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Corrosion Performance of Concrete Cylinder Piles

by

Kingsley Lau

A thesis submitted in partial fulfillment
of the requirements for the degree of
Master of Science in Civil Engineering
Department of Civil and Environmental Engineering
College of Engineering
University of South Florida

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Rajan Sen, Ph.D.
Peter Stroot, Ph.D.

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Corrosion Performance of Concrete Cylinder Piles

Kingsley Lau

ABSTRACT

Concrete cylinder piles produced by a centrifugally cast, vibrated, roller compacted process have shown promising corrosion durability in marine environments. Three bridges in the Florida Panhandle with approximately 40 years of service in aggressive marine service were examined. A newly constructed marine bridge utilizing concrete cylinder piles was also examined to verify corrosion performance of piles manufactured with modern building materials and construction compared to piles built several decades ago. Survey of the marine bridges showed minimal corrosion distress despite low design concrete cover to steel hoop reinforcements (20-40 mm). Typical concrete distress included minor rust staining but not necessarily indicating corrosion of reinforcement steel. Thin longitudinal cracks were frequently observed but were likely caused by mechanical damage from pile driving rather than stemming from corrosion distress. Chloride ion diffusivity was low, in the order of $1 \times 10^{-13} \text{ m}^2/\text{s}$. Other measured parameters such as concrete resistivity, porosity, and water absorption indicate low permeability. Chloride analysis of cracked and uncracked concrete cores from the older bridges in this study did not show

pronounced preferential chloride penetration. Chloride analysis from the contemporary marine bridge did show some preferential transport of chloride ions at shallow depths through cracks with further evidence of lower electrical resistivity indicating enhanced electrolyte transport. The pore water pH of concrete samples from the contemporary bridge was high despite the presence of pozzolanic materials suggesting that normal chloride threshold values may be valid. Simplified corrosion durability modeling projections indicate that a moderate relaxation of current FDOT concrete cover requirements may be valid without severely compromising the corrosion durability of concrete cylinder piles.

1 Introduction

1.1 Project Scope

The corrosion damage mechanism of steel in concrete induced by chloride ions is viewed as a two step process of initiation and propagation. In the initiation stage, chloride ions penetrate through the concrete cover by bulk diffusion, capillary suction, and permeation. The build up of chloride ions at the steel surface upon reaching a critical threshold value C_T causes depassivation of the reinforcing steel and initiates active corrosion. In the propagation stage, active corrosion of the steel continues where expansive corrosion products may damage the surrounding concrete until concrete distress is manifested as cracks, spalls, and concrete delamination. The durability of the structure is reduced by the effects of steel cross-section loss and concrete distresses. The corrosion service life of a concrete structure can be viewed as the time to concrete distress (time elapsed during corrosion initiation (t_i) and propagation (t_p)).

Durability design strategies seek to extend t_i as much as possible, since control of corrosion after initiation is comparatively difficult. The time t_i becomes longer with greater concrete cover, and less permeable concrete. Increased C_T , and lower chloride levels at the concrete surface also extend t_i but are less

amenable to control during design. As a result, design guidelines including those adopted by the Florida Department of Transportation (FDOT) address minimum concrete cover and concrete quality as the principal means to obtain durability expectations of many decades. Concrete cover is particularly important since corrosion forecasting models indicate that, all else being equal, t_i is proportional to the square of the concrete cover thickness (for example, reduction of the cover from 3 to 1.5 inches (7.5 to 3.8 cm) would shorten the initiation stage by 75%). Consequently, there is reluctance to reduce clear cover requirements unless exceptional circumstances arise.

Other durability factors than have been noted in the literature including improper quality control and construction difficulties of cylinder piles [1]; longitudinal cracking may result from differential shrinkage during fabrication, improper construction and pile driving. The presence of cracks may result in faster chloride ion penetration.

Indications of apparently exceptional corrosion resistance have been obtained in a 1997-8 assessment [2] of the substructure of the Escambia Bay Bridges, No. 580071 and 580058, located in highly aggressive estuary water with chloride concentrations at times exceeding 10,000 ppm. The substructure of these bridges was built in 1966 using circular Centrifugally Cast, Vibrated, Roller Compacted (CEN-VI-RO process) Raymond piles. There are 268 54-inch (1.4 m)

diameter and 1,218 36-inch (0.9 m) diameter cylinder piles in water. The nominal clear concrete cover for the spiral hoop reinforcement of either size of pile is only 1.375 inch (3.5 cm), and actual cover values as small as 0.75 inch (1.9 cm) were recorded. Despite the low cover, aggressive environment, and 30-year long exposure, no indication of significant corrosion damage was observed. Detailed examination of chloride penetration was performed in 17 cores extracted from the piles. Analysis of chloride concentration profiles yields the apparent chloride diffusion coefficient of the concrete D , which is an indication of its permeability to chlorides. The analyses indicated diffusivity values that in most cases were less than $0.01 \text{ in}^2/\text{y}$ ($2 \times 10^{-13} \text{ m}^2/\text{s}$)—among the lowest values ever determined for conventional FDOT concrete bridge substructures examined to date [3].

Although D was small, the concrete cover was thin which allowed appreciable chloride buildup at the hoop reinforcement depth. In the tidal zone, the chloride concentration (0.188% by concrete weight or 4.5 kg/m^3 assuming a concrete unit weight 2400 kg/m^3) [2] had already exceeded commonly accepted C_T values for plain steel bars. The absence of corrosion damage indicated that either corrosion was already in progress but at a very slow rate (a large value of t_p) or that the value of C_T for the spiral steel/concrete combination was higher than encountered in conventional reinforced concrete. Chloride concentrations close to commonly accepted C_T values were observed in the splash-evaporation zone, suggesting that corrosion may begin in the near future.

A corrosion degradation forecast model was developed for the Escambia Bay bridges [2] based on the corrosion assessment data and information on the performance of structures elsewhere. The model parameters included estimated values for C_T and t_p that reflected the present condition of the structure. The forecast functions projected significant but still moderate corrosion damage development over the next 60 years of service. The damage projections were used together with cost data for repair procedures to estimate life-cycle costs over that period. The life cycle cost estimates for the next 60 years indicated that keeping the present substructure in service even beyond the 75 year design goal was preferable to demolition and erection of a new bridge.

Cylinder pile construction is optimized for relatively low concrete covers. There is considerable interest in establishing whether normal cover requirements (presently 3 in (7.6 cm) adopted by FDOT [4]) could be relaxed for that type of pile to thicknesses comparable to those used in the Escambia Bay bridges. The feasibility of reliably constructing 36-in (0.9 m) or 66-in (1.7 m) diameter cylinder piles with 3 in (7.6 cm) covers has been questioned [5], and there is also concern that cracks may develop more easily when the cover is thick. Unfortunately the information available from the 1997-8 survey is too limited to fully substantiate the use of concrete covers much thinner than present FDOT specifications for a 75-year design service life goal. Only a limited number of cylinder piles were

examined in detail; as presented earlier, the deterioration models had to estimate critical parameter values in lieu of limited or absent information.

The present investigation is intended to expand the knowledge on the present and future performance of cylinder piles in marine service. The investigation examined whether other bridges built with cylinder piles showed corrosion resistance comparable to that encountered in the Escambia Bay bridges. The investigation also sought to identify any factors responsible for that performance, with a view to considering redefining cover guidelines for cylinder pile construction.

Cylinder piles of three approximately 40 year old FDOT bridges, and another one newly built, were examined to establish corrosion performance. Activities performed included visual surveys, coring to assess chloride penetration and steel condition, and other electrochemical and physicochemical concrete property measurements in the field and laboratory.

Much of the research described in this thesis was conducted in a collaborative effort by the author and Lan Yao at the University of South Florida Corrosion Engineering Laboratory and researchers at the Materials Office of the Florida Department of Transportation. Results from early experimentation and preliminary data analysis contributed by Ms. Yao are acknowledged when

introduced in the corresponding sections. Thorough evaluation of all data was conducted by the author under the direction of Dr. Alberto Sagüés¹. A brief review of documented cylinder pile performance, compiled by Ms. Yao, is presented in Table 1.1. Methodology and findings of the field and laboratory investigations are detailed in the subsequent sections.

1.2 Performance of Cylinder Piles

Cylinder pile manufacturing has been reviewed in detail elsewhere [6- 10]. The cylinder piles are made of segments typically 16 ft (4.9 m) in length. A spiral wire cage with welded longitudinal spacing bars is placed in an assembly consisting of the cylinder pile form and end rings that maintain the placement rods that form the strand ducts typically 1.375 in (3.5 cm) in diameter. Concrete is placed into the form that is being rotated in the Cen-Vi-Ro process where the concrete is centrifugally cast into the form, vibrated, and roller compacted. After concrete curing and form stripping, the segments are aligned by the duct openings for pile assembly. Long piles are made by assembling several cylinder pile segments together by post-tensioning prestressing strands through the duct openings. The strands are tensioned and pressure grouted. The pile segment joints are covered with epoxy. During bridge construction, the piles are cut to

¹This thesis was the basis of the Final Report to the Florida Department of Transportation "Corrosion Performance of Concrete Cylinder Piles" BC353 RPWO#10 (2005) -605 Suwannee St. MS30 Tallahassee, FL 32399- by K. Lau, A.A. Sagüés, and L. Yao. The author of this thesis was the senior contributing author to that report under the direction of the Principal Investigator, A.A. Sagüés, Ph.D, P.E. Much of the research described here is also in the report to FDOT.

length. The diameter of the cylinder piles typically range from 36 – 66 in (0.9 – 1.7 m). Design information from the Hathaway Bridge construction drawings, archived in the FDOT Structures Office, indicate cylinder pile wall thickness 5 in (12.7 cm) with a clear cover approximately 1.75 in (4.4 cm); however, these design values have been enlarged in newer FDOT specifications. An example of a cylinder pile cross section from the FDOT Structures Manual is shown in Figure 1.1.

Figure 1.1 Cylinder Pile Cross-section. From FDOT Structures Manual 2004.

Based on the available records reported in literature [9,11-15], personal communication with FDOT personnel and Bayshore Concrete Products Corporation (BCP), it is conservatively estimated that at least 75 bridges have been built as of 2002 with Raymond Piles in the United States. Concrete cylinder

piles typically have high cement factors, 650 - 800 pcy (385 - 474 kg/m³) [9]; the water to cement ratio has been reported to be 0.35 - 0.45 [2]. Records from the St. George Island Bridge cylinder pile mix design from Gulf Coast Prestress, Inc. also indicate high cement factors, 840 pcy (~500 kg/m³) and low water to cement ratio (0.31).

A number of investigations have documented cracks in Raymond Piles [12-14]; however, most of the cracks were found to be minor or moderate without severe chloride buildup in adjacent areas [12-13]. The cracks were not likely caused by the expansion of corrosion products but rather by elastic rebound of prestress ducts, differential shrinkage, frost damage, or damage from hammering [13]. In a 1995 inspection of the twin bridges over Lake Ponchartrain, LA cracks appeared to extend to strand depth but without corrosion rust staining [1]. In the same inspection, localized damage to the transverse joint epoxy seal was observed. A brief survey of Raymond Pile bridges as of 2002 was prepared and a sampling of findings is listed in Table 1.1. As shown in Table 1.1, Raymond piles are used in coastal states such as Florida, Georgia, Alabama, Mississippi, Louisiana, South Carolina, North Carolina, Virginia, Maryland, New York, New Jersey, and Rhode Island. The results of the survey indicated generally good corrosion performance, with the caveat that thin cracking, usually longitudinal and normally neither a cause nor a consequence of corrosion, has been reported in many instances.

Table 1.1. Sampling of Bridges Built with Circular Piles and Performance Notes- 2002 summary (compiled by Lan Yao).

State/ No.	Bridge Name	Location	Year Built	Appx. Length	Pile Diam.(in)	Performance
Florida						
1	Escambia Bay (EB)	Escambia Bay	1966	13597'	54 & 36	No significant corrosion damage found on piles in 1997 (1)
2	Escambia Bay (WB)	Escambia Bay	1966	13597'	54 & 36	No significant corrosion damage found on piles in 1997 (1)
3	Brooks	Okaloosa	1964	1318'	36	Substructure was in satisfactory condition in June, 2001 (2)
4	Pensacola Bay	Pensacola Bay	1960	15640'	54	Substructure was in satisfactory condition in June, 2001 (2)
5	Hathaway	Panama City	1960	3358'	54	Substructure was in fair condition in February, 2001 (2)
6	Lake Underhill	Orlando	1974	N/A	36	N/A
7	St. George Island	St. George Island	2003	3.1 mile	54	Under construction in 2002
Louisiana						
1	Lake Pontchartrain Causeway	New Orleans	~1955	24 mile	54	Vertical hairline cracks were found on some piles in 1995 (3) some localized damage to transverse joint epoxy seal (4)
2	Lake Pontchartrain Causeway	New Orleans	~1965	24 mile	54	Vertical cracks were found on some piles in 1995 (3)
3	4500700651 (EB)	Baton Rouge	N/A	N/A	54	Moderate damage and hairline cracks were found on some piles in 1998 (5)
4	4500700652 (WB)	Baton Rouge	N/A	N/A	54	
5	4500614951 (EB)	Baton Rouge	N/A	N/A	54	N/A
6	4500614952 (WB)	Baton Rouge	N/A	N/A	54	Hairline cracks on some piles 1998 (5)
7	4501400001 (EB)	New Orleans	N/A	N/A	54	Hairline cracks on some piles 1998 (5)
8	4501400002 (WB)	New Orleans	N/A	N/A	54	Hairline cracks on some piles 1998 (5)
9	US 11 bridge	New Orleans	N/A	4.76 mile	54	N/A
10	west pearl bridge	N/A	N/A	N/A	N/A	Hairline crack over post-tensioning duct (4)
11	Wax Lake Outlet bridge	New Orleans	N/A	N/A	N/A	Hairline crack over post-tensioning duct (4)
Georgia						
1	Elba Island	Elba Island	N/A	1780'	54	N/A
2	State Road 303 Turtle River	Glynn Co.	1963~1965	2968'	54 & 36	N/A
3	South Bronswick River	Glynn Co.	1963~1964	1160'	54 & 36	N/A

