 16 cores were extracted from these low elevation locations. In the field, no evidence of corrosion was observed on the surface of the epoxy coated reinforcement exposed by coring, except for vestigial rust at small coating breaks such as high points on ribs where the coating had been damaged during or before construction. That rust did not appear to reflect ongoing corrosion. Half cell potentials ranged from values indicative of passive behavior to <- 600 mV. The more negative values were observed at some (but not all) of the lowest elevations. As mentioned above, the significance of these values is limited. The surface resistivity of the concrete at elevations where the concrete was very wet (e.g. <=0.3 m AHT) (Figure 2b) ranged from (~15-150 k Ω -cm), consistent with the low permeability concrete used in this bridge. The large aggregate consisted of limestone. The volumetric porosity of the concrete was ~20%.

The trestle cap beams, at elevations ~7-8 m AHT, often had wider structural cracks (up to 0.6 mm, (Figure 1)), some with heavy efflorescence. No concrete delamination was observed or detected. Moisture was more prevalent at some of the cap beams which were exposed to runoff water at deck expansion joints. Isolated concrete spalls (apparently not corrosion related) in the same vicinity of the cracks were occasionally found in cap beams. A total of 10 cores were extracted from the cap beams. As in the lower elevations, minor to no corrosion was observed on the reinforcement exposed by coring except for vestigial rust at small breaks in the polymer coating. Potentials measured at exposed ECR locations ranged from -70 mV_{CSE} > E >-490 mV_{CSE}. The concrete surface resistivity was 70-300 k Ω -cm with no clear difference between sound and cracked locations.

<u>Howard Frankland (HFB)</u>. Vertical cracks were frequently observed on the concrete footers; several large cracks were as wide as 1.0 mm. The trace of the crack observed on the top of the footers of the larger cracks was several feet deep. Cracks of this type had been documented in previous inspections and are likely due to differential curing in the bulk of the concrete. Subsequent coring revealed that cracking sometimes propagated past reinforcement depth (10.2 cm). Six pairs of on-crack/off-crack core samples were extracted at 9.7 to 47 cm AHT from 5 footers (in one of those footers, coring was done on two separate faces). Significant localized corrosion (morphology discussed later) was observed on 4 out of 7 on-crack bars extracted from 4 locations at 3 of the footers. In one instance where two bars were extracted from the same core, only the bar with deeper cover (11.6 cm, 1.6 cm deeper than the bar with lower cover)

showed corrosion. No physical indication of corrosion was observed at any of the matching sound concrete locations. Concrete delamination was not detected at either sound or cracked locations. Concrete clear cover to horizontal reinforcement was 10.9 to 11.7 cm, meeting nominal design requirements (10.2 cm). Half-cell potentials ranged from -200 to -690 mV_{CSE} measured at elevations from tidal zone to 0.9 m AHT. The more negative values were from locations where ECR corrosion took place. As noted above, caution is in order on generalizing the significance of this observation. Concrete surface resistivity was very high (M Ω -cm range) even in the tidal zone (Figure 2b). The large aggregate consisted of granite, and the volumetric porosity of the concrete was ~12%. The high measured surface concrete resistivity may be due in part to low permeability of the large aggregate.

The corrosion observed on the four epoxy coated reinforcement samples extracted from cracked locations from this bridge merits note. The crack plane was usually perpendicular to the rebar. Corrosion products were generally observed around locations with coating defects, especially near the intersection of the crack plane with the rebar. Upon removal of the coating (which was found to be fully disbonded) the underlying surface was relatively dry, with dark corrosion product regions.

<u>Lillian (PER).</u> Very little concrete deterioration was observed. Thin hairline cracks were observed occasionally. The main span footer had larger cracks (~0.3 mm) with indication of efflorescence, some of which had been repaired earlier on by epoxy-injection. Concrete cores were extracted sampling on and off a crack location at 1.1 m AHT. Also cores were extracted from a column with no concrete deterioration at 0.9 and 1.2 m AHT. Concrete cover ranged from 10.4 to 13 cm. Half-cell potentials ranged from -183 to -656 mV CSE at tidal to 1.5 m AHT. Concrete resistivity of the footer and column ranged from 113 to 275 k Ω -cm at 0.3-2 m AHT (Figure 2b). The large aggregate consisted of granite. The volumetric porosity of the concrete was ~14%. No corrosion or only vestigial signs of surface corrosion discoloration were observed on extracted epoxy coated reinforcement.

<u>Sunrise Blvd (ITB).</u> Like the Group 4 bridges described in the next section, the substructure concrete had a surface coating extending down to the high tide level. Only minor concrete cracking, <0.2 mm (Figure 4 (combined with ITA)) was observed. A concrete core was extracted on a vertical crack at ~80cm AHT and another core extracted on sound concrete offset 15 cm on center but at an elevation 20 cm above the crack concrete location to avoid other concrete cracks. The crack propagated deep into the concrete past reinforcement depth, Xc~11cm. The half-cell potential ranged from -61 to -218 mV_{CSE} at tidal zone to 1.2 m AHT. Concrete resistivity ranged from ~9 to 180 k Ω -cm from the tidal zone to 1.2 m AHT. The large aggregate consisted of limestone and river rock. The volumetric porosity of the concrete was ~20%. No observation of corrosion was observed on the surface of ECR. The ECR in cracked concrete was found to be

disbonded, but good adhesion was maintained for ECR in the sound concrete location.

3.2.c Group 4 Bridges (Moderate D_{CI}- Bridges)

<u>Lehman Causeway (IT2/3).</u> Very little concrete deterioration was observed, although pre-existing damage prior to application of surface texture paint may have been obscured. Concrete cracks (typically ~0.08mm wide) were thin yet penetrated deep into the concrete. The cracks were often traced across the width of the footers and penetrated past reinforcement depths (Xc~7-10 cm). Concrete cores were extracted sampling ECR on and off crack locations ~50-120 cm AHT. Half-cell potentials ranged from -41 to -627 mV_{CSE} at tidal level to 1.2m AHT; low concrete resistivity was observed, 4 to 12 k Ω -cm at 0.1-1.2 m AHT (Figure 2b). The large aggregate consisted of limestone and river rock. The volumetric porosity of the concrete was ~20%. Only vestigial surface corrosion under defects of the epoxy coating was observed.

<u>S. Andrews Ave (NWR).</u> Only the bascule rest pier was accessible. There, no concrete deterioration was externally observed. However, surface paint may have masked pre-existing cracks as one thin vertical crack (0.05mm) was revealed after removing some of the paint. One core was extracted to sample the underlying bar there (~1 m AHT) and another core on sound concrete at an elevation ~40cm below (no side by side cores were extracted at this location). The crack was found to be very shallow, propagating only a few centimeters below the concrete surface. The half-cell potential of exposed ECR in the cracked concrete location was -400mV_{CSE}. Concrete surface resistivity was not measured as it would have necessitated removing the paint coating and damaged surface finish in a tourist attraction bridge. The large aggregate consisted of limestone and river rock. The volumetric porosity of the concrete was ~20%. Only vestigial surface corrosion of steel under breaks in the epoxy coating was observed.

<u>Sunrise Blvd (ITA).</u> Like the other bridges in Group 4, the substructure concrete had a surface coating extending down to the high tide level. Only minor concrete cracking, <0.2 mm (Figure 4 (combined with ITB)) was observed. A core was extracted from concrete with no deterioration ~62 cm AHT. The half-cell potential ranged from -200 to -370 mV_{CSE} at tidal zone to 1 m AHT. Concrete resistivity ranged from ~4 to 10 k Ω -cm from the tidal zone to 1 m AHT. The large aggregate consisted of limestone and river rock and the volumetric porosity of the concrete was ~20%, similar to ITB. No observation of corrosion was observed on the surface of ECR and was found to be disbonded.

4 CHLORIDE PENETRATION AT SOUND AND CRACK LOCATIONS

4.1 Review and Discussion of Results from Earlier Investigations

A brief overview of results on chloride penetration through concrete from earlier FDOT investigations was given in Section 2. Preferential chloride penetration was observed for the majority of bridges in the 2002 study [1] at low elevations with continuous sea splash exposure and much less pronounced at higher elevations. Fast chloride penetration through cracks occurred even though some of those cracks were very narrow (<0.3 mm) and concrete had cement factors as high as 445 kg/m³ and with pozzolanic additions. In contrast, chloride penetration through deep cracks as wide as 0.3 mm in spin-cast cylinder piles was not observed (in the limited number of core extractions) even in low elevation locations with continuous sea-splash exposure [7,8]. Selected results from References [7.8] are reproduced in Figure 4d. It has been suggested that autogenous healing can occur in thin cracks in these piles. If autogenous healing precipitates prevent enhanced chloride penetration, early corrosion initiation would not be expected. Simplified corrosion damage forecasts suggested that very long service lives with minimum corrosion-related maintenance are possible [8].