Table 1.1 (Continued)

State/ No.	Bridge Name	Location	Year Built	Appx. Length	Pile Diam. (in)	Performance
New York						
1	Loop Parkway	Nassau Co.	~1999	N/A	66	N/A
2	Peconic River Bridge	N/A	N/A	N/A	54	N/A
3	Lake Chautauqua Bridge Cooper & Wando Rivers Bridge	Chautauqua Co.	N/A	N/A	36 & 54	N/A
4		N/A	N/A	N/A	36	N/A
5	Ponquogue Ave. Bridge	Long Island	N/A	N/A	54	N/A
6	Sag Harbor Bridge	N/A	N/A	N/A	36	N/A
7	Wantagh Parkway over Goose Creek	N/A	N/A	N/A	54	N/A
8	NYDOT Pedestrian	N/A	N/A	N/A	36	N/A
9	Cross Bay Boulevard	N/A	N/A	N/A	54	N/A
Virginia						
1	James River Monitor –Merrimac Bridge Tunnel (#2236)	Newport News	1980	23200'	54	In good or fair condition (7)
2	Monitor –Merrimac Bridge Tunnel (#2237)	Newport News	1990	16685'	54	In good or fair condition (7)
3	Hampton Roads Bridge Tunnel (EB)	Newport News	1990	16636'	54	In good or fair condition (7)
4	Hampton Roads Bridge Tunnel (WB)	Norfolk	1974	5925'	54	In good or fair condition (7)
5	City of Norfolk (#1865)	Norfolk	1957	6151'	54	In good or fair condition (7)
6	City of Norfolk (#1866)	Lafayette River	1994	1755'	36	In good or fair condition (7)
7	Coleman Bridge	Lafayette River	1970	1755'	36	In good or fair condition (7)
8	Chesapeake Bay Bridge Tunnel	Yorktown Chesapeake Bay	1952	3750'	54	N/A
9			1960~1964	17.6 mile	54	Vertical cracks found on some piles (1988) (5.6)

Table 1.1 (Continued)

State/ No.	Bridge Name	Location	Year Built	Appx. Length	Pile Diam. (in)	Performance
Maryland						
1	William Preston Lane Memorial (WB)	Annapolis	~ 1973	4.3 mile	54	Severe cracks found on piles; cracks were repaired with epoxy (5)
2	Patuxent River Bridge	N/A	N/A	N/A	54	N/A
3	Severn River Bridge	Anne Arundel	N/A	N/A	54	N/A
4	Bridge over Bear Creek	N/A	N/A	N/A	36	N/A
5	Miles River Bridge	Rt. 370	N/A	N/A	54	N/A
6	Choptank River Bridge	Dorchester Co.	N/A	N/A	54	N/A
7	Sassafras River Bridge	N/A	N/A	N/A	42	N/A
8	Nanticoke River Bridge	Dorchester Co.	N/A	N/A	66	N/A
9	Back River Bridge	N/A	N/A	N/A	54	N/A
10	Deal Island Bridge	Somerset Co.	N/A	N/A	54	N/A
11	Bohemia River	Cecil Co.	N/A	N/A	N/A	N/A
North Carolina						
1	Nelson's Bay	Carteret Co.	1982	1480'	54	Piles were in satisfactory condition in 1999 (8)
2	Lake Drummond Canal (Bridges #43)	Camden Co.	1982	2132'	54	Piles were in satisfactory condition in 1999 (8)
3	Lake Drummond Canal (Bridges #44)	Camden Co.	1982	2132'	54	Piles were in satisfactory condition in 1999 (8)
4	Bonner Bridge	Dare Co.	1962	12,865'	66	Piles were in acceptable condition in 1999 (8)
5	Sneads Ferry	Onslow Co.	1993	2781'	54	Piles were in poor condition and repaired in 2000 (8)
6	Oregon Inlet Bridge	NC 12	N/A	N/A	66	Piles were in good condition in 2000 (8)
7	Chowan River Bridge	Bertie Co.	1996-2000	9500'	66	N/A

Table 1.1 (Continued)

State/ No.	Bridge Name	Location	Year Built	Appx. Length	Pile Diam.(in)	Performance
New Jersey						
1	Great Egg Harbor Inlet	Ocean City	2001	3450'	54	Under construction (9)
2	Watson's Creek	Rt. 29, 10B	1990	1500'	36	Acceptable (9)
3	Watson's Creek	Rt. 29, 10B	1991	1000'	54	Acceptable (9)
4	Risley's Channel	Rt. 152, 1B	1989	1425'	54	Acceptable (9)
5	Crosswick's Creek	Rt. 295	1994	1400'	54	Acceptable (9)
6	Watson's Creek	Rt. 295, 17H	1991	700'	36	Acceptable (9)
7	Watson's Creek	Rt. 295, 17H	1991	700'	54	Acceptable (9)
8	Navasink River	Rt. 35	2000	1000'	54	Acceptable (9)
9	Barnegat Bay (WB)	Rt. 37	1974	4750'	N/A	Acceptable (9)
10	Absecon Inlet Bridge	N/A	N/A	N/A	36 & 54	N/A
11	Middle Thorofare Bridge	Cape May Co.	N/A	N/A	36	N/A
South Carolina						
1	Cooper River Bridge	Charleston	~ 1992	5.1 mile	54	N/A
Rhode Island						
1	Jamestown-Verrazzano Bridge	N/A	N/A	7350'	36	N/A
California						
1	San Diego-Coronado Bridge	San Diego	1967~1969	11,179'	54	Cracks appeared on piles during construction and delivery in 1969 (5)
Mississippi						
1	Bayou Bernard	Harrison Co.	2000	1378'	66	N/A
2	west pearl bridge	N/A	N/A	N/A	54	N/A
Alabama						
1	Mobile Bay Causeway Bridge	N/A	N/A	N/A	54	Vertical hairline cracks randomly distributed on the piles (5)

Table 1.1 (Continued)

Location/ No.	Bridge Name	Location	Year Built	Appx. Length	Pile Diam.(in)	Performance
Puerto Rico						
1	Arecibo River Bridge	N/A	N/A	N/A	36	N/A
2	Rio LaPlata Bridge	Toa Baja	~ 1984	2541'	54	N/A
3	Victor Rojas (Rehabilitation)	N/A	N/A	N/A	54	N/A
4	Santo Domingo	N/A	N/A	N/A	54	N/A
5	Deigo Expressway	N/A	N/A	N/A	36	N/A
6	Rio Grande De Loiza	N/A	N/A	N/A	54	N/A
Saudi Arabia						
1	Ju'Aymah Trestles	Saudi Arab	1977~1979	6 mile	54 & 66	Marine mollusk borers attacked the piles 1982 (10); vertical cracks were found on the piles (11)

Table 1.1 Notes

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4. Ahn, Woosuk, et al. "A Study on Corrosion Resistance of Prestressed Marine Concrete Piles." *Corrosion/2003*. March 16, 2003.
5. Avent, R. and D.J. Mukal, "Investigation of Cracks in Cylindrical Spun-Cast Concrete Piles in a Marine Environment." Final Report for Louisiana Transportation Research Center. 1998.
6. Garder, Donald. "Inspection of Substructure of the Chesapeake Bay Bridge-Tunnel Above and Below the Waterline." *Transportation Research Record*. p.130-137.
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10. Snow, R. "Attack by Lithophaga and Cliona on Marine Concrete Structures: Some Prevention and Control Measures." Oxford & IBH Publishing Co. PVT. Ltd, 1988
11. Gerwick, Ben. "International Experience in the Performance of Marine Concrete." *Concrete International*. May 1990. p.47-53.

2 Investigation Methodology

2.1 Bridges Investigated

Cylinder piles of three approximately 40 year old FDOT marine bridges, and another one newly built (Table 2.1), were examined to establish corrosion performance. The newly built bridge, constructed in 2003, was examined to compare corrosion performance of cylinder piles representative of current FDOT building guidelines to that of cylinder piles several decades old. The cylinder piles in all bridges showed joints with segmented construction, which is a characteristic of Raymond Piles. Further evidence of Raymond Pile construction included the presence of grout around strands exposed by coring in the Hathaway and Pensacola Bay Bridge. Personal communications² confirmed that Raymond Pile construction was used for the Hathaway Bridge.

² Don Theobald, V.P. Engineering, Gulf Coast Prestress, Inc. and Bob Bruce, Ph.D., Catherine and Henry Boh Chair in Civil Engineering, Tulane University, formerly of Raymond International.

Table 2.1. Surveyed Bridges

Bridge	Number	Year Built	Inspection Date	Location	Water Chloride Content (ppm) ³	Length	No. of Bents/ Cylinder Piles	Pile Diameter
Hathaway	460012	1960	Dec. 2002	Panama City, FL	25 560	3 359 ft. (1 024 m)	34 / 74	54 in. (1.37 m)
Pensacola Bay	480035	1960	March 2003	Pensacola, FL	7 374	15 640 ft. (4 767 m)	257 / 916	54 in. (1.37 m)
Brooks	570034	1964	March 2003	Ft. Walton Beach, FL	9 833	1 318 ft. (402 m)	14 / 80	36 in. (0.91 m)
St. George Island (SGI)	490003	2003	March 2004	St. George Island, FL	8 875	21 615 ft. (6 588 m)	~165 / ~646	54 in. (1.37 m)

The Escambia Bay Bridges, also in the Florida Panhandle coast and built with Raymond Piles, were surveyed in 1997 in a manner similar to that used here. The results of that investigation have been reported elsewhere [2] and are used here for comparison whenever appropriate

2.2 Methodology

Field activities involved in the inspection of the concrete cylinder piles were coordinated among personnel from the Corrosion Laboratory of the FDOT

³ FDOT records.

Materials Office and the Corrosion Engineering Laboratory of the University of South Florida (USF). Subsequent laboratory experiments were conducted by USF personnel at the Corrosion Engineering Laboratory at the University of South Florida and at the FDOT Materials Office; some chloride analyses were contracted to A&S Laboratories, Clearwater, FL.

2.2.1 Field investigation methods

Field bridge evaluation included visual surveying for corrosion distress, concrete core sampling, testing of probable corrosion status, and concrete internal relative humidity measurements. Prior accounts of bridge deficiencies as documented by FDOT inspections can be found in the FDOT Bridge Management System bridge inspection reports [16-18]. All accessible marine piles in each bridge were visually examined, except for the newly constructed St. George Island (SGI) Bridge where only selected piles installed at the beginning of construction were assessed. A general survey of the piles was taken first by boat from approximately 6 ft (2 m) away, and systematically progressing along both sides of pile group bents. Attention was focused on the tidal and splash zones of the piles where corrosion of steel reinforcement is expected to be most severe. Notation and photographic documentation of distress type and location were made. Piles that exhibited severe signs of corrosion distress were selected for examination at close range by docking the boat to the pile group bent. Hammer soundings were conducted on selected piles with severe rusting or

other signs of distress to determine the presence and extent of concrete delamination (internal separation of concrete). The presence of concrete spalls (actual, partial or total loss of cover concrete) either pre-existing or as a result of hammer testing were noted as well. The type, width (measured with a CTL crack comparator gage), and probable cause of observed cracking were assessed. For those piles where reinforcement steel had been exposed during coring, half-cell potentials were measured with a saturated copper/copper sulfate reference electrode (CSE). Electrical continuity between the steel reinforcement hoop wire and strands was tested when possible.

Concrete internal relative humidity (IRH) measurements [19], were made when permitted by time constraints. A Vaisala humidity/temperature probe was inserted into a sealed cylindrical cavity, bored to a depth of 1.25 in (3.2 cm) from the pile external surface (Figure 2.1), effectively sampling the humidity of air in equilibrium with the concrete at the bottom of the cavity. A closed-cap stabilization time of approximately 20 hours was typically allowed after drilling/installing the fitting and the first IRH measurement; the fittings were left in place for future monitoring. The internal relative humidity was sampled at elevations approximately 6 in (0.15 m) and 5 ft (1.5 m) above high tide (AHT).

Figure 2.1 Schematic of concrete internal relative humidity set-up

Concrete core samples were extracted from the piles at the tidal and splash zones as well as at higher elevations. A nominally 2 in (5 cm) diameter cylindrical core drill bit was used resulting in cores of approximately 1.75 in (4.5 cm) in diameter. Coring of concrete samples was made to depths of approximately 3.5 in (8.9 cm) or upon reaching post-tension strands. The intersecting spiral wire and spacing rebar were typically severed by coring in the older bridges in the study; they were not severed in the SGI Bridge. The strand assembly was not severed, but may be nicked by the coring drill bit. The bit was continuously cooled with fresh water while drilling. Core sample elevations ranged from -6 in to 5 ft (-0.15 to 1.5 m) AHT. The cores were typically extracted in pairs along the same elevation 6 in (0.15 m) apart on center. If a noticeable vertical concrete crack was observed, two pairs of concrete cores were extracted

around a given elevation. One core from each pair was centered on the crack, and the four cores formed the corners of a square 6 in (0.15 m) on the side.

Additional samples including 3.5 in (8.9 cm) diameter concrete cores of piles that have not been placed in service were gathered from cutoff pile segments from construction of the SGI Bridge.

2.2.2 Laboratory experimental methods

For chloride analysis selected cores were wet cut with a fine blade tile saw, cooled with fresh water, to obtain thin slice sections. Thinner sections were cut near the external concrete surface to better assess the rapid change of chloride concentration near the external surface. The thin sections were ground into powder for analysis. The initial 0.08 in (2 mm) of the concrete core was directly powdered by shaving in a lathe and carefully collecting the powder. The concrete cores extracted from the SGI Bridge piles, which had very narrow chloride concentration profiles, were sampled for near surface chloride concentration using profile grinding [20]. A 0.56 in (1.4 cm) diameter carbide-tipped T-slot cutter was used to mill layers from the concrete core as thin as ~ 0.04 in (1 mm). Thinner and more frequent layers were milled at the external surface of the concrete core. Thicker sections were subsequently sampled from portions deeper into the concrete by milling larger depths and cutting thin slices with a fine blade tile saw.

Some of the 3.5 in (8.9 cm) diameter concrete cores from unused pile sections of the SGI Bridge were used to make specimens for ponding with salt solution following a modified Nordtest method [21], to measure chloride penetration parameters. Cut samples, approximately 1.5 in (3.8 cm) in length, were sealed with epoxy leaving only the cut surface exposed to 2.8 M NaCl solution by means of a dam constructed for each individual sample. The salt solution was filled to a depth 0.5 in (1.3 cm) and was periodically changed with new solution. Ponding was conducted for an extended period (approximately 22 months) in consideration of the anticipated high quality of the concrete. Profile grinding was conducted by the milling procedure described above.

The sampled concrete powder was normally analyzed for total chloride content following the regular FDOT procedure [22]. Samples of concrete cores where chloride penetration had been significant due to longer exposure to seawater were contracted for chloride analysis to a certified FDOT provider (A&S Laboratories, Clearwater, FL). For concrete cores where near-surface chloride penetration was sampled by profile grinding the FDOT procedure was modified to use a smaller amount of powder per test (typically 1.5 g instead of 3 g). Control tests with concrete samples of known chloride content showed no significant loss in accuracy; the homogeneity of the powder sample was addressed by thoroughly mixing the sample and making duplicate measurements. In all cases the concrete powder was oven dried (105°C) and digested with boiling nitric acid

solution. The dissolved chlorides were filtered from the undissolved remnants such as sand. Potentiometric titration, with a silver ion selective electrode and AgNO_3 titrant, was conducted to measure the chloride content of each sample. Results are reported in Section 2.4.1 as a ratio of chloride mass to dry concrete mass, mg Cl^- per g of dry concrete.

The apparent chloride diffusivity, D , chloride surface concentration, C_s , and native chloride concentration, C_o , were obtained by fitting the measured chloride content and depth profile to an idealized diffusion profile described by Eq.1 [23].

$$C(x,t) = C_s - (C_s - C_o) \operatorname{erf} \frac{x}{2\sqrt{Dt}} \quad (1)$$

where t is the time of service in seawater upon extraction and x is the depth from the concrete core external surface. It is noted that this profile results from assuming ideal Fick's 2nd law diffusion behavior with time- and space-invariant diffusivity in a homogeneous medium, as well as a constant surface concentration and uniform initial concentration [23]. The resulting fit parameters are consequently only nominal values.

Additional tests aimed at assessing concrete permeability were conducted by conditioning selected cores in 100% relative humidity chambers. Concrete water absorption was monitored upon reaching constant saturated weight.

Electrical resistivity was measured with a CNS Electronics RM MKII Wenner-array probe or a Nilsson Model 400 resistivity meter set up with a 4-point electrode configuration. The terminal wet resistivity, corrected for core geometry [24] is reported as ρ_{wet} . Internal concrete relative humidity for selected core samples was monitored for ambient and conditioned environments. A sealed cylindrical cavity was bored from the original surface of concrete samples collected from the unused portion of the SGI Bridge piles. Similar to field measurements, a Vaisala humidity/temperature probe was inserted into the cavity sampling the humidity of the air in equilibrium with the concrete. The IRH of the concrete was initially recorded in ambient conditions and subsequently measured as the samples were being conditioned in 100% relative humidity chambers. Concrete permeability was also tested by density, absorption, and porosity measurements following ASTM C 642 standard procedures [25].

Figure 2.2 Schematic of ISL concrete samples. (dimensions in cm)

Other concrete core samples were conditioned in 100% relative humidity and used to measure the pH of pore water solution using the in-situ leaching technique (ISL) [26]. Different sample configurations (Figure 2.2) were made depending on the geometry of the specimens. Acrylic washers and rubber stoppers were used to seal residue-free equidistant cavities drilled with a 0.125 in (3.2 mm) masonry drill bit to depths as shown in Figure 2.2. Distilled water was pipetted into the cavities depending on the size of the cavities and in consideration for time required for equilibrium between the surrounding pore water and the cavity water. For the 3.5 in (8.89 cm) and 1.75 in (4.44 cm) diameter cores, and core fragments with shorter lengths, the amounts of distilled water used were 0.15 mL, 0.10 mL, and 0.08 mL respectively. If the water in the cavities dried up, the pH measurements were repeated by introducing new distilled water into the cavities. Measurements were made with a micro-pH electrode calibrated with certified pH buffer solutions.

3 Field Investigation Results

3.1 Hathaway Bridge

A detailed review of the survey results is included in Appendix I Table A1a. Cylinder piles, all 54-in diameter, were typically configured four to a bent and located in the approach spans. As a naming convention for this thesis, the bents were numbered from the west and the piles were numbered from the south. Throughout this thesis individual piles are named as in the following example: H-8-2 for Hathaway Bridge bent 8, pile 2. Center span piers⁴ numbered 15 to 28 were comprised of square piles and will not be considered here. A total of 76 piles were examined. Unpublished data from field activities conducted for the Hathaway Bridge used in this thesis was collected by Lan Yao for USF in 2002.

3.1.1 Concrete Observations

The majority of the piles showed no visual signs of corrosion distress. Seven piles had rust stains typically 1 ft (0.30 m) below pile cap in the atmospheric exposure zone where severe chloride-induced corrosion is less

⁴ Numerical labels of center span piers are continuous with approach span bents as convention for report.

common. These stains were likely a result from steel exposed due to mechanical damage from pile driving. Three piles had rust stains at intermediate elevations, and only four piles showed rust stains at low elevations (splash zone). Few cracks were noted.

Coring in this bridge was conducted in one pile each from bents 8 and 29. Pile H-8-2 was one of those exhibiting rust stains in the tidal/splash zone as well as some concrete delamination. Six cores were extracted in pairs at elevations below high tide, within splash zone, and at atmospheric exposure zone. The core samples were labeled H-8-2-1 through H-8-2-6. Pile 3 of bent 29 (H-29-3) had a hairline crack ~0.013 in (0.3 mm) thick, but no rust was observed. Four cores were extracted in pairs from the splash zone and at atmospheric exposure zone, labeled H-29-3-1 to H-29-3-4.

3.1.2 Steel Condition Observations

The coring of the piles sampled exposed steel that was cut and removed with the core or steel that showed at the bottom of the core hole where the core broke off. Most of the exposed steel was 0.25 in (6.3 mm) diameter spiral hoop reinforcement. Post tensioning strand was exposed at the bottom of two of the cores. One of those cores, H29-3-3 clearly showed grout and the trace of the

duct opening around the strand. A spacing rebar 0.25 in (~ 1.27 cm) diameter was cut in one core.

Visual condition of the exposed steel and clear concrete cover values are detailed in Appendix III, Table A3. For this and the other older bridges, the exposed steel in most instances could be broken cleanly from the surrounding concrete and had a dull to lustrous gray mill scale appearance, with no discernable rust. This no-corrosion condition, exemplified in Figure A4, is denoted by NC in the following. In a few instances the exposed hoop steel or spacing bar showed faint rust-colored discoloration on part of the exposed surface, as illustrated in Figure A5. Also in a few cases the exposed strand showed similar minor discoloration together sometimes with tiny rusty dots or stringers on the matching strand imprint on the surrounding grout (Figure 3.3). No cross sectional steel loss of any significance was observed associated with these cases, and it was not clear whether the discoloration represented corrosion that took place in service, or prior to casting. These light corrosion situations are denoted by LC in the following.

All steel exposed by coring in this bridge fell under the NC or LC categories. Despite the presence of an external rust stain on the concrete in the pile cored in Bent 8, both the steel reinforcement and strand exposed there had an NC visual appearance. In the Bent 29 pile the core revealing a strand and a

spacing rebar was drilled centered on a vertical hairline crack, and both steel surfaces had an LC rating. However, as shown later there was no evidence of significant enhanced chloride penetration from analysis of that core.

The cumulative distributions of clear cover values for this and the other bridges investigated are shown in Figure A2, and the results are summarized in Table A2. The average cover value for spiral reinforcement in the cores from this bridge was 1.20 in (3.1 cm). While this value represents measurements from only two columns in the bridge, it is close to the 1.5 in (3.8 cm) cover value inferred from the construction drawings for this bridge archived in the FDOT District 3 Structures Office. There the cover to the strand duct opening is listed as 1.75 in (4.4 cm); subtracting the 0.25 in thickness of the hoop wire assumed to be placed immediately next to the opening yields 1.5 in.

As expected, there was electrical continuity between spiral hoop reinforcement exposed at the perimeter or bottom of different core holes of the same pile. The one spacing rebar exposed was also continuous with the spiral wire. The two exposed strands were found to be not continuous to the other steel in the same pile, which is consistent with the construction method of Raymond piles where the strand is placed and grouted afterwards.

3.1.3 Field Measurements of Steel Potential and Concrete Moisture

Half-cell potentials as a function of elevation are listed in Appendix III Table A5 and a summary is shown in Figure 3.1. Potential levels of -200 mV and -350 mV vs. CSE are traditionally associated with indications of unlikely and likely ongoing corrosion, respectively, in atmospherically exposed concrete [27]. While the potential readings more negative than -350 mV CSE in Figure 3.1 are not a reliable indicator of corrosion in progress in wet concrete [28], values more positive than -200 mV CSE as seen at elevations above 1 to 3 ft (0.3 to 0.9 m) strongly suggest that the steel above those elevations was in the passive condition at the time of the measurements. The potential of the spacing rebar with surface rated as LC in the core drilled on a crack 5 ft (1.52 m) AHT in H-29-3 was -96 mV vs. CSE at that elevation, suggesting that active corrosion was not in progress there when tested.

Figure 3.1 Hathaway Bridge half-cell potential measurements. Grey symbols, near steel under LC rating.

Concrete internal relative humidity measurements were taken from the same two piles (H-8-2 and H-29-3). Measurements were taken at elevations within the tidal/splash zone and atmospheric exposure zone as listed in Appendix III Table A7. IRH measurements were made approximately 18 hours after the test fittings, described in Section 2.2.1, were installed and sealed. IRH measurements were made for approximately 1 hour after inserting the Vaisala probe into the fitting. Figure 3.2 shows the measured IRH as function of time upon insertion of the Vaisala probe into the fitting. These tests show that concrete IRH was generally higher at the tidal/splash zone (expected to have a

higher water content) than at higher elevations. Ambient conditions upon completion of testing were approximately 85% RH and 15°C.

Figure 3.2 Hathaway Bridge concrete internal relative humidity. Measurement readings as function of time after probe insertion, 18h after fitting installation. ○ ● IRH. □ ■ Temperature.

3.2 Pensacola Bay Bridge

A detailed review of the inspection survey is included in Appendix I, Table A1. Bents in the approach spans of the Pensacola Bay Bridge are typically arranged by four (normal piers) or six (tower piers, typically every 15 bents) cylinder piles, all 54-in diameter. The bents are numbered from the north and piles from the east. Bents 256 and 257 were not surveyed due to accessibility issues. Piles from piers 112-145 from the center span column piers were below tidal zone and were not inspected. A total of 908 circular piles were examined.

3.2.1 Concrete Observations

A cumulative count of deterioration features is shown in Figure A3 in Appendix II. The most commonly observed features, seen on approximately 25% of the piles examined, were rust stains typically one per affected pile and only a few square in ($1 \text{ in}^2 \sim 6.3 \text{ cm}^2$) in size. The stains were larger, of the type shown in Figure A1, in only 15 of the 908 piles examined. Repair patches were the next most common feature, often located just below the pile cap and occasionally at intermediate elevations approximately 1 to 3 ft (0.3 to 0.9 m) AHT. Nearly all of the patches were in the south approach spans, and approximately 40% of the observed rust stains in the south approach spans were on preexisting repair patches. As shown in Figure A3, rust stains not on repair patches were nearly evenly distributed along the bridge.

Concrete spalls already in place and associated with rust stains were observed only on six piles, each deteriorated area on separate piles. Those spalls were all small and in four instances (not further examined) only of thumbnail size and very shallow. As shown below, later hammer sounding revealed delaminations and created new spalls at nine other rust stain areas from eight other piles. Thin vertical cracks were occasionally observed but nearly always not together with any corrosion damage. The cumulative deterioration counts in Figure A3 (b) were roughly evenly distributed along the bridge.

Sixteen piles (fourteen among those that exhibited sizable rust stains and two with thin cracks approximately 0.013 in (0.3 mm) in width but no rust stains, Appendix I, Table A1b) were selected for closer inspection. Twelve of those piles, with severe rust stains not associated with repair patches or preexisting spalls, were hammer sounded to detect delamination. In four of the twelve the concrete was found to be sound despite signs of external rust staining. Delaminations were detected in the remaining eight and hammering continued to remove the affected cover concrete. The delaminated areas were not very large, ranging in size from approximately 4 in² to 81 in² (26 cm² – 523 cm²), and the depth of the newly created spalls was typically only 0.5 in (10 mm) with no steel directly exposed. Coring was performed on the center of the spall in pile P-185-2 and on the center of the top spall in pile P-185-4, revealing in both cases clearly corroded spiral wire immediately below the bottom of the spall. The sixteen piles included also the two piles with significant preexisting spall cases, which showed no directly exposed steel. Coring was performed on the center of one of those spalls (P-110-2-2), revealing steel approximately 0.25 in (6 mm) with an LC surface appearance below. Hammer sounding of the piles with longitudinal cracks (P-4-1 and P-213-4) revealed no delamination, and coring on the cracks showed steel with no visual signs of corrosion.