4.2 Chloride Analysis from Current Investigation

4.2.a Methodology

Chloride ion penetration profiles were obtained for the field-extracted concrete cores. Powdered concrete samples obtained at various depths from the surface were analyzed for total (acid-soluble) chloride concentration; results are given in mg of Cl⁻ ion per gram of dry concrete. Diffusion coefficients, D, for the sound concrete profiles were estimated by least-error-fitting of the chloride content data to a solution to Fick's second law

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

that assumes constant D and constant surface chloride concentration, Cs. Chloride profile data are given in Appendix II.

Chloride transport at cracks does not follow simple Fickian diffusion [1] so a single descriptor such as a diffusion coefficient was not derived from the present work. Instead, the penetration profiles at cracked and adjacent sound concrete are presented by comparison. As it will be shown in the modeling section, a fictitious enhanced diffusion coefficient for on-crack transport was used there but only as a rough working approximation.

4.2.b Group 2 Bridges (High D_{CI}- Bridges)

<u>Vaca Cut (VA1/2) and Snake Creek (SNK).</u> The chloride penetration profiles are shown in Figure 4a. The D values for VA1/2 were very high $(1x10^{-7} \text{ to } 2.76x10^{-7} \text{ cm}^2/\text{s})$ and less but still indicative of high permeability for SNK (4.73x10⁻⁸ cm²/s). The average Cs value was similar to those measured in other Florida marine bridges (~7 mg/g, ~17 kg/m³) [1,9]. The chloride content of the cracked concrete samples were higher than in the sound concrete within the area of the drilled shaft susceptible to sea splash (<46 cm AHT) which gives indication of preferential chloride penetration through the cracks.

<u>William Marler (CHO).</u> Preferential chloride ion penetration through cracks in this bridge (similar to the other Category 2 bridges, Figure 4a) was overshadowed by fast bulk diffusion through the sound concrete at low elevation locations exposed to sea splash. Chloride concentrations at reinforcement depths (10.2 cm)) for sound and cracked concrete locations were larger than the commonly assumed 0.7 kg/m³ conservative chloride ion threshold value. The average chloride ion diffusivity for sound concrete from this bridge was lower than that measured in an earlier investigation [9] but it was within the range of calculated diffusivities from the same investigation. Nevertheless, a high value was still calculated (1.8x10⁻⁸ cm²/s).

4.2.c Group 3 Bridges (Low D_{CI}- Bridges)

<u>Sunshine Skyway(SSK).</u> Little to no corrosion was observed at any examined on-water locations despite the presence of pre-existing structural cracks as wide as ~0.3 mm and with well manifested enhanced chloride penetration there at SSK (Figure 4b). At elevations exposed to sea splash, chloride ion concentration at reinforcement depth for cracked concrete was ~2 kg/m³, close to or exceeding typically assumed threshold values [10].

The chloride concentration at the trestle caps (7-8 m AHT) cores was low and chloride penetration was very slow as manifested by the rapid decrease in concentration with depth (Figure 4b). Quantitative chloride diffusion parameters were not calculated due to the resulting uncertainty. Nevertheless, chloride surface concentration can be estimated to be in the order of only a few kg/m³, comparable to that obtained at similar elevation in an earlier survey [1]. The concentration profiles suggest enhanced chloride penetration through cracks, but the effect is uncertain as chloride concentrations were quite low (less than ~0.1 mg/g (~0.2 kg/m³)) which is typical of the native background chloride concentration) at depths greater than a few centimeters from the concrete surface).

<u>Howard Frankand (HFB).</u> Distinct preferential chloride penetration at cracks was observed in this bridge (Figure 4b), similar to that noted for the SSK, at elevations exposed to sea splash. Chloride ion concentration at the 10 cm

reinforcement depth for cracked concrete was greater than commonly assumed conservative threshold values 0.7 kg/m³). Much lower chloride levels were measured at bar depth in adjacent sound concrete, consistent with the low chloride bulk diffusivity \sim 7x10⁻⁹ cm²/s determined for this low permeability concrete.

<u>Lillian (PER).</u> Chloride diffusivity values were very low (~3x10⁻⁹ cm²/s) in agreement with earlier measurements [9]. Enhanced chloride penetration was not observed in the limited number of samples extracted (Figure 4b) but similar condition as that measured for the Sunshine Skyway may exist.

<u>Sunrise Blvd (ITB).</u> The chloride diffusivity was 3.8×10^{-9} cm²/s. Earlier measurements [1] were on average 5.1×10^{-9} cm²/s. As with the other Group 3 bridges, chloride penetration was slow. The surface chloride concentration was ~3 mg/g (~8 kg/m³). Enhanced chloride penetration through cracks was moderate and may have been somewhat mitigated by low surface chloride concentration reflecting presence of surface coating.

4.2.d Group 3 Bridges (Moderate D_{CI}- Bridges)

<u>Lehman Causeway (IT2/3).</u> The average chloride diffusivity was 3.9×10^{-8} cm²/s, consistent with that measured earlier [9]. The chloride surface concentration was however low, 0.7 mg/g (~2 kg/m³) (Figure 4c), approximately ten times lower than that typically encountered in the tidal region in similar environments in other bridges. As in Group 2 bridges, any enhanced transport through cracks was likely overshadowed by fast bulk diffusion.

<u>S. Andrews Ave (NWR).</u> The chloride diffusivity was $1.8 \times 10^{-8} \text{ cm}^2/\text{s}$, consistent with earlier measurements [9]. As with IT2/3, chloride penetration was fast but with a low concentration profile. The surface chloride concentration was only ~0.2 mg/g (~0.5 kg/m³). Any enhanced chloride penetration through cracks was likely overshadowed by fast bulk diffusion.

<u>Sunrise Blvd (ITA).</u> For this bridge only one core, on sound concrete, was available. Crack locations were not reached. The chloride diffusivity was 2.5×10^{-7} . Earlier measurements [1] were on average 2.6×10^{-8} cm²/s. As with the other Group 4 bridges, chloride diffusivity was high, but the entire profile consisted of low values, including the surface chloride concentration which was only ~0.2 mg/g (~0.5 kg/m³).

5 PREFERENTIAL CARBONATION AND MOISTURE PENETRATION

This section examines indications of preferential carbonation and moisture penetration at cracks.

5.1 Carbonation

5.1.a Methodology

The depth of concrete carbonation (from the external surface) was measured by lightly spraying 1% phenolphthalein in ethanol solution on freshly fractured concrete core samples. In concrete cores from cracked locations, the core was separated exposing the cracked surface, and phenolphthalein was sprayed on it. The depth (if any) from the external surface to which the crack surface remained colorless was recorded as the on-crack carbonation depth. In selected cores the halves corresponding to each side of the crack were further broken transversally to the crack, to determine how carbonation had progressed into the sides of the crack.

5.1.b Carbonation - Results

For the Group 2 Bridges there were only limited tests performed, but in those carbonation penetration from the external surface of the concrete at elevations exposed to sea splash was typically negligible (<1mm). The on-crack carbonation depth was also typically small, although in a few cases it reached up to about 1cm.,

For the Group 3 Bridges carbonation tests were conducted routinely with findings for the sea splash zone similar to those noted for Group 2. However, in a core sample extracted 65 cm AHT from ITB for which bulk penetration was negligible, the on-crack carbonation depth was significant, extending beyond reinforcement depth. In the high elevation SSK cores, there was evidence of deep on-crack carbonation up to reinforcement depth (Figure 5) reflecting dryer conditions. Carbonation in those cores was usually minor through bulk concrete (~1-2 mm), although in one instance it reached ~5 mm.

For the Group 4 Bridges, for which only the splash zone was sampled, carbonation depth on sound concrete was negligible. However, the on-crack carbonation depth was as large as ~4cm for IT2.

In all cases of significant on-crack carbonation depths, the carbonation penetration into the sides of the crack was found to be negligible (<1mm).

Extensive and deep carbonation at cracks would be a problem in that it could seriously lower locally the chloride initiation threshold [11]. The results suggest that carbonation was minor at most splash zone locations, except for the

aforementioned case in ITB where higher quality concrete and a surface coating may have contributed to creating a dryer local environment. Surface chloride concentration for that bridge was low, which may mitigate potential adverse effects of carbonation there. Continuing monitoring of those locations is nevertheless recommended.

5.2 Moisture Penetration

Preferential moisture penetration at cracks was examined indirectly by seeking evidence from concrete resistivity and electrochemical impedance measurements.

5.2.a Methodology

Resistivity measurements were conducted at both off and on crack locations following procedures indicated in Section 3 and in the companion Report for Project BD544-23 [4]. Electrochemical impedance spectroscopy (EIS) measurements were made for SSK epoxy coated rebar (ECR) samples embedded in concrete core samples. Additional details on EIS measurements are given in Reference [4]. A schematic of the test set-up is shown in Figure 6.

5.2.b Bulk Concrete Resistivity - Results

Bulk concrete resistivity results were given in Section 3 as indication of general concrete permeability. Side-by-side surface resistivity comparisons were also made on cracked concrete locations for SSK at high elevations and HFB where there were instances of significant concrete cracking. There was no distinct difference in concrete resistivity on cracked and sound locations (Figure 7 and 8). Overall moisture content in the bulk concrete material likely has a greater effect on surface resistivity than crack presence. Electrochemical impedance spectroscopy measurements, as described in the next section, are more sensitive to increased electrolyte conductivity through cracks.

5.2.c Electrochemical Impedance Spectroscopy - Results

As expected and shown in the cumulative curves in Figure 9, the measured solution resistance was smaller at the low elevation locations as well as in cracked samples, where moisture content and water penetration is expected to be higher.

Impedance curves generally showed high frequency (hf) loops with diameters that varied by orders of magnitude in side by side comparisons of EIS responses of epoxy coated rebar from cracked and non-cracked concrete from low elevations. As seen in Figure 10, the ECR from cracked locations generally had smaller hf loops than the ECR from the sound concrete. The hf loop diameter corresponds to the combined coating pore resistance, R_{po}. Cumulative

fraction curves of the pore resistance for all ECR measured with EIS (cracked/non-cracked concrete) are shown in Figure 11. Smaller R_{po} values generally observed at cracked concrete locations likely indicative of larger electrolyte conductance to the steel at crack locations.

From the above it may be concluded that while not manifested by surface resistivity measurements, substantial moisture penetration can take place at crack locations with consequent potential for locally accelerating corrosion.

6 LOCALIZED CORROSION PRODUCTS

6.1 Exploration of Solid Corrosion Product Developed in HFB

6.1.a Methodology.

The corrosion products developed in the crack locations of HFB (examples shown in Figure 12) were examined with metallographic cross sections by optical metallography, and by secondary and backscatter Scanning Electron Microscopy (SEM) supplemented by elemental analysis with Energy-dispersive X-Ray Spectroscopy (EDS).

6.1.b Optical Metallography - Results

ECR undercoating corrosion products were compact and dark, consistent with a low state of oxidation, suggestive of anaerobic conditions whereas other field situations and laboratory tests have shown reddish liquid corrosion products. Exploration with a sharp knife tip at the dark corrosion region pried out some of the products, revealing pit-like features as shown in Figure 12. Sectioning of the bar with a thin diamond blade and water-free lubrication was performed at a pitlike location, with subsequent metallographic mounting of the section. The cross section (Figure 13) revealed that corrosion had affected a wide region, having proceeded in relatively uniform fashion within the region to as much as ~1 mm deep. Except for some surface reddening, the corrosion products in that region were dense and dark-gray, suggesting a low oxidation state. The corrosion product-base metal interface was examined at higher magnification revealing upon etching a ferrite-pearlite grain structure that extended, with no indication of microstructural alteration, all the way up to the corrosion penetration front where it was being consumed. Representative features are shown in Figure 14. This observation nearly rules out ascribing the observed features to causes alternative to corrosion, for example the presence of an isolated defect in the form of trapped slag or mill scale during rolling, since such condition would have been manifested by microstructural changes near the interface. Metallographic sections of the other bar samples from crack locations revealed corrosion penetration of depth and morphology similar to the one shown, always near the region of intersection of the rebar with the crack. Exposure of the metallographic sections to laboratory air resulted in slow reddening of initially dark products, as seen in Figure 13, suggesting that the corrosion product was evolving toward a higher iron oxidation state. No indications of severe corrosion were externally observed or revealed metallographically, on ECR from the peer cores on the sound concrete location next to the cracks.

6.2.c Scanning Electron Microscopy/Elemental Analysis - Results

SEM micrographs (Figures 15 and 16) of the corrosion region showed a porous structure. Consistent with optical observations, the pearlitic grain

structure of the steel extended to the corrosion front with no indication of microstructural alteration, further confirming the identification of the observed material as a corrosion product rather than as mill scale having been trapped in a rolling defect or similar unlikely alternative. The larger pores in the corrosion product were in the order of 1 μ m. EDS results are exemplified for bulk steel, bulk corrosion product, within pore in corrosion product, and outside pore in corrosion product in Figure 16. The EDS analysis of the corrosion product shows a region rich in iron and oxygen (consistent for steel corrosion) and also significant presence of chlorine (3 to 11 wt%). The chlorine is very likely present as a chloride but the EDS spectrum by itself cannot indicate whether the chloride is as free ions or bound in iron-chloride complexes. However, space-resolved EDS elemental analysis did not show any significant difference in chloride presence between points in the bulk of the corrosion product and within a pore region suggesting that the chloride is an integral component of the corrosion product. Trace amounts of other elements are attributed to steel alloy elements and contamination from metallographic preparation.

6.2 Solid Corrosion Product Formation in De-aerated Conditions

Corrosion was accelerated for laboratory epoxy coated steel samples with introduced defects placed in simulated concrete pore water solution in aerated and deaerated conditions to compare corrosion development and morphology with that observed in HFB field samples, and to determine whether enhanced oxygen access is needed for the in-crevice corrosion product development.

6.2.a Methodology

Sample ECR bars (1.6 cm diameter) that were free of mechanicallyinduced coating defects were cut (lengths 23 cm) and intentional coating defects were introduced by drilling to the steel substrate with a 1.6 mm diameter drill bit to a nominal depth from the coating outer surface to the tip of the drill cavity of 1 mm. Defect locations were located in bar areas between adjacent deformation ribs. Total exposed steel area was ~0.45cm². The two cut ends of the bar samples were coated with epoxy patch compounds. The bottom 1.9 cm of the bar was set in metallographic epoxy to cap the bottom end of the bar to prevent corrosion there, and to serve as a base stand for the sample. A stainless steel set screw was tapped on to the top end for electrical connection.

The samples were tested in simulated concrete pore solution (SPS) environments with chloride ions; solution constituents and pH are listed in Table 3. The bar samples were partially immersed (19 cm length (~95 cm² surface area) in contact with liquid) in each solution with the set screw ~2 cm above water. Test chambers were sealed to avoid solution carbonation. In the aerated cell, air scrubbed of carbon dioxide through a filter cell with saturated calcium hydroxide was bubbled into the test solution at a rate ~0.01cm³/s (1 bubble of diameter 0.3 cm per second). In the deaerated cell, nitrogen gas was

continuously bubbled in the cell at a similar flow rate as the aerated cell. In both cells, the gas outlet was fed into another filter cell. Solution temperature was 22 ± 2 °C.

The samples were potentiostatically polarized anodically at $\sim 100 \text{mV}_{\text{SCE}}$ to accelerate corrosion for 1 week after which the samples were removed and corrosion development was examined.

6.2.b De-aeration/ Aeration Cells - Results

Anodic current for the aerated and deaerated cells were similar throughout the experiment. The effect of concentration polarization where lower corrosion would occur in the deaerated condition was not manifested due to the large applied anodic polarization at +100mV_{SCE}. The cumulative charge for the aerated and de-aerated cells were both ~900 coulombs.