3.2.2 Steel Condition Observations

Spiral reinforcement steel exposed by coring at most locations in this bridge was rated NC, and the surface of the spiral steel on the core drilled through the delamination/spall at 14 in (0.36 m) AHT on P-110-2-2 was only rated LC. However, the cores on spalled or delaminated areas in piles P-149-2, P-185-4 drilled 26 and 32 in (0.66 and 0.81 m respectively) AHT respectively revealed steel with localized but appreciable corrosion loss of metal. The damage is illustrated in Figure A6 (representative of both cases), showing that as much as 25% of the wire cross section was lost near one end of the wire segment cut out by the core. With the exception of damage near the pile caps noted earlier, these two cases were the only direct observations of severe corrosion in all the field inspections and are denoted by SC in the following. All spacing bars exposed in this bridge, including one next to the SC steel in P-185-4 were rated NC.

Pressurized grout post-tensioned strands have been identified in four core locations (Table A3). The strands exposed from P4-1-4 and P110-2-2 were rated LC (Figure 3.3), and the other two (237-1-2, and 252-1-1) were rated NC. Surrounding grout was clearly visible in all cases strand was exposed. The grout around the NC strand from cores P237-1-2 and P252-1-1 appeared to have partial (< 0.40 in (10 mm)) voids suggesting partial consolidation (Figure 3.4). No

evidence of corrosion was found associated with these voids, but good grout consolidation is an important quality control issue for future construction.

Figure 3.3 Observation of surface rusting on strand, Pensacola Bay Bridge. Faint rust on strand. Traces of rust on grout imprint, indicated by the arrows, shown in the right of the figure. a) P4-1-4. b) P-110-2-2.

Figure 3.4 Pressurized grout consolidation, Pensacola Bay Bridge. Voids indicated by the arrows. a) P237-1-2. b) P252-1-1 (triangular region).

There were signs of other steel (possible spacing rebar) exposed at the bottom of the core hole at three other coring locations, but residual concrete there prevented detailed observation other than noting absence of any signs of rust.

The distribution of measured clear cover of spiral hoop steel is shown in Figure A2, Appendix II. As shown in Table A2 the average concrete clear cover to hoop reinforcement was thin — 2.33 cm which is ~ 20% lower than in the other older cylinder pile bridges investigated. All spiral wire observed was $\frac{1}{4}$ in (0.64 cm) diameter. Clear cover over strand ranged from 1.85 to 2.5 in (4.7 to 6.4 cm).

Exposed spiral reinforcement was found to be electrically continuous to each other. Strand was found not to be continuous with the spiral wire.

3.2.3 Field Measurements of Steel Potential and Concrete Moisture

Half-cell potential measurements were made for exposed steel reinforcement in eight of the piles. Results are shown in Figure 3.5 and are listed in Table A5. Four locations having LC and SC ratings are indicated by black symbols in the figure below: P110-2, P110-2-2, P149-2, and P185-4. Duplicate measurements are shown where available. Notably, those corrosion locations had potentials more negative than -200 mV. While most reinforcement in delaminated or previously spalled concrete showed highly negative potentials suggestive of ongoing active corrosion, half-cell potential in the delaminated/spalled concrete of P252-1 located approximately 3 ft (1 m) AHT, was only -150 mV vs. CSE. This may indicate that the spall/delamination there was not caused by corrosion of the spiral wire/bar or that the spiral wire/bar located 3 ft (1 m) AHT is not continuous with the reinforcements located 1 ft (0.3 m) AHT. However as mentioned above, limited electrical continuity checks from other piles showed that spiral wires severed by coring were electrically continuous to one another.

Figure 3.5 Pensacola Bay Bridge half cell potential measurements. Solid symbols, near corroded reinforcement. Grey symbols, near rust stained/delaminated concrete. Open symbols, no observed distress.

Concrete internal relative humidity measurements were made for P-110-2 at 14 in (35.6 cm) and 5 ft (1.5 m) AHT. IRH measurements were made starting approximately 22 hours after the fittings had been installed, and monitored for approximately 1 hour. Consistent with measurements from the Hathaway Bridge, the IRH is higher within the splash zone than that measured in the atmospheric exposure zone. This is likely due to the higher degree of water saturation near the surface of the tidal zone. Ambient conditions were approximately 65% RH and 18°C.

Figure 3.6 Pensacola Bay Bridge concrete internal relative humidity as function of time since probe insertion, 22 hrs after test fitting installation. • IRH. ■ Temperature. Horizontal line represents ambient humidity.

3.3 Brooks Bridge

The Brooks Bridge approach spans have 36-in (91 cm) diameter cylinder piles with a ten pile bent configuration. The bents are labeled numerically from the south and the piles are labeled from the west. Bents 2 and 10-13 were constructed on shore and were not examined. End bents 1 and 14 at the face of the back wall as well as piers 5 – 8 were not examined. View of bent 9 was limited to the southern face due to boat accessibility. It should be noted that pile bents 3 and 9 were in shallow water. A total of 30 circular piles were examined.

3.3.1 Concrete Observations

Superficial rust staining was observed on Br-3-4 and Br-3-8. Shallow concrete spalling (with no reinforcement exposed) was noticed on two piles in bent 9, consistent with recent FDOT bridge inspection reports. Thin longitudinal cracks were common on many of the piles. Upon closer inspection of Br-3-2, two parallel longitudinal cracks approximately 10 in (25.4 cm) apart from one another were documented. The width of both cracks was approximately 0.008 in (0.2 mm).

3.3.2 Steel Condition Observations

Upon coring, an LC condition was observed on the spiral wire and spacing rebar in Br-3-2-5, located 6 in (15.2 cm) AHT. An NC condition was observed on a spiral wire in Br-3-2-6, located 6 in (15.2 cm) AHT. As shown in Table A2, the concrete clear cover to hoop reinforcement there was approximately 1 – 1.25 in (2.5 – 3.2 cm), comparable to that noted for the Hathaway bridge and consistent with the design drawings for that bridge. The cover for the bar in Br-3-2-5 was 1.5 in (3.8 cm). Strand was not exposed in any of the corings of this bridge. All spiral wire was 0.25 in (0.64 cm) diameter. Direct continuity measurements were not measured

3.3.3 Field Measurements of Steel Potential and Concrete Moisture

Half-cell potential measurements were made at elevations from high tide to 4 ft (1.2 m) AHT as well as below the waterline on piles Br-3-2-5 and Br-3-2-6. Potential measurements were taken for reinforcements exposed by coring from a crack line and the companion core hole 6 in (15.2 cm) horizontally apart on center. Potential mapping of the LC-rated wire and rebar in Br-3-2-5 showed highly negative values characteristic of corrosion activity. Continuity measurements are not available for this spot, but the reinforcements from the cracked concrete location had close potential values indicative of electrical continuity between hoop reinforcements and spacing rebar.

Figure 3.7 Brooks Bridge half-cell potential measurements as function of elevation.

3.4 St. George Island Bridge

The SGI Bridge was newly constructed (2003) at the time of the survey. Approach span bents contain each three 54-in (1.3 m) diameter piles. The bents are numbered from the south direction to the north direction, and the piles are identified by position and location in the pile bent: (E) east pile, (C) center pile, (W) west pile, (TP) test pile, and (N) north pile face, (S) south pile face. Not only is the bridge significantly newer than the other bridges in this study, but the cylinder piles in the bridge reflect current FDOT design specifications including design clear cover on the spiral reinforcement 3 in (7.6 cm) instead of 0.75-1.5 in (1.9-4.4 cm) in the older bridges examined. Also, the concrete mixture proportions included approximately 22% fly ash cement replacement and 8% microsilica cement replacement and a total cementitious content of ~840 pcy (~500 kg/m³) for segments above tide level.

A limited survey of fifteen piles was conducted, twelve of which were being evaluated by FDOT operators for a construction-related assessment independent of the present investigation. Those twelve piles had superficial rust staining or fine longitudinal cracks and only visual appearance is reported here. The remaining three piles, among those earliest erected during construction, were investigated by coring as well. Piles 162-W-S and 163-W-S were selected for closer inspection due to the presence and condition of thin longitudinal cracks measuring less than 0.002 in (0.051 mm) in width. Those piles were driven July

2001 and January 2002, respectively. A test pile, not part of the bridge but standing by itself next to bent 152 was inspected as well. It is thought that the test pile was driven at about the same time as the bent 152 piles (December 2001).

Core samples were extracted to hoop reinforcement depth, which was found at 2.8 – 3.5 in (7.2 -8.9 cm) (see Figure A2), consistent with design specifications. Hoop reinforcement exposed at the bottom of the cores was in all cases in the NC condition. Strand was not exposed during coring. As expected, the exposed hoop reinforcements were electrically continuous. Half cell potential measurements results for the exposed spiral wire are shown in Figure 3.9. It is noted that the core samples extracted from the test pile did not show any physical signs of concrete distress, yet the potential values were generally similar to those on the piles with longitudinal cracks.

Concrete cores from three unused pile cutoff sections labeled 878, 943, and 1404 from the SGI Bridge were retrieved as well.

Figure 3.8 St. George Island Bridge half-cell potential measurements.

4 Laboratory Experiment Findings

4.1 Electrical Resistivity

Resistivity measurements were performed in field-extracted cores to determine the ionic permeability of the concrete. Initial resistivity measurements for cores from the Hathaway Bridge, Pensacola Bay Bridge and SGI cutoffs were contributed by Lan Yao. The tests involved simultaneous determination of moisture pickup (by mass gain) which was informative of the permeation properties of the concrete as well. Recall that internal relative humidity in the piles was typically in the order of only approximately 70%. As shown in figures 4.1, 4.4, 4.7, 4.9, and 4.13 mass gain of most specimens tested in the 100% RH chambers began to show signs of stabilization (to a small value, approximately 1% of original specimen mass) after approximately 100 days of exposure, which was the normal test period. However, a few concrete cores that were exposed for extended periods showed additional mass gain in measurements made approximately 1 year later (figures 4.1, 4.4, and 4.13). The effect probably results from more accessible pores being saturated early, while the less accessible pores become eventually saturated at longer exposure times [29]. The modest and slow water pickup observed is a reflection of the generally high quality of the concretes evaluated.

Resistivity measurements were successfully conducted per the specified procedures in most cases. However, it should be noted that the cores from the Hathaway Bridge and the Pensacola Bay Bridge contained fragmented edges, so geometric corrections [24] were only approximate. Also, several cores extracted from the Hathaway Bridge and Pensacola Bay Bridge had lengths less than 1.5 in (3.81 cm) so the effect of non-uniformities in the aggregate and cement distribution would become more notable with consequent variability in the results. Data from cores shorter and longer than 1.5 in (3.81 cm) are noted in figures 4.2, 4.3, 4.5, and 4.6 which show some differentiation between the results of each class, although the overall trends are essentially the same. It is noted also that since moisture pickup takes place from the specimen surface inwards, the measured resistivity of specimens that are not terminally equilibrated is only an approximate spatial average of outer and inner regions of the specimen.

Figure 4.1 Hathaway Bridge core mass change as function of exposure time to 100% RH.

Figure 4.2 Hathaway Bridge core resistivity measurements as function of exposure time to 100%RH. ▲ Cores > 1.75 inch (4.45 cm). Δ Cores < 1.5 inch (3.81 cm).

Figure 4.3 Hathaway Bridge core apparent wet resistivity as function of mass increase due to water absorption. ▲ Cores > 1.75 inch (4.45 cm). △ Cores < 1.5 inch (3.81 cm).

Hathaway Bridge specimens developed wet resistivity in the order of 100 – 200 kΩ-cm after 70 days and 60 – 100 kΩ-cm after extended exposure of 635 days. Those are high values, characteristic of highly impermeable concrete. Water mass pickup and resistivity varied in inverse proportion as exposure time increased (figure 4.1 and 4.2), as expected. The moisture mass gain was only 1% after 600 days of conditioning in 100% RH which is also indicative of low pore connectivity.

Figure 4.4 Pensacola Bay Bridge core mass change as function to exposure time to 100% RH.

Figure 4.5 Pensacola Bay Bridge core wet resistivity measurements as function to exposure time to 100% RH. $\blacklozenge$ Cores > 1.75 inch (4.45 cm). $\diamond$ Cores < 1.5 inch (3.81 cm).

Figure 4.6 Pensacola Bay Bridge core apparent wet resistivity as function of mass increase due to water absorption. ◆ Cores > 1.75 inch (4.45 cm). ◇ Cores < 1.5 inch (3.81 cm).

The resistivity of some of the Pensacola Bay Bridge (Figure 4.5) cores reached values at 70 days (70 – 100 kΩ-cm) that were comparable to those of the Hathaway Bridge. Five other Pensacola Bay cores show a measured wet resistivity of only approximately 25 – 50 kΩ-cm after 70 days. Four of those cores were short and possibly subject to the test artifacts indicated earlier. Nevertheless, these resistivity values are still quite high for wet concrete and indicative of low permeability. The long term moisture mass gain was only 0.8% – 2%, also suggestive of low interconnected porosity. Mutual water pickup and resistivity trends were as expected (Figures 4.4 – 4.6).

The SGI Bridge cores from the two in-service piles evaluated showed the highest long term resistivity levels of the bridges in the study (approximately 50 to 200 k Ω -cm) and low mass gain (Figures 4.7- 4.11); the as received resistivity values of concrete cores extracted from the SGI Bridge is shown in Figure 4.12. High resistivity is expected from the concrete formulation that included pozzolanic additions intended to further reduce permeability. It is noted that resistivity tended to be higher in specimens for Pile 162 than in those of Pile 163. As it will be shown later the apparent chloride diffusivities for concrete cores from pile 162 were significantly lower than for those from Pile 163, which is as expected as wet resistivity and ion diffusivity are opposite indicators of concrete permeability [30]. Moisture-resistivity trends were as expected and seen for the other bridges (Figure 4.11).