Visual differences were seen between the aerated and deaerated samples throughout the duration of the experiment (Figure 17). External corrosion in aerated cells consisted of reddish tubercle formation consistent with a high oxidation state iron oxide. Greenish tubercle formation was observed in deaerated cells consistent with lack of oxygen. Undercoating corrosion however for both conditions were similar, where a dark solid corrosion product formed at the defect site. Undercoating pH as determined by applying pH paper to residual moisture there shortly after extraction of the samples from the test cells was ~5-6 in both cases. It is recognized that such measurement is subject to considerable artifacts due to mixing of the in-crevice solution with that on the exterior and due to the time internal between the test and interruption of the polarization. Nevertheless, the result is indicative of a radically different environment inside the crevice compared with the highly alkaline conditions existing outside.

Even though oxygen availability was high external to the bar surface in the aerated experiments, transport to sites immediate to the active region under the coating crevice was likely very limited, creating nearly anaerobic conditions there. Oxygen reduction would likely continue at neighboring defect sites under disbonded polymer coating. Thus, the conditions inside the crevice in the aerated case appear to have been no greatly different than those encountered in the deaerated cells. In the active crevice site in either case, the low pH environment caused by autocatalytic process of chloride accumulation and subsequent concentration of H⁺ ions would lead to increased corrosion by well recognized mechanisms [12].

6.3 Corrosion product identification and remaining issues

A solid corrosion product consistent with the SEM-EDS compositional observations for HFB and also capable of creating a significant in-crevice decrease in pH through hydrolysis of Fe ions [12] is chloride-substituted

Akaganeite, which has the nominal composition $Fe(OH)_{2.7} Cl_{0.3}$ [13,14]. Such product would account for the formation of an expansive in-crevice solid and the ability to sustain crevice corrosion by promoting substantial local acidification [12]. The dark expansive solid corrosion formation in both the aerated and deaerated the laboratory experiments is similar in appearance to that observed in HFB and is expected to have similar composition, a hypothesis that will be tested in continuation research work which also will include attempts to further identify the compounds by powder X-Ray diffraction.

A factor casting some doubt on identifying the corrosion products as Akaganeite is that it involves a ferric state of oxidation which is usually associated with red rather than black coloring. Indeed, the in-crevice products of both HFB and the laboratory quickly turn superficially red upon laboratory air exposure. However, color by itself is not a certain indicator of oxidation state and the observed change may be related to the development of superficial species of different relevance to be determined in follow up work.

7 CORROSION FORECAST MODELING

At present there is only very limited data (that from HFB) on the extent of damage that may result from corrosion at preexisting cracks. The following is an initial formulation that may serve as a starting point pending the development of additional field evidence.

7.1 Basic statements

The following definitions, assumptions and simplifications apply:

1. Per extension of the findings in Reference [1] cracks exist at the rate of Nc cracks per unit length of substructure perimeter on water.

2. The average crack length is h'

3. The portion of the substructure subject to corrosion has a height h and a total perimeter length W, so its total area is A_T =h W

4. The portion of rebar subject to corrosion where the crack intersects the rebar has a length Lc $\sim \Phi$ were Φ is the rebar diameter.

5. Rebar diameter Φ is significantly smaller than the average rebar cover Xc'

6. The crack emanating from the affected rebar results in a spall corresponding to a crack front fanning out at a 90 degree angle from the corroded zone. Therefore the resulting spall has a width Ws~ 2 Xc'

7. Each crack can affect multiple rebar intersections resulting a potential spall of length h' and width Ws.

8. The total area potentially affected by spalls at cracks and the rest of the substructure area subject to corrosion develop corrosion separately (e.g mutual macrocell effects and mutual spall mechanical interference are ignored for simplicity)

9. Each bent of the bridge has a total external area Af subject to corrosion

10. A damaged area equal to Ae counts as one spall.

11. Chloride transport into the crack is assumed on first approximation to occur by simple diffusion, with an apparent diffusivity $D_{capp} = \beta D_{app}$, where D_{app} is the apparent diffusivity in the concrete bulk and β is a multiplier >1.

12. The corrosion initiation threshold for rebar intersecting the crack is equal to the value C_T prevalent at the bulk.

13. Once corrosion starts at the rebar/crack intersection the local corrosion rate is macrocell-enhanced with respect to that in the bulk by a factor $\gamma > 1$.

14. Per extension of the arguments made in Reference [3] the amount of metal penetration by corrosion needed to cause a spall by corrosion where the rebar intersects the crack is greater than that prevalent at the bulk by a factor $\delta = (1+Lc/Xc')^2$. It is noted that Lc is assumed to be equal to Φ per an earlier assumption.

15. From the above, it may be said on first approximation that, of the total substructure area subject to corrosion A_T , there is a portion αA_T that is subject to crack-induced corrosion and a portion (1- α) A_T that corrodes regularly as in initially sound concrete.

16. The fraction of the surface experiencing regular corrosion that spalls by time t shall be called fr(t); the fraction of the surface potentially affected by crack-induced corrosion that spalls by time t shall be called fc(t)

7.2 Implementation

Based on the above assumptions and definitions the value of α is given by

$$\alpha$$
 = 2 Xc' Nc h' W / (h W) = 2 Xc' Nc h'/h 2)

The function fr(t) for the part corroding regularly can be calculated using the same treatment introduced earlier for sound concrete. For the crack portion function fc(t) can be calculated in the same manner but using γ Dapp instead of Dapp, and ε k' instead of each k' where $\varepsilon = \delta/\gamma$ because the propagation time would be shortened by the increase γ in the local corrosion rate, and extended by the increase δ in the amount of corrosion needed for local cracking. All the other parameters remain the same as for the sound concrete model

The number of spalls per bent by time t is therefore given by

$$N(t) = (Af/Ae) \left(\alpha fc(t) + (1-\alpha) fr(t) \right)$$
3)

7.3 Cases examined

Because of the paucity of actual data indicated above, only illustrative calculations are presented.

The conditions chosen correspond to a Group 3 bridge with 20 m² surface area (~200 sq. ft.) bents, and declaring a spall when the spall area is 0.32 m^2 (~3 sq.ft). Numbers smaller than one spall ber bent may be interpreted as indicating the appearance of only one spall in a correspondingly large number of bents.

Rebar size and cover are assumed Φ =1 in (2.54 cm) and Xc'=4 in (10.2 cm), so δ = 25. As a variation, a condition will be considered in which the crack runs parallel to the rebar length instead of crosswise. In such case the length of the corroded zone can be Lc>>Xc', so δ is simply =1.

The macrocell acceleration factor is unknown but experiments and calculations [2] suggest that there may be an order-of-magnitude effect so $\gamma = 10$ will be assumed.

Acceleration of chloride transport into the crack will be also assumed to be an order-of magnitude effect but possibly greater for concrete with very low D_{app} in the bulk, so cases with β =10 and β =100 will be adopted.

Visual observations suggest most cracks are in the 3 to 6 ft (0.9 to 1.8 m) length range which is in the order of half of the elevation range normally associated with potential for sever corrosion, so h'=0.5 h will be assumed. The value of Nc observed in Ref [1] ranged from 0.01 m⁻¹ to 0.19 m⁻¹ so calculations will be conducted for those two extreme values, as well as for Nc=0 as a sound concrete baseline.

The above conditions and variations are named as follows:

No crack:	Baseline, sound concrete.
C:	Crack perpendicular to rebar, δ =25
L:	Crack along rebar, δ =1
10X:	Transport along crack 10 times faster than in bulk, β =10
100X:	Transport along crack 100 times faster than in bulk, β =100
Nc:	0.01 m ⁻¹ . Low incidence of preexisting cracking
Nc:	0.19 m ⁻¹ . High incidence of preexisting cracking

All other parameters are as in case Group 3 described in Table 5 of the report for the companion Project BD544-23.

7.4 Results

Results keyed to the above nomenclature are presented in Figures 18 and 19 for a 100 years time base. As can be seen there, the model projects no dramatic effect on the damage function for any of the variations concerning the low incidence of cracking case. Substantial effects are projected however for some of the variations the high incidence case (0.19 m⁻¹ corresponds to one crack each ~15 ft of linear perimeter in water). The greatest effect is equivalent

to a 300% increase in the incidence of spalling for service ages from about 15 years to 50 years compared to the sound concrete case, for lengthwise cracks and a 100X multiplier in chloride transport at the cracks compared with sound concrete. The smallest projected effect is almost immaterial (maximum ~ 20% increase in damage) and corresponds to cracks crosswise to the rebar length, and only a 10X increase in chloride transport in the crack.

The projections indicate that, as expected, relatively isolated cracking should only create topical concrete damage with reduced maintenance requirements. However, even though assuming that the incidence of damage is limited to a small region around the crack, if the crack orientation with respect to the rebar were adverse and chloride transport were greatly enhanced (as it could be expected in relatively wide cracks), corrosion damage from localized concrete deficiencies could significantly increase maintenance costs. These findings underscore the need for continuing monitoring of locally deficient concrete locations in otherwise high quality concrete structures exposed to aggressive environments. Judicious application of the predictive model developed in this project may aid in exploring the cost effectiveness of alternative corrosion protection methods, such as corrosion resistant rebar to prevent local damage for a given extent of existing or anticipated concrete deficiency.

8 CONCLUSIONS

- 1. The present investigation confirmed prior noted trends of extensive preferential chloride intrusion at preexisting cracks at on-water locations in a majority of cases of substructure members in Florida bridges built with low permeability conventional concrete (apparent chloride diffusivity in sound concrete Dapp well below 10⁻⁸ cm²/sec). An exception noted from previous research was cylinder piles in bridges several decades old.
- 2. Preferential chloride intrusion was not noted in bridges with relatively high permeability concrete (Dapp typically above 10⁻⁸ cm²/sec) as in those cases bulk diffusion was rapid enough to mask any faster transport through cracks.
- 3. Corrosion clearly associated with the presence of preexisting cracks was noted in only one structure, the Howard Frankland Bridge, 15 years old at the time of inspection and built with Epoxy Coated Rebar (ECR). Corrosion there was severe at multiple locations were preexisting cracks intersected rebar. Local corrosion penetration at the affected locations approached 1 mm. It is recommended that those and similar locations be carefully monitored in the future.
- 4. Examination of corrosion products showed them to grow underneath the epoxy coating and containing chloride ions. The composition of the products was consistent with the makeup of chloride-substituted Akaganeite which can support significant local acidification in a crevice environment.
- 5. Laboratory experiments showed that local expansive in-crevice corrosion products formed similarly in epoxy-coated rebar under anodic polarization in simulated concrete pore solution with chlorides regardless of the presence or absence of oxygen outside the crevice. The result suggests that increased oxygen access at a preexisting crack is not necessary for the development of expansive corrosion products.
- 6. There was no clear differentiation in surface concrete resistivity on cracked and sound locations in the field. Overall moisture content in the bulk concrete material likely has a greater effect on surface resistivity than crack presence. However, electrochemical impedance measurements indicted that substantial moisture penetration can take place at crack locations with consequent potential for locally accelerated corrosion.
- 7. For bridges with low chloride diffusivity carbonation penetration at elevations exposed to seawater splash was typically negligible in both sound and cracked concrete, except for one case in concrete with an external coating where on-crack carbonation extended beyond

reinforcement depth. Deep on-crack carbonation was noted at high elevations in pile caps where concrete was drier. Although chloride penetration was small there, continuing monitoring of those locations is recommended. Except for one isolated case, carbonation penetration was also typically negligible both on and off-crack locations of the splash zone of high to medium chloride diffusivity bridges.

- 8. A model for projecting impact of preexisting cracking on spall damage was developed based on working assumptions. While recognizing that validation of the model will need to await development of data over a longer time period, the projections indicate that, as expected, relatively isolated cracking should only create topical concrete damage with reduced maintenance requirements.
- 9. However, model projections indicated also that even though assuming that the incidence of damage is limited to a small region around the crack, if the crack orientation with respect to the rebar were adverse and chloride transport were greatly enhanced (as it could be expected in relatively wide cracks), corrosion damage from localized concrete deficiencies could significantly increase maintenance costs. The added projected damage was for certain service times in the order of 300% greater than for some concrete in some of the scenarios addressed.
- 10. The above findings underscore the need for continuing monitoring of locally deficient concrete locations in otherwise high quality concrete structures exposed to aggressive environments. Judicious application of the predictive model developed in this project may aid in exploring the cost effectiveness of alternative corrosion protection methods, such as corrosion resistant rebar to prevent local damage for a given extent of existing or anticipated concrete deficiency.

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	ECR cover (cm)			7.6			15.3	15.2	0.01	15.3	7.6	10.2	10.2	10.2	12.0	11	10.9	c	Я. Г
	Widest crack width (mm)	× × × × ×		> , †	ac C	0.40	0.28	>1 [†]	0.64	0.25	ž	0.18	hairline	0.18	30.0	CZ.U			
	Fly ash no					ОЦ			20% Type F	Yes	35% Type C	Yes		о С					
	Cement factor (kg/m ³)	(kg/m ³) 388				388			445	I	388	I	I	I		I			
	p (kΩ-cm)	5-24	I	I	I	I	0.4-12	A EO	4-30	4-50	16-128	150	113-275	high MΩ-cm	9-180	I	4-10	4-12	
	Average D (cm ² /s)	2x10 ⁻⁷	2x10 ⁻⁷	~10 ⁻⁷	~10 ⁻⁷	2.9x10 ⁻⁷	~10 ⁻⁷	2.6x10 ⁻⁷		9.0x10 ⁻⁸	1.8x10 ⁻⁸	1.1x10 ⁻⁹	3.1x10 ⁻⁹	7.3x10 ⁻⁹	3.8x10 ⁻⁹	1.8X10 ⁻⁸	2.5x10 ⁻⁷	8-07.00 c	0.3X IU
	Year Built	1982	1980	1981	1982	1982	1981	1982	1982	1981	1979	1986	1981	1991	1989	1981	1989	1983	1983
	Bridge Name	900026	900117	900095	900098	900094	900097	900124	900126	900077	570082	150189	480140	150210	860467	860319	860466	870607	870606
ated		IML	NIL	INK	CH5	LOK	CH2	VA1	VA2	SNK	СНО	SSK	PER	HFB	ITB	NWR	ITA	IT2	IT3
vle 1. Bridges Investig:		Seven Mile	Niles Channel	Indian Key	Channel 5	Long Key	Channel 2	Vaca Cutt SB	Vaca Cut NB	Snake Creek	William Marler (over Choctawachee Bay)	Sunshine Skyway	Lillian (over Perdido Bay)	Howard Frankland	Sunrise Blvd. (over Intracoastal Waterway)	S. Andrews Ave. (over New River)	Sunrise Blvd. (over Intracoastal Waterway)	Lehman Cswv.	(over Intracoastal Waterway)
Tab			Group 1					Group 2				Group 3			Group 4				

10 TABLES AND FIGURES`

[†] Spalled Concrete.
‡ The side-by-side bridges VA1 and VA2 are treated as one bridge in the following unless indicated otherwise. This follows a recent FDOT change in numbering, to designate bridges 900124 (VA1, southbound) and 900126 (VA2, northbound) under the single number 900126.

Clear cover (cm)	Clear cover (cm) 9.9		8.9	9.4	8.3	ı	8.2	11	3.1	2.3	2.9	8.1
Widest crack width (mm)	0.08	I	0.33	0.15	0.10	0.3	0.4	0.15	0.3	0.3	0.2	0.05
Fly ash	20% Type F	20% Type F	ou	20% Type F	20% Type F	ou	ou	ı	ou	ou	ou	yes
Cement factor (kg/m ³) 445		434	06£	434/390	434	068	362	390	390-474		~200	
ր (kΩ-cm)	137	-	52	99	1 5	10	36	115	100- 200	70- 100	-	50- 200
Average D (cm ^{2/} s)	4x10 ⁻⁹	6.5x10 ⁻⁹	7.5x10 ⁻⁹	8.5x10 ⁻⁹	7.1x10 ⁻⁹	2.7x10 ⁻⁸	3.1x10 ⁻⁸	1x10 ⁻⁸	1x10 ⁻⁹	1.9x10 ⁻⁹	2.5x10 ⁻⁹	1.8x10 ⁻⁹
Year Built	1992	1995	1989	1993	1990	1990	1962/86	1986	1960	1960	1964	2003
	150138	870772	720518	154259	150202	790152	150052	170158	460012	480035	570034	490003
Name	ccc	MAC	DPE	BSB	SHB	NSB	BCB	NPB	-	-		SGI
Bridge	Courtney Campbell Cswy	McArthur Cswy	Dames Point	Bay Side	Safety Harbor	New Smyrna	Boca Ciega Bay	New Pass	Hathaway	Pensacola Bay	Brooks	St. George Island
	2002 Investigation [1]								2005	Investi	gati	on [7]

Table 2. Bridges from Earlier Investigations

Table 3 Solution constituents (g) per 1 L H₂O

	NaOH	KOH	Ca(OH) ₂ †	NaCl	pН				
SPS	3.7	10.5	2.1	35	13				
t Not fully dissolved in solution									

† Not fully dissolved in solution.