For both piles the resistivity of the core drilled on a crack was noticeably smaller than that of the core on sound concrete (Figure 4.8, Figure 4.10). This behavior indicates enhanced ionic transport through the crack region. It should be noticed that similar enhanced transport may be present in cracks in cores from the other bridges.

Figure 4.7 St. George Island Bridge (Pile 162) core mass change as function of exposure time to 100% RH.

Figure 4.8 St. George Island Bridge (Pile 162) core wet resistivity measurements as function of exposure time to 100% RH.

Figure 4.9 St. George Island Bridge (Pile 163) core mass change as function of exposure time to 100% RH.

Figure 4.10 St. George Island Bridge (Pile 163) core wet resistivity measurements as function of exposure time to 100% RH.

Figure 4.11 St. George Island Bridge core apparent wet resistivity as function of mass increase due to water absorption.

Figure 4.12 St. George Island concrete resistivity measurements on as-extracted cores.

The concrete samples from the SGI Bridge cutoff sections not in service were tested similar to the in-service cores. Three sets of concrete samples were monitored: 878, 943, and 1404. The designations were labels for concrete mixes used in construction of the SGI Bridge. Concrete sample 878 included river rock coarse aggregate. Concrete samples 943 and 1404 had as coarse aggregate primarily limestone, but with some river rock. The core samples extracted from the pile cutoffs were cut into two specimens; one (avg. length 4.8 cm) consisted of all cut edges and the other (avg. length 1.9 cm) retained the original outer surface of the core. The original outer surface may contain a surface layer in which water absorption [29] and ionic diffusion may be slower.

Upon testing, the uncut original concrete surface did not appear to have affected significantly water absorption or wet resistivity, but it should be noted that the perimeter and opposite face of the specimen were cut surfaces. Similar to other bridge core samples conditioned in high humidity, water absorption appears to continue upon extended exposure. Resistivity values were high, consistent with the expected high concrete quality, but as seen in figures 4.13 and 4.14 the 878 mix with river rock had not as high wet resistivity ($\sim 100 \text{ k}\Omega\text{-cm}$) as the 943 and 1404 mixes ($\sim 200 \text{ k}\Omega\text{-cm}$). The 878 mix also showed faster water absorption that compared to the other mixes.

Figure 4.13 St. George Island Bridge pile cutoff mass change as function of exposure time to 100%RH.

Figure 4.14 St. George Island Bridge pile cutoff wet resistivity measurements as function of exposure time to 100%RH.

4.2 Porosity

Volumetric porosity measurements generally following ASTM C642-97 were made for samples from the concrete cores extracted the bridges including from SGI Bridge pile cutoffs. Concrete mix 943 and 1404 had similar results (approximately 8 – 11%); concrete mix 878 showed high porosity in three out of four tests (approximately 13%) as shown in Figure 4.15. As mentioned in Section 4.1, mix 878 (limestone-river rock mix) had markedly lower wet resistivity than mixes 943 and 1404 which would be in agreement with the observed higher porosity. It is noted that the effectiveness of soaking and boiling methods used in ASTM C-642 to saturate high quality concrete has been questioned in the literature [31]. One porosity test result for concrete mix 878 was significantly lower than those of replicate measurements. It is noted that the river rock was intended in the SGI Bridge for concrete mixes for pile sections to be placed below tidal zone.

Volumetric porosity measurements were also made for concrete core fragments extracted from the Pensacola Bay Bridge and Hathaway Bridge (Figure 4.16). The volumetric porosity of the concrete cores from the Pensacola Bay Bridge was approximately 9% in three out of four tests. The porosity of the concrete cores from the Hathaway Bridge was lower (approximately 6-7%). As shown in Section 4.1, the electrical resistivity of the concrete cores from the Hathaway Bridge was generally higher than those compared with the Pensacola

Figure 4.15 St. George Island Bridge pile cutoff concrete porosity. Four replicate measurements, as shown by the bars, were made for each sample: 878, 943, and 1404.

Figure 4.16 Pensacola Bay Bridge and Hathaway Bridge concrete porosity. Replicate measurements shown by bars.

Bay Bridge, in agreement with the difference noticed in porosity. No differences in aggregate type (mostly river rock) or content were noticed; however, since small core fragments (1.75 in (4.4 cm) diameter, approximately 2 in (5.1 cm) length) were used, an unrepresentative proportion of aggregate and mortar may be present. Cores from both bridges contained aggregate as large as 1.25 in (3.2 cm). Low porosity for the approximately 40 year old bridges was consistent with the low water to cement ratio and advanced hydration age of the concrete.

4.3 Chloride Ion Penetration Profiles Observations and Diffusion Parameters

Chloride concentration data from potentiometric titrations of all powdered concrete samples are shown in Table A4 of Appendix III. The average chloride content of each concrete core slice —or milled layer— as well as the idealized profile fit per Eq. (1) (whenever appropriate) of all tested specimens are shown in figures 4.17 to 4.21. Results from sound or cracked concrete locations as well as elevation regions (below high tide, splash and atmospheric) are so noted. Calculated apparent diffusion parameters are given in Tables A3 and A4 of Appendix II. In some instances the datum for the outermost concrete sample of the core was excluded for diffusion parameter calculations as being unrepresentative of bulk transport, or due to suspect analysis methodology; those cases are so identified in Table A4 of Appendix III. In all instances it was assumed that the native chloride ion concentration of the concrete (C_o in Eq. (1)) was 0.08 mg/g (approximately 0.32 pcy (0.2 kg/m³)) for typical concrete with unit

weight of 4,000 pcy (2,400 kg/m³). This value is a rounded off average of values measured at large depths in the SGI Bridge cores in service as well as from piles not in service, and it is also in the order of the average of the two lowest concentration measurements obtained in the Hathaway and Pensacola Bay bridges where several deep cores were extracted. The value is also in the order of typical native chloride values commonly observed in other FDOT investigations of marine substructure. As the cylinder piles are large in diameter and penetration depths very limited, a one-dimensional flat-wall approximation was used instead of a cylindrical geometry treatment in the diffusion models.

Figure 4.17 Hathaway Bridge chloride ion penetration profile. a) H-8-2. b) H-29-3. ▲ Below high tide. ▲ Splash/ tidal zone. △ Atmospheric exposure zone. X Cracked concrete. Lines represent chloride penetration profiles predicted by Eq. 1.

Figure 4.18 Pensacola Bay Bridge chloride ion penetration profile. a) P-4-1. b) P-213-4 c) P-110-2 d) P-185-4 e) P-219-4, P-237-1, and P-252-1 ◆ Splash/ tidal zone. ◇ Atmospheric exposure zone. X Cracked concrete. Lines are chloride penetration profiles predicted by Eq. 1.

Figure 4.19 Brooks Bridge chloride ion penetration profile. ■ Splash/ tidal zone. □ Atmospheric exposure zone. X Cracked concrete. Lines represent chloride penetration profiles predicted by Eq. 1.

Figure 4.20 St. George Island Bridge chloride ion penetration profile. a) 162WS. b) 163WS. ● Splash/ tidal zone. X Cracked concrete. Lines represent chloride penetration profiles predicted by Eq. 1.

Figure 4.21 Salt-solution-ponded concrete chloride ion penetration profile. Nordtest Method NT Build 443. St. George Island Bridge pile cutoffs. Lines represent chloride penetration profiles predicted by Eq. 1.

The most notable feature of the chloride concentration data from piles in service in all four bridges was the very low resulting values of the apparent chloride ion diffusion coefficient. Figures 4.22 and 4.23 show cumulative distributions of the diffusivity and surface concentration obtained for each of the structures assessed as well as for the previous assessment of the Escambia Bay bridges. The overall median value for the four bridges was only approximately $1 \times 10^{-13} \text{ m}^2/\text{s}$ ($\sim 0.005 \text{ in}^2/\text{y}$), which is less than the median value recently obtained for the best performing structures built with modern FDOT class V concretes for aggressive marine service, approximately $4 \times 10^{-13} \text{ m}^2/\text{s}$ ($0.02 \text{ in}^2/\text{y}$) [3]. The range of diffusivity values observed in this study are also in the same class as those obtained for cylinder piles at the Escambia Bay bridges [2], underscoring the overall low permeability of the concrete in these components.

Figure 4.22 Cumulative Fraction of D_{eff} values for cylinder piles in study and Escambia Bay Bridge study [2]

Figure 4.23 Cumulative Fraction of C_s values for cylinder piles in study and Escambia Bay Bridge study [2].

Measurement and model uncertainty are inherent to the process of evaluation of the apparent diffusion parameters obtained here [32]. In addition, there is large actual variability of transport parameters with factors such as elevation, as shown in figures 4.24. and 4.25. Consequently, the differences in the diffusivity and surface concentration cumulative distribution curves (figures 4.22 and 4.23) of the Escambia Bay, Pensacola Bay and Hathaway bridges may not be statistically meaningful. For example, the D values obtained for the Brooks bridge samples are on average 3 times greater than for the other three bridges of similar age, but it should be noted that only 4 samples were evaluated, and all came from a single pile. Consequently, without additional samples from other locations in that bridge it cannot be ascertained whether the concrete used in those piles was more permeable than in the comparable bridges. Of more varied origins were the data for the new SGI Bridge, which include cores from two piles in service (Figure 4.20), to which have been added from laboratory ponding tests of cores extracted from unused pile sections (Figure 4.21). Chloride penetration was shallow as expected for the limited time of service in seawater. Nevertheless, the overall results tended to show values comparable to those obtained in the other, much older structures. It is recalled that concrete with pozzolanic additions tends to show a marked decreasing apparent diffusivity trend with time, typically described by $D=D_0 t^{-n}$, which n is in the order of 0.5 to 0.7 [33]. Such trend reflects the decreasing connectivity of the pore network as the pozzolanic reactions progress over long times. As a result, it is possible that

the median apparent diffusivity of concrete in this structure at age 40 years (approximately one order of magnitude greater than present age) may become several times smaller than at present. As pointed out earlier, SGI Bridge cores extracted from 162WS showed markedly deeper penetration than those from 163WS, even when both piles supposedly were made with the same type concrete with the same mixture proportions. It has been speculated that differences in pozzolanic content in the concrete of both piles may be responsible for the difference, and magnetic susceptibility measurements for determination of fly ash presence were made as discussed in Section 2.4.4. There may also be some segregation of aggregate during concrete pile casting which would also affect the permeability of the concrete at near surface layers. The concrete of the SGI Bridge pile segments above tide level typically comprise of limestone and some river rock. No dramatic differences in aggregate amount or spacing were visually noticeable between 162WS and 163WS; however, it should be noted that the small diameter of the cores precludes drawing definite conclusions when comparing only two samples. Thus, the causes for the observed differences in chloride penetration of both piles remain undetermined.

Figure 4.24 Apparent chloride ion diffusivity as function of pile elevation.

Figure 4.25 Chloride surface concentration as function of pile elevation.

In agreement with a commonly observed trend in marine substructure [3], the apparent diffusivity and surface concentration tended to decrease with increasing elevation AHT (figures 4.24 and 4.25). This behavior is visible in the example of Figure 4.17a where the profiles are significantly wider for cores extracted from below tide than for cores in the atmospheric exposure zone. The composite graph in Figure 4.24 shows diffusivity as function of elevation for all of the core locations in this survey as well as those from the Escambia Bay bridges. Although there is significant local scatter, there is a clearly decreasing trend in D as elevation increases. The decrease is expected as the water content of the concrete decreases with increasing elevation, with consequent reduction in the ion transport ability through the concrete.

As shown in Table 2.1, the typical chloride ion concentration of the water traversed by the bridges in study is approximately 10,000 ppm; St. Andrews Bay traversed by the Hathaway Bridge has a moderately higher chloride concentration (approximately 25,000 ppm). The chloride content of Escambia Bay was documented to be 5,640 ppm [2]. These values are characteristic of the chloride content of gulf and intracoastal water in the Florida Panhandle; St. Andrews Bay is comparable among the most aggressive chloride environments statewide.

Figure 4.26 shows that the ranges of C_s values observed in the various bridges investigated generally overlap, roughly independent of the chloride content of the water in the span of salinity encountered. The similarly overlapping data from an additional statewide sampling of Florida bridge [3] substructures made with conventional concrete is also shown. This behavior has been noted elsewhere [35] and it reflects the development of nearly salt-saturation conditions (and hence approximately equal concentration independent of salinity of the source) on much of the surface of the concrete by evaporative concentration of marine or estuary water.

Figure 4.26 Comparison of water chloride content and concrete chloride surface concentration C_s at elevations -0.5 to 5 ft AHT (-0.14 to 1.52 m) AHT. (-) data from [3] for various FDOT bridges built with conventional modern concrete.

The cylinder piles in the old structures, and especially those in the newer SGI Bridge, had many instances of fine (< 0.013 in at the surface) longitudinal cracks. The measured crack widths of the concrete are tabulated in Table A3 of Appendix III. The cracks often were observed to run along the entire depth of the cores drilled at crack locations, even at SGI Bridge where the concrete cover is thick. It is noted that the thickness of the cracks fluctuated along the core length, and also it is cautioned that the coring process may have disrupted the cracks and possibly created artifacts that alter the observed internal width. While the cracks likely originate from mechanical factors or other phenomena not related to corrosion, it is of much interest to determine whether preferential chloride ingress along the cracks (and consequent localized corrosion initiation) could become important. A recent investigation of FDOT marine substructures built with highly impermeable concrete [3], in marine environments similar in chloride content as the bridges in the current study as shown in Figure 4.22 (approximately 10,000 to 20,000 ppm), showed much enhanced chloride penetration along cracks as narrow as those observed here. Chloride transport modeling for those cases suggested that enhanced transport could be important even in very narrow cracks [3], and other work suggest similar behavior [36]. Examination of the chloride profiles obtained in this investigation showed some limited indications of enhanced chloride penetration, for example the Pensacola Bay Bridge P-4-1-1 and P-4-1-2 cracked/sound concrete pair in Figure 4.18, and the shallow penetration (0.4 in (1 cm)) pair from the SGI Bridge, 162WS1-1 to 162WS1-4

(Figure 4.20). As mentioned earlier, there are indirect resistivity indications of increased ionic transport in cracked specimens in SGI Bridge. These cases however appeared to be much less developed than the dramatic examples of enhanced chloride penetration documented elsewhere for conventional marine substructure [3]. In addition, the present steel corrosion survey failed to show any examples of clear association between prior presence of cracks and preferential corrosion initiation, even in the bridges that had been in service for 40 years. Consequently, there is no clear indication at present that fast chloride transport at thin cracks is a significant corrosion initiation factor in cylinder piles, at least those built with traditional relatively low external clear concrete cover. It is cautioned however that evidence from mechanically distressed piles at SGI Bridge, being investigated independently by FDOT, may require revisiting this issue in the future for the case of new cylinder pile construction using thicker concrete cover. It is noted also that surface rust observations did not appear to be associated with joints between pile segments.

As in most instances the presence of cracks did not appear to significantly alter the shape and depth of the chloride profile compared to that of companion sound cores, the apparent diffusion parameters calculated from both sources have been considered together in the rest of this report.

4.4 Concrete Pore Water pH

ISL measurements of pore water pH were made for samples of in-service piles from the Hathaway and Pensacola bridges. Pore water pH measurements made by Lan Yao for samples from SGI Bridge piles cutoffs that were not in service are considered as well. Results are shown in Figures 4.27 and 4.28. Terminal pH values for the SGI Bridge cutoffs ranged from 12.8 to 13.5. Those values are within the commonly observed range for present day concretes [37], so it appears that the pozzolanic additions used in the SGI Bridge concrete formulation did not result in a pronounced pore water pH drop. Some of the samples the older bridges also had terminal pH values of approximately 13, but most of the cores yielded values between 12 and 13 (and in one instance as low as approximately 11). Experimental artifacts, likely when using small cored specimens of highly impermeable concrete, may account for some of the lowest values observed. The overall modest pH values observed in the older bridge samples are nevertheless intriguing, considering that these older piles were expected to be made using unblended cement, and that pH lowering processes such as carbonation do not penetrate deeply in lower marine substructure elements [19]. It is possible however that the cements used in the older piles had lower alkali content than modern cements. Further assessment of this issue is recommended.

Figure 4.27 Pore water pH of St. George Island Bridge pile cutoffs by ISL technique.

Figure 4.28 Pore water pH by ISL technique. a) Pensacola Bay Bridge. b) Hathaway Bridge

4.5 Magnetic Susceptibility Measurements

Magnetic susceptibility measurements were performed in the SGI Bridge samples as an indirect means to detect presence of fly ash as an admixture, and reveal possible pozzolanic addition shortage in the samples from 163WS which had shown relatively fast chloride penetration. Figure 4.29 shows that the magnetic susceptibility, χ_m , of all the SGI Bridge samples tested (with no clear trend differences between samples from both in-service piles) is of the same order or greater than the typical values encountered for other concretes known to have fly ash addition [3]. Therefore, all the SGI Bridge samples show a positive fly ash indication per this method. In Figure 4.29 the apparent diffusivity measured in the same or in the companion cores is used as a plotting parameter. Interestingly, for the SGI Bridge samples the apparent diffusivity tended to be smaller for the samples with the greatest magnetic susceptibility. However, magnetic susceptibility is only qualitatively representative of fly ash content and the observed trend may only be fortuitous.

Figure 4.29 Magnetic susceptibility indication of fly ash presence and apparent chloride ion diffusivity. ● St. George Island Bridge samples. ○ Laboratory ponded samples. ✕ Cracked concrete. + Concrete samples with known fly ash [3]. X Concrete samples without fly ash [3]. – Concrete samples with unknown presence of fly ash [3]. Vertical line represents susceptibility detection limit.

4.6 Laboratory IRH Measurements

Concrete cores from unused pile cutoffs from the SGI Bridge were used to measure internal relative humidity as indication of concrete permeability. A cylindrical cavity 1 in (2.5 cm) in diameter and 1.5 in (3.8 cm) in length was drilled centered into the flat surface of 3.75 in (9.5 cm) diameter, approximately 3 in (7.6 cm) length, concrete cores for IRH measurements as described in Section 2.2. IRH measurements were initially made storing the core samples in ambient laboratory conditions where the room humidity and temperature was typically in

the range of 60 to 75% RH and 20 to 25°C, respectively for approximately 100 days. The concrete IRH reflected a similar range in humidity level during that time. Figure 4.30 shows the moisture pickup in the core samples from conditioning in a 100% RH chamber as measured by IRH. The concrete IRH only began to show signs of stabilization after approximately 100 to 200 days which is an indication of low concrete permeability. It should be noted that the entire surface area of the concrete core was exposed to conditioning and may experience faster moisture gain than that of actual field conditions.

Figure 4.30 Concrete IRH and temperature as function of time conditioned in ambient conditions and 100% RH. The vertical line represents the time when the samples were placed in high humidity.

5 Durability and Pile Design

5.1 Performance of Cylinder Piles from Older Bridges

The field and laboratory findings are indicative of very good corrosion performance of older cylinder piles over service periods of several decades, confirming similar findings in the previous Escambia Bay bridges investigation⁵. Exceptions were locations such as the upper end of the pile by the pile cap, where mechanical distress during construction had exposed reinforcement that was not sufficiently covered afterwards. Early repair patches at the Pensacola Bridge appear to reflect similar local construction deficiencies. Those cases will not be considered further as they involve detailing control common to most pile types. Conspicuous rust stains at intermediate elevations, notably in the Pensacola Bay Bridge, appear in most cases to have resulted from corroding steel debris or other auxiliary components but not related to corroding reinforcement or strand. In the following, only the condition of the normally constructed and exposed portion of the piles will be addressed.

⁵ A caveat applies in that only the tidal and higher elevation portion of the substructure was surveyed in these bridges. Evidence from other FDOT concrete substructures indicates that most corrosion related damage manifests itself first above water, as evaporative chloride accumulation and oxygen availability is greater there. That trend is assumed to apply here as well but deterioration below the waterline cannot be discounted without direct evidence. Autopsy of the submerged portion of cylinder piles removed from service in the future is recommended to confirm absence of severe damage there.

No telltale rust patterns on the surface of the piles, reflecting the position of spiral reinforcement or strand, were noticed upon examination. Extraction of cores focused on spots selected for showing unusually high rust staining, and yet only two cases of appreciable loss of steel due to corrosion were found in the three bridges investigated. All the other observations of steel corrosion (including some on the post tensioning strand) were of only minor rust or discoloration, some of which may have been present prior to embedment in concrete or grout. Steel potential measurements were equally indicative of a passive steel condition over much of the substructure region investigated. The general evidence of little or no corrosion is striking in view of the aggressiveness of the environment, the four-decade long service life of the structures investigated, and the thin clear steel concrete cover existing (especially in the Pensacola Bay bridge, with <1 in (2.54 cm) cover on average).

The observed high corrosion resistance of these older piles appears to stem from a combination of very slow chloride ion penetration rates through the concrete with either or both of the following: a substantial critical chloride concentration threshold for steel corrosion initiation, and low corrosion propagation rates if the threshold was exceeded. These factors are considered next.

5.1.1 Chloride penetration

As shown in Figure 4.22, the range of surface chloride concentrations observed in the cylinder pile surfaces were similar to those encountered in other marine and estuarial Florida bridges built with conventional concrete. Thus the driving force for chloride penetration in the older cylinder piles was not unusually high or low. The observed chloride diffusivities were however extremely small (typically about 10^{-13} m²/s) which caused chloride penetration to be greatly retarded in the bulk of the concrete. Furthermore, in contrast to observations in conventional FDOT substructures [3], rapid chloride transport through thin cracks was not strongly apparent in the older cylinder piles. Thus, even though thin cracks were frequently observed, inward chloride ion penetration appeared to be a function of bulk parameters.

Low bulk chloride diffusivity was expected in high quality concrete with high cement content, low water to cement ratio and good compaction, as used in the cylinder piles. Both the low bulk diffusivity and the lesser importance of thin cracks may have been promoted by low internal concrete moisture as indicated by the low internal RH levels measured in the field. Low internal moisture can exist even under wet external conditions if the concrete is dense and the low w/c induces self-desiccation upon long term hydration of the cement [38]. Under those conditions ionic transport in the bulk, already slow due to a tight pore network, is made slower by fewer liquid paths within the pores. Transport

through thin cracks is lowered as well, since the presence of liquid along the crack line is equally diminished. This condition is manifested, for example, by the lesser importance of crack transport even in conventional substructure, when cores containing cracks are extracted at higher elevations where the concrete is drier [3].

5.1.2 Critical chloride threshold

Because most corrosion observations involved LC cases, it is uncertain whether sustained active corrosion was actually in progress. Indeed, in some instances the assignment of an LC rating was accompanied by steel potential readings indicative of passive conditions. A highly conservative chloride threshold estimate can be made nevertheless by assuming that LC ratings represent active corrosion in progress, and calculating the value of the chloride concentration at the corresponding steel depth. To that end, the data on the condition of the steel and chloride concentration at the recorded steel depth in Table A3, Appendix III were used to construct the composite cumulative distribution in Figure 5.1. The chloride concentration at the time of the survey (although for the two SC instances corrosion initiation probably took place much earlier) at the steel depth was calculated from the diffusion parameters determined by the core that exposed the steel. For cases where no chloride data or cover was available for that core, estimates were made as detailed in Table A3, Appendix III. Only data for spiral steel and spacing rebar were used for this

graph; conditions of the deeper, grout-surrounded strand are considered separately.

The trend in Figure 5.1 (although affected by large uncertainty from the many assumptions made) suggests that no corrosion initiation took place at concentrations below ~ 0.5 mg/g (or ~ 2 pcy (~ 1.2 kg/m³) for typical 4000 pcy (2400 kg/m³) concrete), and that in many instances the chloride content was higher and not even incipient signs of corrosion took place. Thus a value of 0.5 mg/g may be suggested as a conservative lower bound for the critical chloride threshold of reinforcing steel in these piles, recognizing that the actual effective threshold value (or range of values⁶) is likely to be significantly greater. It is worthwhile noting that corrosion thresholds reported in the literature for steel in conventional concrete but with high cement content similar to that present in the circular piles are in the order of approximately 1 mg/g [41]. Factors such as better concrete compaction in the cylinder piles may result in higher threshold values compared with that in conventional cast concrete.

⁶ Corrosion initiation in concrete, even under controlled conditions, is of a stochastic nature [38]. In addition, corrosion initiation at a given place on a steel assembly may be significantly retarded if the local potential of the passive steel has been driven to more negative values by ongoing corrosion in nearby regions of the same assembly, or by low oxygen availability [39]. Those phenomena may have been responsible for the absence of corrosion in some of the higher chloride concentration regimes.

Figure 5.1 Cumulative fraction of estimated chloride concentration at steel depth determined for locations where spiral or spacing bar steel was exposed by coring in the Hathaway, Pensacola, and Brooks bridges. See Table A3, Appendix III for details and assumptions. Solid symbols, SC. Grey symbols, LC. Open symbols, NC. X, core drilled on crack.

The trend in Figure 5.1 suggests that corrosion initiation was just beginning to affect a small fraction of the pile inventory in these bridges at the time of the survey. This is consistent with a combination of normal chloride surface concentrations, very low diffusivities and considerable threshold levels. By application of Eq. (1) this combination results in long projected times to corrosion initiation, even though the observed clear concrete covers were so thin. For example, one may assume values representative of the present findings: $C_s=10$ mg/g, near zero native chloride, $D=10^{-13}$ m²/sec and $x_c=2.5$ cm, together with $C_T=1$ mg/g. Eq.(1) then projects a time to corrosion initiation of 37 years. A higher threshold value would extend the projected time to initiation accordingly.

The above discussion applies to spiral or rebar steel. Little can be said from the present findings about the effective threshold for strand steel as in most cases the projected chloride penetration to the strand depth was very small, and the few instances of rust on strand wires are not clearly related to ongoing corrosion. In the absence of other information, it will be assumed that the corrosion threshold for properly grouted strand is comparable to that of the other steel in these structures, with consequent expectation of much longer times to corrosion initiation due to the deeper clear cover of the strand. A potential concern is the observation of two cases of clearly visible grout voids (core P237-1-2 and P252-1-1), which may locally lower the corrosion threshold. Control of grouting quality is essential.

5.1.3 Corrosion propagation rates

From the previous section and the generally low external manifestations of corrosion, it can be said that corrosion propagation (topical extraneous damage excluded) has either not commenced in most of the piles, or is proceeding at a very low rate. The steel exposed by coring that most commonly showed some signs of corrosion (LC condition) had very small average corrosion penetration, perhaps only a few micrometers. If that corrosion actually occurred in service, the previous discussion suggests that initiation took place relatively recently, so typical corrosion propagation rates should be in the micrometer per year range (barely above usual estimates of passive corrosion rates [42]). This modest

range estimate is not surprising considering that the concrete in the piles, even when wet, had high resistivity (reflecting low pore connectivity and often moderate internal humidity), of a level commonly associated with very low or negligible corrosion rates [28].

The corresponding time necessary for a corrosion-induced crack to occur may be evaluated using the expression by Torres-Acosta [43].

$$x_{\text{CRIT}} (\text{mm}) \sim 0.011 \left(\frac{x_c}{\Phi} \right) \left(\frac{x_c}{L} + 1 \right)^{1.8} \quad (2)$$

where x_{CRIT} is the steel corrosion penetration necessary to cause a crack in the concrete, x_c is the concrete cover thickness, Φ is the steel bar diameter, and L is the corroding length. Assuming conservatively that L is large compared to x_c , and taking representative dimensions for the spiral wire in the old bridges ($\Phi = 6$ mm, $x_c = 30$ mm) the result is $x_{\text{CRIT}} \sim 0.055$ mm. Corrosion is likely to be more localized during the early initiation events. In such case if e.g $L \sim x_c$, then x_{CRIT} would be twice as large. With corrosion rates in the micrometer per year range [3], the typical time required to develop a corrosion induced crack after the initiation event would be two or more decades.