Figure 1. Widest crack width per substructure unit (e.g. per column, footer, etc.)



Figure 2. Concrete surface resistivity evaluation as function of elevation. Refer to Appendix II for numeric data listing and detailed location



Figure 3. Concrete porosity. Refer to Appendix I for listing of cores used for porosity determinations.



Figure 4. Chloride penetration profiles. Solid symbols: sound concrete. Open symbols: cracked concrete. Thick line, arrowed: high elevation trestle cap SSK locations.
 Horizontal line: conservative chloride threshold value ~0.3 mg/g (C_T~0.7kg/m³). Cylinder Pile Bridge data from references [7,8]. Numeric chloride profile data and core identification are given in Appendix I.



Figure 5. On crack carbonation testing: SSK high elevation trestle cap concrete core. A. On site crack location (SSK140E1). B. Transverse cross section from crack surface (SSK33W1). C. On crack surface (SSK33W1).



Figure 6. Field-Extracted ECR Concrete Core EIS Test Set-up.



Figure 7. Concrete surface resistivity comparison on and off crack. SSK High Elevation Trestle Caps. Solid line: Sound concrete. Dotted line: Cracked concrete. Refer to Appendix II for numeric data listing and detailed location identification



Figure 8. Concrete surface resistivity comparison on and off crack. HFB Footers. Solid line: Sound concrete. Dotted line: Cracked concrete. Refer to Appendix II for numeric data listing and detailed location identification



Figure 10. ECR Impedance manifestation of presence of a crack in the core (low elevation SSK samples). A,B,C Sound concrete core; A', B', C' Companion on-crack core; crack widths were 0.13, 0.25 and 0.23 mm respectively. (A, A':118W4,118W3. B, B': 118W2, 118W1. C,C': 117E3, 117E1).



Figure 11. Pore resistance in ECR from SSK sound and crack concrete locations. – sound concrete, -- crack concrete.



Figure 12. ECR corrosion at crack location (HFB 59E1) A. Corrosion products on bar surface. The line represents the location of the crack intersection. B. Appearance after partially removing the coating and exploring into the corrosion products.



Figure 13. Cross section of ECR bar showing severe corrosion at a crack location of HFB 59E1. Left: Entire Bar cross-section. Right: Close up of corrosion penetration.



Figure 14. Corrosion product - base metal interface showing progression of corrosion into the microstructure of the rebar steel (HFB 59W1).



Figure 15. SEM Image of corrosion product (HFB59E1).





Figure 17. External and undercoating corrosion development in De-aerated (left) and Aerated (right) Environments.



Figure 18. Model projections for the high cracking incidence cases.



Figure 19. Model projections for the low cracking incidence cases.

APPENDIX I

Core Sampling Locations

Each figure in the following pages shows schematic representations of substructure elements of each bridge investigated, existing cracks if any noted, position of extracted cores and their elevation with respect to the high tide level. Straight lines within core outline indicate rebar intersected by core.

Table A1-1 Extracted Concrete Cores.

Group	Bridge Name	Pier/Element	Core #	Designation	Disposition
	CH2	-	-	-	-
			1	VA2A1	-
		W2A	2	VA2A2	R,C
	VA1		3	VA2A3	R,C,P
			1	VA3B1	R,C,P
		VVJD	2	VA3B2	R,C,P
	VA2	-	-	-	-
			1	SNK8A1	R,C
		ο Λ	2	SNK8A2	C,P
2	SNK	οA	3	SNK8A3	R
	SINK		4	SNK8A4	R,C
		OP	1	SNK9B1	R,C
		90	2	SNK9B2	R,C,P
			1	CHO29S1	R,P
			2	CHO29W2	R,C,P
	CHO	20	3	CHO29W3	R,C
	СПО	29	4	CHO29N4	R,C
			5	CHO29E5	R
			6	CHO29E6	R,C
			1	SSK18W1	R
		10	2	SSK18W2	R
		10	3	SSK18W3	R
			4	SSK18W4	R
			1	SSK117E1	R,C,Z
			2	SSK117E2	R,C,Z
		117	3	SSK117E3	R,C,Z
		117	4	SSK117E4	R,C,Z
			5	SSK117E5	R
			6	SSK117E6	R,C,Z
			1	SSK118W1	R,C,Z
		110	2	SSK118W2	R,C,P,Z
3	SOK	110	3	SSK118W3	R,C,Z
5	001		4	SSK118W4	R,C,Z
		130	1	SSK130W1	R
		150	2	SSK130W2	R
		33	1	SSK33W1	R,Z
			2	SSK33W2	R,Z
		84	1	SSK84W1	R,Z
		04	2	SSK84W2	R,Z
		140	1	SSK140E1	R,Z
		140	2	SSK140E2	R,Z
		143	1	SSK143E1	R,Z
		טדו	2	SSK143E2	R,P
		167	1	SSK167E1	R,C
		107	2	SSK167E2	R,C

Group	Bridge Name	Pier/Element	Core #	Designation	Disposition
		E0\\/	1	HFB59W1	R,P
		5900	2	HFB59W2	R
		505	1	HFB59E1	R,C
		59E	2	HFB59E2	R
		60N	1	HFB60N1	R,C
		0011	2	HFB60N2	R,C
	HFB	539	1	HFB53S1	R,C
		555	2	HFB53S2	R,C
3		559	1	HFB55S1	
(cont)		555	2	HFB55S2	R,P
(cont.)		52N	1	HFB52N1	R,C
		JZIN	2	HFB52N2	R
		52W	1	HFB52W1	R
		26	1	PER26W1	R,C,P
	PER	20	2	PER26W2	R,C
	FLIX	36	1	PER36N1	R,C,P
		50	2	PER36N2	R,C
	ITR	В	1	SRIB1	R,C
	ПD	D	2	SRIB2	R,C,P
	172	В	1	LEHB1	R,C,P
	112	D	2	LEHB2	R,C,P
	IT3	C	1	LEHC1	R,C
4	115	C	2	LEHC2	R,C
	ITA	D	1	SRID1	R,C,P
		Δ	1	NWR1	R,C
		~	2	NWR2	C.P

Table A1-1 Extracted Concrete Cores (continued).

Disposition Key.

R: Rebar Sample Analysis C: Chloride Profile (Total Chloride) P: Concrete Porosity

Z: EIS







Figure A2. Snake Creek (SNK). Drilled Shafts.



Figure A3. William Marler Bridge (CHO). Footer.

Concrete Cracking max. width 0.6mm . Severe Corrosion: Core 5 max width 0.6 mm ▲ Core 4 severe Pier 29 South Footer 18 cm delaminated concrete Ħ Core 5 severe max width 0.3 mm Core 6 ა 9 cm 55 cm

52

Figure A4. William Marler Bridge (CHO). Footer.



Figure A5. Sunshine Skyway Bridge (SSK). Trestle.



Figure A6. Sunshine Skyway Bridge (SSK). Elliptical Column.

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Figure A7. Sunshine Skyway Bridge (SSK). Elliptical Column.



Pier 130 Elliptical Column, West Column, South Face

Figure A8. Sunshine Skyway Bridge (SSK). Elliptical Column.



Figure A9. Sunshine Skyway Bridge (SSK). Trestle Caps.











Figure A12. Howard Frankland Bridge (HFB). Footers.



Figure A13. Lillian Bridge (PER). Main Span Footer.



Figure A14. Lillian Bridge (PER). Footer.





Figure A15.Sunrise Blvd. (ITB). Bascule Pier.





Figure A16. Lehman Causeway (IT2). Footer.
GROUP 4 BRIDGES



Figure A17. Lehman Causeway (IT3). Footer

GROUP 4 BRIDGES



Figure A18. Sunrise Blvd. (ITA). Bascule Pier.

GROUP 4 BRIDGES





Figure A19. S. Andrews Ave. (NWR) Footer.