After the first corrosion induced cracks take place, the corrosion rate is likely to increase so subsequent deterioration would be faster and concrete

delamination would quickly ensue. The two recorded incidents of severe wire corrosion may reflect such occurrence. In one of those cases the estimated clear cover was only 1.4 cm, and corrosion propagation may have been in progress for many years already. It is noted that even in those cases, the actual metal loss was quite localized and much of each wire segment extracted retained the initial dimensions.

The longitudinal steel strands in these structures are placed about 33% to 50% deeper than the spiral wires or spacing bars. All other factors being equal, the time necessary to reach threshold chloride levels at reinforcement depth is approximately proportional to the square of the cover (Eq.1). Corrosion of the strand is then expected to start much later in the life of the structure. Thus, any future corrosion-related severe damage to the concrete, and related repair needs, are likely to result first from the corrosion of the spiral reinforcement.

5.1.4 Corrosion-related durability prognosis

A small fraction of piles have already shown indications of corrosion damage. The factors considered above indicate that incipient indications of concrete damage from corroding reinforcement beyond those observed may already exist in a small fraction of the piles, and might become apparent upon extensive re-examination of the structures. However, threshold values may span a wide range above the value tentatively identified above, and evolution of the

corrosion propagation stage is expected to be slow. Thus, development of corrosion damage serious enough to require repair or rehabilitation appears to be several decades into the future. There is then a good chance that, despite their low concrete clear cover, the cylinder piles bridges built in the 1960's and still commissioned will perform with little corrosion-related maintenance needs well into a 75-year service life span. The quantitative durability treatment developed earlier for the Escambia Bay bridges [3] reached similar conclusions using assumptions for the initiation and propagation stage that were approximately equivalent to the findings from the present findings. Similar forecast models for the Hathaway, Pensacola Bay, and Brooks bridges are discussed in Section 6.

5.2 Performance of Cylinder Piles Built Under Current FDOT Guidelines

The very early information developed to date for the piles at the SGI Bridge indicates, on average, excellent bulk concrete properties to slow down chloride ingress. Attention is called to the somewhat higher (but still quite small in absolute terms) diffusivity encountered in one of the piles tested; its chloride penetration behavior should be monitored in the future and testing extended to other piles to assess any possible systematic deficiency. Nevertheless, because of the high pozzolanic content of the concrete, bulk chloride transport is expected to become even slower in the future. Surface chloride concentration levels were indicative of no unusual driving force for chloride penetration.

The cylinder piles in the SGI Bridge section investigated showed frequent thin cracks (not caused by corrosion), in some instances propagating deep into the reinforcement level. It is too early to assess the effect of those cracks on chloride transport, except to note that preferential chloride ingress was observed in two instances in the shallow range (approximately 0.5 in (1 cm)) where chloride profiles were measurable. It is possible however that the same conditions that seem responsible for mitigating chloride crack transport in the older bridge piles will be active here as well. Future evaluation as the structure ages is important to better assess this issue.

Chloride threshold cannot be directly evaluated at present, as no reinforcement corrosion cases were documented (rust spots seen on the surface of some SGI Bridge piles appear to be related to steel debris or other extraneous sources). However, the pore water assessment indicated a high pH in spite of the high pozzolanic content. Thus, it appears likely that normal chloride threshold values may apply. Corrosion propagation would be expected to be limited in these piles by the same factors prevalent in the older bridges, except that the thicker concrete cover may further extend the time to corrosion induced cracking if the cover to steel diameter is greater. As in the older bridges, most of the corrosion-induced damage to the concrete is expected to originate from the spiral reinforcement before any strand corrosion becomes important.

5.3 Concrete Cover and Cylinder Pile Corrosion Performance

If bulk transport of chlorides were the only relevant factor, the corrosion limited durability of the modern cylinder piles would be expected to be exceedingly long. As indicated earlier, projected corrosion initiation times per Eq. (1) are proportional to the square of the clear cover. The average of the cover from the three older bridges (Table A2) is 1.17 in (2.96 cm), while that of the SGI Bridge is 3.18 in (8.1 cm). Thus, if as suggested by the present findings the chloride diffusivity and corrosion threshold are comparable, the length of the corrosion initiation stage in the newer piles would be approximately 7 times greater than for those in the older bridges. Nominal durability far in excess of 100 years would be then projected.

The above simplistic analysis does not take into consideration the potential for localized chloride ingress paths, notably preexisting cracks as those noted earlier. Because of the much thicker clear cover, both the propensity for any cracks and the width of the cracks may be significantly greater than in the older cylinder pile design⁷. The use of microsilica in newer piles may also increase the chances of cracking due to less tolerance to curing anomalies [44]. As indicated in Section 5.2, the extent of enhanced chloride penetration can not be assessed well at this early pile age. Thus, the possibility of severe chloride

⁷ It is noted however that cracking at the piles in the investigated portion of SGI is generally related to other construction issues, examined separately by FDOT.

penetration through preexisting cracks, as noted in other FDOT structures with otherwise highly impermeable concrete, cannot be disregarded and continuing monitoring of those locations is needed.

The above findings suggest that, from a bulk chloride penetration standpoint, cylinder piles constructed with the materials examined here could have clear cover levels significantly less than the currently specified 3 in (7.5 cm) and still result in times to corrosion initiation amply consistent with a 75-year durability goal. In addition to lowered material requirements and fewer fabrication constraints, a smaller cover might reduce the propensity for incidence of cracks. Improved reliability may result as well as fabrication methods would approach established practice that has resulted in the good corrosion performance documented here for the older piles. This expectation is supported by the analysis in Section 6 where the consequences of a hypothetical relaxation of clear cover to a smaller value (2 in (5.1 cm)) as an example are considered in more detail by means of a simplified quantitative model.

Present FDOT Structure Design Guidelines (Section 4.5, Post Tensioning) establish multiple levels of protection for critical bridge components which may be extended to cylinder piles as well. Of special interest is the condition of the tendons at the joint between pile segments. At present protection to the strand there is provided by two consecutive elements: the grout around the strand, and

the parallel combination of the concrete at both sides of the joint and the epoxy or similar filler material placed between the two joined segment faces. A reduction in design cover as considered above would also reduce the length of the thin polymer-filled region between the segment faces possibly facilitating chloride ingress at the joint. Special provision for added quality control or supplemental protection there, as well as dimensioning piles to avoid joint placement in the tidal or splash zones, may merit consideration. A possible enhancement being considered by FDOT is the installation of a consecutive protection element provided by a corrugated polymeric duct cast in the grout channel. The mechanical consequences of such addition need examination for possible disadvantages; placement of similar ducts only near the segment ends may be a desirable alternative with fewer potential drawbacks if consistent with practical implementation. It is noted that no particular association between pile joints (where the joint cover is, as elsewhere, very thin) and evidence of corrosion was noted in the survey of the three older bridges.

6 Deterioration Forecast Model

This section presents an exploratory analysis of the corrosion performance of cylinder piles using deterioration models based on simple Fickian diffusion of chloride and corrosion initiation/propagation concepts⁸. The concepts discussed at the end of Section 5 are examined in more detail, with attention to the consequences of a hypothetical relaxation of current specified clear cover.

The deterioration model used considers the region of the concrete surface where corrosion distress may occur (typically in the tidal region and splash zones), discretized into small surface elements, with independent distributions of concrete properties. The modeling approach, used first for the Escambia Bay bridges, is described elsewhere along with a discussion of the model limitation issues such as time and spatial variant diffusivity, chloride binding, and polarization effects [2,45]. The model inputs are chloride threshold C_T ; distributions of surface chloride concentration C_s , chloride ion diffusivity D , and concrete clear cover X_c as mean and standard deviation values⁹; corrosion

⁸ These analyses are not part of the FDOT report "Corrosion Performance of Concrete Cylinder Piles" BC353 RPWO#10 (2005) 605 Suwannee St. MS30 Tallahassee, FL 32399 and are introduced here as an additional contribution to obtain insight of concrete cylinder pile corrosion performance projections.

⁹ Some of these distributions are truncated at zero or at some representative value (Table 6.1). The terms mean and standard deviation as used in the following indicate the values of those parameters before applying truncation.

propagation time t_p ; and the number of pile surface elements assumed to be susceptible to distress. Calculations using Eq. 1 determine the time of corrosion initiation for each surface element and add the length of the propagation period to obtain the service time at which the element is declared to be distressed (e.g. displaying a corrosion induced crack or spall). The model output is a deterioration function given as the damaged fraction (number of damaged elements divided by total number of elements) of the pile surface area susceptible to damage as function of time. The service life projected by the model may be significantly shorter than that calculated solely with mean values of D , C_s , and X_c . That difference results from having assumed a relatively wide distribution of parameter values (Table 6.1), so early deterioration is ruled by the extreme values of those parameters.

Forecast model input values are listed in Table 6.1. A surface element size of 25 in^2 (0.016 m^2), representative of the size of the corrosion related spalls and severe rust spots observed in the field, was used. The model output is reported as the deteriorated fraction of the susceptible pile surface. Deterioration forecast of the three older bridges has been evaluated together as one generic case. Two other cases evaluated the deterioration forecast of the SGI Bridge with actual observed clear cover and a hypothetical reduced specification clear cover. Distributions of concrete clear cover, effective surface chloride concentration and effective chloride ion diffusivity have been abstracted from analysis of field and laboratory data or from trends identified in earlier studies [2].

Table 6.1 Forecast Model Inputs

	Older Bridges Case	SGI Bridge Cases	
Elevation range (m)	1.5	1.5	
Mean chloride ion diffusivity D	1.18e-13*	1.18e-13*	
Standard deviation σ	9.84e-14**	9.84e-14**	
Min. D truncation (m ² /s)	0	0	
Mean Cl ⁻ surface concentration Cs	20	20	
Standard deviation σ	6.0	6.0	
Min. Cs truncation	0	0	
Max. Cs truncation (kg/m ³)	40	40	
Mean clear cover Xc	2.96	8.1	5.1 [†]
Standard deviation σ	0.43	0.43	0.43
Min. Xc truncation (cm)	1.0	7.2	5.0 [‡]

* Average value from Hathaway and Pensacola Bay bridges.

** Assumed values from trend in earlier study [2], $\sigma \sim 0.5$ mean.

† Example reduced specification clear cover.

‡ Assuming strict control of clear cover.

The average calculated diffusivity of the concrete from the Hathaway and Pensacola Bay bridges was used as the mean value for the older bridges case. For the SGI Bridge, as shown in Figure 4.22, the median chloride ion diffusivity is approximately two times greater than that of the Hathaway and Pensacola Bay bridges. Since the concrete in the SGI Bridge contains pozzolanic materials, the

chloride ion diffusivity is expected become lower with time as the pozzolanic materials continue to hydrate. A diffusivity value representative of the older bridges was chosen for model calculations in the SGI Bridge cases. A standard deviation value σ for chloride ion diffusivity of one-half of the mean value was assumed by analogy with observations in the Escambia Bay study [2]. This was done instead of basing that parameter on the actual observations, since there were discrepancies within the small population of field samples, particularly from in the SGI Bridge samples (as indicated in Section 4.3).

Deterioration forecast model outputs for the older bridges case are shown in Figure 6.1. The figure also includes bounded ranges of values for the damage estimated to be present in each bridge at the time of the survey. The actual corrosion related damage at the time of inspection is not exactly known as the surveys were not comprehensive and the cause of each observed deteriorated area could not be fully established. However, the damage was bounded by an upper limit where each observed rust spot is considered to represent one distressed element — even though the vast majority of those spots may be due to steel exposed by mechanical damage or superficial corrosion not associated with reinforcing or post tensioned steel. A lower bound limit is the amount of corrosion-induced damage confirmed by direct observations (not related to mechanical damage or other preexisting circumstances) in the bridge. In the case of the Brooks Bridge no damage fell in that category, so a lower limit was

assigned as corresponding to at least one distressed element that may have escaped detection during the survey.

Figure 6.1 Older bridges forecast deterioration model output. The vertical line represents the bounded range of observed damage at the time of the bridge survey. The dashed line corresponds to a 0.01 and 0.1 deterioration area fraction for nominal definitions of service life.

Although no continuous distributions were used for chloride threshold C_T and length of the corrosion propagation stage t_p , the calculations include damage projections with alternative combinations of those parameters. These parameters were used to fit the projected deterioration to field deterioration observations from the Hathaway, Pensacola Bay, and Brooks bridges at the time of survey.

The chloride threshold values chosen consisted of two values: a low value corresponding to the lower bound estimated in Section 5.1.2 (1.2 kg/m³) and an

upper value where high chloride content was measured (13.5 kg/m^3) in a field sample but where there was no corrosion distress of the reinforcing steel (Figure 5.1). High chloride threshold values may be possible considering that the concrete has a high cement factor, and in the cases of the older bridges in this study, an absence of pozzolanic materials [46]. Polarization effects are another factor which may cause higher chloride threshold values. Still passive portions of the steel reinforcement may be cathodically prevented from corrosion initiation by galvanic coupling with an active corrosion site and in which case higher C_T values may be possible [40,47]. Chloride threshold values as large as 17 kg/m^3 have been reported for laboratory concrete samples [40,48]

Section 5.1.3 indicates that the propagation period could easily stretch into two or more decades. Long corrosion propagation times may be due to limited transport of oxygen to the steel surface, high concrete resistivity or small reinforcing wire diameter. With that in mind, nominal propagation period durations of 12 years, 24 years, and 48 years (assumed as fixed values [45]) were chosen.