APPENDIX II

Concrete Core Chloride Analysis and Concrete Surface Resistivity Field Data

			1	1	1		1	1	1		1	1						1										
	D (cm ² /c)	n (dii 78)	cracked	2.8x10 ⁻⁷	1.0x10 ⁻⁷	1.4x10 ⁻⁷	4.7x10 ⁻⁸	6.9x10 ⁻⁸	1.1×10 ⁻⁷	cracked	1.4x10 ⁻⁷	cracked	2.0x10 ⁻⁸	cracked	1.6x10 ⁻⁸	cracked	cracked	1.1x10 ⁻⁹	cracked		cracked	-	cracked	-	cracked		cracked	•
	Cs	(mg/g)		9.0	1.7	6.7	5.6	2.9	4.1		6.7		8.7		11.1			16.0			-	-	-	-				1
		G														0.5												
	(g/g	ш											0.2			0.3	1.2											
	nt (mç	ш		6.0	0.9	3.1		1.0	2.0		2.7		0.5			0.8	1.0										0.08	0.02
	Conte	Δ	5.8	6.2	1.0	3.8	2.5	1.3	2.3	4.1	3.7	5.5	0.4		1.3	1.6	1.1	0.06	0.1				0.2		0.1	0.01	0.09	ı
	oride (ပ	6.3	6.9	1.1	4.8	3.2	2.2	3.0	5.2	4.8	8.8	1.2	4.5	3.9	2.1	1.6	0.03	0.03				2.1		0.2	0.02	0.06	0.02
	Chl	в	6.9	8.1	1.5	6.1	4.2	2.5	3.7	6.4	4.8	8.2	3.0	7.6	6.1	3.4	3.1	0.1	0.3			0.03	3.5	0.3	0.4	0.06	0.13	0.03
		۲	6.7	8.7	1.5	5.9	4.9	2.2	3.6	1.7	6.5	9.6	7.6	9.5	8.7	9.9	2.1	4.8	7.5	0.03	2.5	0.04	3.0	6.1	2.1	0.17	0.45	0.30
		ი														13.1												
		ш											16.2			11.4												
	(ר	ш		9.5	8.2	13.2		9.5	9.7		12.1		13.7			7.8	11.5										7	7
	th (cn	Ω	6.5	8.2	6.5	9.8	7.4	7.5	7.0	8.9	9.1	8.1	11.1		7.6	6.4	8.2	10.1	9.5				10.3		5.5	5.5	5.5	5.5
	Dep	с	4.9	4.5	4.8	6.2	4.2	4.3	4.4	6.4	5.7	6.0	8.6	4.8	5.1	4.4	7.8	6.4	7				6.1		4	4	4	4
SIS.		в	3.3	3.0	4.1	3.1	2.9	3.0	2.7	3.8	3.0	3.5	6.0	3.5	3.8	3.8	5.3	3.8	4.1			7.4	3.8	3.8	2.5	2.5	2.5	2.5
Analy		A	1.9	1.8	1.7	1.7	1.7	1.8	1.4	1.3	1.8	1.0	1.0	1.0	1.3	1.3	5.1	1.3	1.3	8.9	7.2	6.9	1.3	1.3	1	1	1	1
	Elevation	AHT (m)	0.5	0.5	1.7	0.3	0.2	1.1	0.7	0.2	0.2	0.2	0.3	0.2	0.2	0.5	0.6	0.5	1.3	0.8	0.9	0.9	0.3	0.3	7.4	7.5	7.2	7.3
Concrete	Sample	D	2A2	2A3	3B1	3B2	8A1	8A2	8A4	9B1	9B2	29N4	29E6	29W2	29W3	117E1	117E2	117E3	117E4	117E6	118W1	118W2	118W3	118W4	140E1	140E2	167E1	167E2
e Az-I.	Bridge	Name		0,111	7/14/				SNK													SSK						_
ap								Gr	ou	o 2											Gr	ou	o 3					

nalysis
Chloride A
Concrete (
A2-1. (
Table

-																					
	D (cm ² /c)		cracked	5.0x10 ⁻⁹	1.4x10 ⁻⁹		cracked	cracked	ı	cracked	cracked	7.3x10 ⁻⁹	cracked	3.8x10 ⁻⁹		1.8x10 ⁻⁸	2.5x10 ⁻⁷	cracked	2.9x10 ⁻⁸	cracked	5.x10 ⁻⁸
	Cs	(mg/g)	ı	0.2	2.4	ı		-	-		-	6.0	-	3.2	-	0.2	0.2	-	9.0		0.8
		ი																			
	g/g)	Ц											0.10	0.02	0.06	0.04	0.09			0.39	0.35
	ent (mç	Ш											0.10	0.02	0.05	0.04	0.13	0.18	0.21	0.50	0.39
	Conte	D		0.1	0.02		1.7	1.6	0.1	1.0	0.8	0.2	0.26	0.02	0.08	0.07	0.18	0.20	0.27	0.64	0.40
	loride	С	0.1	0.1	0.3		2.4	1.4	0.08	2.0	1.5	0.2	1.76	0.06	0.07	0.08	0.19	0.20	0.37	1.16	0.43
	Ch	В	0.2	0.2	1.3	0.2	2.9	2.6	0.07	1.8	2.6	0.2	2.62	0.97	0.12	0.12	0.20	0.34	0.42	1.04	0.59
		A	0.40	0.2	6.3	2.3	5.9	3.9	1.7	2.2	4.4	0.6	2.94	2.04	0.11	0.13	0.13	0.33	0.57	2.0	0.82
		Ċ																			
		ш											8.5	8.5	8.5	8.5	8.5			8.5	8.5
	(m	ш											7	7	7	7	7	7	7	7	7
	pth (c	D		4.8	3.8		8.9	8.9	8.9	8.9	8.9	8.9	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5
	De	С	3.5	2.9	2.5		6.4	6.4	6.4	6.4	6.4	6.4	4	4	4	4	4	4	4	4	4
		В	2.2	1.6	1.0	2.2	3.8	3.8	3.8	3.8	3.8	3.8	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
		A	1.0	1.0	0.4	1.0	1.3	1.3	1.3	1.3	1.3	1.3	١	1	١	١	١	١	١	1	1
	Elevation	AHT (m)	1.1	1.1	0.9	1.5	0.5	0.1	0.1	0.5	0.5	0.5	0.5	0.7	0.9	0.6	0.6	1.2	1.2	0.4	0.4
	Sample	Ω	26W1	26W2	36N1	36N2	52N1	53S1	53S2	59E1	60N1	60N2	SRIB1	SRIB2	NWR1	NWR2	SRID1	LEHB1	LEHB2	LEHC1	LEHC2
	Bridge	Name											ΔĽ	∩ =			ITA		170/172	C11/211	
						Gı	rou	рЗ	3 (C	con	t.)						Gr	oul	p 4		

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ge			-	Elevat	ion AF	HT (m)				Su	Irface F	Resistiv	ity (kΩ-	cm)	
e	LUCATION	A	В	C	D	ш	ш	Ċ	A	В	ပ	Δ	Ш	ш	ი
Ś	2A	0.30	0.46	0.61	0.91	1.22			4.3	9.9	18	ı	33.1		
2	3B	0.30	0.61	0.91	1.22	1.52	1.68	1.82	9.5	20.8	21.6	19.2	45.8	'	25.9
	3A	1.0	2.0	3.0	4.0	5.0			12.4	25.1	31.9	21.9	27.7		
	3A	1.0	2.0	3.0	4.0	5.0			18.5	25.7	22.3	23.6	23.9		
	< 0	0	0.1	0.2	0.3	0.4	0.5	0.6	3.9	8.3	22.2	ı	14.5	1	13.5
	K0	0.8	1.0	1.2	1.4	1.6	1.8	2.0	23.1	38.2	31.6	45.3	41.2	38.2	30.9
¥	8	0	0.1	0.2	0.3	0.4	0.5	0.6	4.6	10.8	24.2		23.3	,	20.1
É	r o	0.8	1.0	1.2	1.4	1.6	1.8	2.0	27.4	37.1	31.9	39.8	35.7	34.8	29
		-0.1	0	0.1	0.2	0.3	0.4	0.6	3.8	7.6		6.4	8.5	9.2	20.1
	20	0.8	1.0	1.2	1.4	1.6	1.8	2.0	27.4	37.1	31.9	39.8	35.7	34.8	29
		-0.1	0	0.1	0.2	0.3	0.4	0.6	2.7	5.2		4.3	9.1	10.3	9.1
	30	0.8	1.0	1.2	1.4	1.6	1.8	2.0	9.2	15	27.1	25.4	24.3	39.3	39.9
		0	0.28	0.56	0.71	0.86	1.0	1.2	24	39	63	66	105	112	81
<u>0</u>	29	1.3	1.5	1.6	1.8	1.9	2.1	2.2	82	65	57	53	62	64	52
		2.4							49						
	18	0.7	1.0	2.7	3.7	4.7	5.7	6.7	29.5	66.3	111.5	117.5	172.5	169.7	129.3
	117	0	0.6	0.9	1.2	1.5	1.8	2.1	86	172	158	160.5	183	182	172
	118	-0.3	0.3	0.6	0.9	1.2	1.5		15	144	111	226	188.5	89	
	84	7.2	7.3	7.5	7.6	7.8	7.9	8.1	120.5	ı	77	70	86.5	107	77.5
×	140^{\dagger}	7.1	7.3	7.4	7.6	7.7			127	134	128	302.5	237		
	140	7.2	7.4	7.5	7.7	7.8			148	122	ı	128.5	66.5		
	143	7.0	7.1	7.3	7.4				119.5	149	180	199			
	167 [†]	6.8	7.0	7.2	7.3	7.5	7.6		86	114	ı	171	154		
	167	7.1	7.2	7.4	7.5				136	135.5	ı	66			
	EDNT	0	0.15	0.3	0.45	0.61	0.76	0.91	147	144	217	224	151	147	121
	NIZC	1.07							184						
g	5 NI	0	0.15	0.3	0.45	0.61	0.76	0.91	190	231	349	409	422	457	417
ב	7170	1.07							398						
	ROW	-0.1	0	0.1	0.2	0.3	0.4	0.5	141	298	364	235	130	107	84
	V 2C	0.6	0.7	0.8	0.9	1.0			56	191	228	286	475		

Table A2-2. Concrete Surface Resistivity.

† Cracked concrete location.

0.3 182 249 251 110 551 260 340 326 161 340 326 117 250 181 121 0.3 69 442 29 326 117 250 181 121 0.1 113 87 71 42 355 35 37 37 0.1 113 87 71 42 41 33 32 0.1 -26 37 31 20 12 23 32 37 0.1 -28 524 31 22 33 32 0.3 500 538 501 533 623 337 523 0.3 660 431 273 370 74 57 0.3 415 337 303 34 41 38 0.3 445	U.3 182 249 251 110 551 260 340 348 121 0.3 69 442 29 326 117 250 181 121 0.3 69 44 29 356 562 443 387 121 0.1 113 87 71 42 41 33 32 0.1 113 87 71 42 41 33 32 0.1 - 33 34 31 20 12 32 0.1 - 33 34 31 20 23 32 0.1 - 33 32 314 117 29 23 0.3 528 501 533 241 13 223 0.3 16 23 37 37 30 37 23 0.3	0.3 182 249 251 110 551 260 340 198 429 326 117 250 181 121 0.3 69 44 29 355 262 443 313 37 37 0.1 113 87 71 42 41 33 32 0.1 113 87 71 42 41 33 32 0.1 -26 37 34 31 223 32 0.1 -28 24 23 34 31 229 23 0.3 500 536 337 266 372 623 623 0.3 166 331 273 370 743 623 0.3 42 28 37 26 30 77 74 57 0.3 44
$ \begin{array}{c ccccccccccccccccccccccccccccccccccc$		
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0.7 0.8 52 41 43 47 51 51 0.3 0.4 0.5 0.6 9.3 44.5 - 30.2 42.7 80.1 92.2 1.0 1.1 1.2 81.6 - 72.7 183.8 103.2 73.2 0.4 0.5 0.6 0.7 - 4.2 - 30.2 1.0 1.1 1.2 81.6 - 72.7 183.8 103.2 73.2 0.4 0.5 0.6 0.7 - 4.2 6.8 - 1.1 1.2 10.4 12.4 11.4 10.4 12.2 6.8 - 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5	$ \begin{array}{c ccccccccccccccccccccccccccccccccccc$	
0.3 0.4 0.5 0.6 9.3 44.5 - 30.2 42.7 80.1 92.2 1.0 1.1 1.2 81.6 - 72.7 183.8 103.2 73.2 0.4 0.5 0.6 0.7 - 4.2 4.2 6.8 - 1.1 1.2 81.6 - 72.7 183.8 103.2 73.2 - 0.4 0.5 0.6 0.7 - 4.2 6.8 - 1.1 1.2 10.4 12.4 11.4 10.4 12.2 - 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5	0.3 0.4 0.5 0.6 9.3 44.5 - 30.2 42.7 80.1 92.2 1.0 1.1 1.2 81.6 - 72.7 183.8 103.2 73.2 0.4 0.5 0.6 0.7 - 4.2 4.2 6.8 - 1.1 1.2 0.6 0.7 - 4.2 4.2 6.8 - 1.1 1.2 0.6 0.7 - 4.2 6.8 - 1.1 1.2 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5	0.3 0.4 0.5 0.6 9.3 44.5 - 30.2 42.7 80.1 92.2 1.0 1.1 1.2 81.6 - 72.7 183.8 103.2 73.2 92.4 0.4 0.5 0.6 0.7 - 4.2 - 4 4.2 6.8 - 1.1 1.2 0.6 0.7 - 4.2 - 4 4.2 6.8 - 1.1 1.2 10.4 12.4 11.4 10.4 12.2 6.8 - 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5 0.3 0.4 0.5 0.6 8.2 4.3 5.6 5.7 7.1 9.4 -
1.0 1.1 1.2 81.6 - 72.7 183.8 103.2 73.2 0.4 0.5 0.6 0.7 - 4.2 - 4.2 6.8 - 1.1 1.2 10.4 12.4 11.4 10.4 12.2 6.8 - 0.4 0.5 0.6 0.7 - 4.2 6.8 - 1.1 1.2 10.4 12.4 11.4 10.2 6.8 - 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5	1.0 1.1 1.2 81.6 - 72.7 183.8 103.2 73.2 0.4 0.5 0.6 0.7 - 4.2 - 4.2 6.8 - 1.1 1.2 10.4 12.4 11.4 10.4 12.4 11.2 - 4.2 6.8 - 0.4 0.5 0.6 0.7 - 4.2 6.8 - - 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5 0.4 0.5 6.8 6.4 - 6.9 7.7 8.6 6.9 8.5	1.0 1.1 1.2 81.6 - 72.7 183.8 103.2 73.2 0.4 0.5 0.6 0.7 - 4.2 - 4.2 6.8 - 1.1 1.2 10.4 12.4 11.4 10.4 12.5 6.8 - 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5 0.3 0.4 0.5 0.6 8.2 4.3 5.6 5.7 7.1 9.4 -
0.4 0.5 0.6 0.7 - 4.2 - 4 4.2 6.8 - 1.1 1.2 - 10.4 12.4 11.4 10.4 12.2 - 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5	0.4 0.5 0.6 0.7 - 4.2 - 4.2 6.8 - 1.1 1.2 - 10.4 12.4 11.4 10.4 12.2 - 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5	0.4 0.5 0.6 0.7 - 4.2 - 4.2 6.8 - 1.1 1.2 10.4 12.4 11.4 10.4 12.2 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5 0.3 0.4 0.5 0.6 8.2 4.3 5.6 5.7 7.1 9.4 -
1.1 1.2 10.4 12.4 11.4 10.4 12.2 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5	1.1 1.2 10.4 12.4 11.4 10.4 12.2 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5 1 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5	1.1 1.2 10.4 12.4 11.4 10.4 12.2 0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5 0.3 0.4 0.5 0.6 8.2 4.3 5.6 5.7 7.1 9.4 -
0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5	0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5 8.5	0.4 0.5 0.6 0.7 9.5 - 6.9 7.7 8.6 6.9 8.5 0.3 0.4 0.5 0.6 8.2 4.3 5.6 5.7 7.1 9.4 -
	6.8 6.4	0.3 0.4 0.5 6.4 6.4 1 <th1< th=""> <th1< th=""> <th1<< td=""></th1<<></th1<></th1<>

Table A2-2. Concrete Surface Resistivity (continued).

† Cracked concrete location.