Figure 6.1 shows the projections resulting from the various combinations (labeled A through F) of the alternative parameter values chosen. Combination A, having the lowest t_p and C_T , over-projects the damage at the time of survey and is too conservative. Combination F, with the opposite extremes of t_p and C_T ,

is too optimistic. Therefore, when using single values of t_p and C_T , results are better represented by intermediate combinations such as B to E. It is noted that a range of chloride threshold values and propagation times for the surface elements is likely to be a more realistic representation of field conditions than singly assumed C_T and t_p values. Because of the cathodic prevention effects mentioned earlier [40], lower C_T values are expected to be prevalent earlier in the life of the bridges, while higher values may exist near the end. Thus, combinations B-C (or even A) may better describe earlier behavior while D-F may apply to the later life stages. If low C_T values dominate earlier behavior, then propagation times that are short or very short (e.g. 5 years, as often assumed for other marine substructure or deicing salt service cases [3]) do not seem likely as the resulting projected damage would greatly over-predict observations. This suggests that the corrosion durability of concrete cylinder piles benefits from long propagation times (>12 years assuming $1.2 \text{ kg/m}^3 < C_T < 13.5 \text{ kg/m}^3$). The projections also suggest that long service times are achievable.

The service life of a bridge can be nominally defined as the time when a given fraction (e.g. 1% or 10% (0.01 or 0.1)) of the pile surface elements is projected to be distressed. Values of 1% to 10% correspond to the appearance of about 4 to 40 damaged elements (having a combined area of approximately 100 to 1000 in^2 , respectively, which is in the order of a typical repair patch [2]) on each pile of the bridge. Such level of distress may not represent severe

structural damage but could require extensive maintenance repairs with considerable cost. The level chosen, however, is only used as an example and the calculations can be performed for any other situation reflecting appropriate economic factors. Note that in Figure 6.1 this nominal service life range is reached at various times depending on the parameter combinations assumed. However, if the actual performance falls somewhere in between those projected for B-C and D-E (as suggested above), the projected times to reach 1-10% damage may be some decades into the future, consistent with the generally good present condition of the bridges. Under the most likely assumed parameter combinations, it appears quite possible that these structures will achieve a 75 year service life without excessive corrosion damage.

The deterioration forecast model output for the SGI Bridge is shown in Figure 6.2. Projected damage is considerably delayed compared to that in Figure 6.1 for the older bridges. Corrosion initiation times are projected to be large (>200 years assuming mean values of D , C_s , and X_c) because of the low assumed chloride ion diffusivity (same as that of the older bridges taking into consideration beneficial aging of the concrete) combined with the much larger measured concrete cover (3.2 in (8.1 cm)) in SGI. The projected corrosion related service life times for the scenarios considered in Figure 6.2 extend into the far future.

Figure 6.2 St. George Island Bridge forecast deterioration model output, $X_c=3.2$ in (8.1 cm). The dashed line corresponds to a 0.01 and 0.1 deterioration area fraction for nominal definitions of service life.

Following on the discussion at the end of Section 5.3, the model was applied to tentatively evaluate the consequences of relaxing the specification of concrete clear cover to spiral reinforcement to 2 in (5.1 cm). This clear cover value is approximately 1.73 times greater than the average 1.17 in (2.96 cm) cover of the three older bridges. From the form of Eq. 1, all other things being equal, the proposed cover value would result in a nominal increase in time to corrosion initiation by a factor of 3 over the case of the older bridges. The corresponding deterioration model output for the SGI Bridge is shown in Figure 6.3. For this hypothetical case, the standard deviation of the concrete clear cover was assumed to be same as for the other bridges. A strict minimum clear cover of 1.97 in (5 cm) was also assumed for the model. Figure 6.3 shows that even in

this reduced clear cover case the projected performance is still considerably improved over that for the older structures, and that there is good prognosis for meeting a 75 year service life goal under likely parameter combinations.

Figure 6.3 St. George Island Bridge forecast deterioration model output, $X_c = 2$ in (5.1cm). The dashed line corresponds to a 0.01 and 0.1 deterioration area fraction for nominal definitions of service life.

It is cautioned that the model projections are subjected to large uncertainty, as evidenced by the broad range of possible alternative outcomes shown in figures 6.1 to 6.3 and as detailed in Ref. [2,45]. Thus, the model outputs should be viewed as an aid in examining relative consequences of possible alternative design scenarios and not as the outcome of absolute prediction tools. With that caveat, the results are encouraging of further consideration of the adoption of a relaxed cover guideline for this type of pile.

That consideration should be made in the context of the other structural and economical issues determining performance of concrete cylinder piles.

The previous calculations are exploratory in nature. Future analysis of the corrosion performance of concrete cylinder piles should include consideration of the increase of C_T of still passive regions of reinforcing steel induced by prior corrosion of nearby steel [47]. Other modeling improvements should include quantifying oxygen transport through the clear cover as a determining factor of the corrosion propagation time and the possible effect of local concrete deficiencies (e.g. cracks) on accelerating onset of corrosion.

7 Conclusions

1. Examination of three 40 year old cylinder pile bridges (Hathaway, Pensacola Bay, and Brooks) indicated in general minor or no corrosion distress of the spiral reinforcement or strand in the piles, in spite of small clear concrete cover values of ~ 0.4 to 1.5 in (10 to 39 mm). Some corrosion noted near pile caps as well, or the presence of topical patches, appear to reflect early mechanical damage unrelated to normal exposure. Similar good corrosion performance was found in a previous FDOT examination of the Escambia Bay bridges, also built with cylinder piles.
2. The rate of chloride ion penetration in the 40-year old bridges, as indicated by the apparent diffusion coefficient, had a median value of only ~ 0.005 in^2/y ($\sim 1 \times 10^{-13} \text{ m}^2/\text{s}$), lower than that of the best performing modern FDOT class V concretes specified for aggressive marine service. High electrical resistivity and moderate internal humidity of the concrete were consistent with the slow chloride transport observed. These properties appear to have resulted from high cement content, low water to cement ratio, and good compaction inherent to the pile fabrication process.

3. Thin (< 0.013 in (< 0.3 mm)) concrete cracks were observed in some of the piles in the 40-year old bridges. However, unlike behavior recently noted in other FDOT bridges with conventional concrete substructure, preferential chloride penetration along the crack was less pronounced.
4. Tests of the early performance (~ 2 years) of new cylinder piles built to current FDOT guidelines (thicker cover, pozzolanic admixtures) for the recently constructed Saint George Island Bridge also show excellent average resistance to chloride penetration. No reinforcement corrosion was identified at this early bridge age. Thin cracks were observed here as well, together with evidence of enhanced chloride penetration at shallow depths.
5. The survey information from the older bridges was used to estimate a lower bound value for the chloride threshold for corrosion initiation in the order of ~ 2 pcy (1.2 kg/m^3). This bound is consistent with the reported high cement content of the concrete and the observation of pH values in extracted cores approaching normally expected values. Effective threshold values could be significantly higher. The concrete in the new bridge piles had a desirably high pore water pH despite its high pozzolanic content, suggesting that chloride threshold values in the new material will be in the normal range.

6. Simplified corrosion damage forecasts for the older and new piles suggest that very long service lives with minimum corrosion-related maintenance are possible. The projections suggest also that if concrete quality could be sufficiently assured, moderate relaxation of absolute minimum cover requirements for new construction could be made without severely compromising the requirements for minimum corrosion damage within a 75-year service life goal.

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Appendices

Appendix I Bridge Survey

Table A1a Hathaway Bridge Survey 12/17/02

Bent	General Remarks	Inspection Notes	Potential	Resistivity	Continuity	IRH	Core Extraction
1	Abutment. Begin bridge.						
2	Begin pile bent. Four 54" cylinder piles.						
3	H-3-2 rust stain near top of pile.						
5	H-5-3 rust stain near top of pile. H-5-4 repair patch near top of pile.						
6	H-6-2 rust stain at top of pile. H-6-3 rust stain.						
7	H-7-2 minor rust stain near top of pile. H-7-4 rust stain near top of pile.						
8	H-8-2 rust stain in splash zone.	H-8-2 rust stain on east side of pile in splash zone. Removed delaminated concrete, but hoop wire was not observed.	H-8-2	H-8-2	H-8-2	H-8-2	H-8-2
9	H-9-1 crack near top of pile. H-9-4 rust stain at intermediate elevation.						
11	H-11-1 delaminated concrete.						
13	H-13-4 rust stain near top of pile.						
14	H-14-1 repair patch near top of pile.						
15-28	column piers	H-16 rust stain on footer. H-27 rust stain on footer. H-28 severe rust staining on footer.					
29	Pile bent. Four 54" cylinder piles. H-29-3 crack.	H-29-3 crack 0.3mm width.	H-29-3	H-29-3	H-29-3	H-29-3	H-29-3
30	H-30-3 crack. H-30-4 rust stain in splash zone.						
31	H-31-2 spall in tidal zone. H-31-3 spall at intermediate elevation.						
32	H-32-2 rust stain at intermediate elevation.						
33	H-33-1 crack repair. H-33-3 rust stain in splash zone. H-33-4 rust stain in splash zone.						

Appendix I (continued)

Table A1b Pensacola Bay Bridge Survey 3/10/03–3/11/03

	General remarks	Inspection Notes	Potential	Resistivity	Continuity	IRH	Core Extraction
1	Abutment. Begin Bridge.						
2	Begin pile bent. Four 54" piles. P-2-2 rust stain on west face. P-2-3 crack below pile cap.						
3	P-3-1 crack below pile cap. P-3-2 rust stain on east face above tidal zone.						
4	P-4-1 crack. P-4-3 crack below pile cap.	P-4-1 vertical thin crack 0.3mm width.	P-4-1		P-4-1		P-4-1
5	P-5-1 rust stain on north face above tidal zone. P-5-3 rust stain on east face at tidal zone.	P-5-3 large delaminated area ~63in ² .					
6	P-6-1 rust stain on west face. P-6-3 rust stain on north face. P-6-4 rust stain on north face at tidal zone.						
8	P-8-1 rust stain on northeast face. P-8-3 rust stain on north face. P-8-4 rust stain on west face at tidal zone.						
9	P-9-1 crack on north face. P-9-4 rust stain on south face.						
10	P-10-1 rust stain on south face						
12	P-12-2 rust stain on west and east face.						
13	P-13-2 crack on south face.						
14	P-14-3 rust stain on south face. P-14-4 rust stain on east face.						
15	P-15-2 rust stain on south face.						
16	Tower bent. Six 54" piles. P-16-2 rust stain on south face. P-16-3 rust stain on west face. P-16-6						
18	P-18-1 rust stain on west face. P-18-2 rust stain on west face.						
19	P-19-2 rust stain on south face. P-19-3 rust stain on west face. P-19-4 rust stain on north face.						

Appendix I (continued)

Table A1 b (continued)

Bent	General Remarks	Inspection Notes	Potential	Resistivity	Continuity	IRH	Core Extraction
21	P-21-1 possibly crack. P-21-3 rust stain on west face.						
23	P-23-1 rust stain on east face.						
24	P-24-1 rust stain on northeast and southeast face.	P-24-1 delaminated concrete removed ~4in ² .					
25	P-25-3 rust stain on south face. P-25-4 rust stain on north face.						
26	P-26-1 rust stain on north face.						
27	P-27-3 rust stain on east face. P-27-4 rust stain on west face.						
29	P-29-2 rust stain on east face.						
30	P-30-3 crack near pile cap. P-30-4 rust stain on west face						
31	Tower bent.						
32	P-32-1 rust stain on south face.						
33	P-33-3 rust stain on west face.						
34	P-34-4 rust stain on south face below pile cap.						
35	P-35-1 rust stain on east face.						
37	P-37-1 rust stain on south face.						
38	P-38-2 rust stain on north face.						
40	P-40-1 rust stain on west face.						
42	P-42-1 rust stain on west face.						
43	P-43-1 rust stain on east face.						
46	Tower bent. P-46-1 rust stain on east and west face.						
48	P-48-4 rust stain on south face.						
50	P-50-3 rust stain on W face.						
51	P-51-1 rust stain on east face. P-51-2 rust stain on NE face at tidal zone.						
52	P-52-2 rust stain on N face at tidal zone.						

Appendix I (continued)

Table A1 b (continued)

Bent	General remarks	Inspection Notes	Potential	Resistivity	Continuity	IRH	Core Extraction
53	P-53-1 rust stain on NE and S faces.						
55	P-55-2 rust stain on W face at tidal zone. P-55-4 rust stain on W face.						
56	P-56-1 rust stain on E and W face.						
57	P-57-4 rust stain on S face.						
58	P-58-1 rust stain on S face.						
59	P-59-3 rust stain on S face. P-59-4 rust stain on W face.						
60	P-60-2 rust stain on west face.						
61	Tower bent.						
63	P-63-1 rust stain on E face near top.						
64	P-64-2 crack near pile top.						
65	P-65-1 rust stain on E face.						
66	P-66-1 rust stain on E face.						
67	P-67-2 crack near pile top.						
69	P-69-3 rust stain on NW face.						
71	P-70-3 rust stain on NE face,						
74	P-74-4 crack.						
76	Tower bent. P-76-4 rust stain on N face.						
77	P-77-1 rust stain on E face.						
79	P-79-1 severe rusting on E face.	P-79-1 rust stain near pile cap. No indication of delamination.					
80	P-80-4 pop-out on E face.						
81	P-81-4 rust stain on E face at tidal zone.						
83	P-83-1 rust stain on E face near pile top.						
84	P-84-1 rust stain on N face. P-84-3 rust stain on W face. P-84-4 rust stain W face.						
86	P-86-4 rust stain on W face near top.						

Appendix I (continued)

Table A1 b (continued)

Bent	General Remarks	Inspection Notes	Potential	Resistivity	Continuity	IRH	Core Extraction
87	P-87-4 pop-out on E face.						
88	P-88-3 rust stain on W face.						
91	Tower bent.						
94	P-94-2 popout. P-94-4 crack.						
96	P-96-2 rust stain on N face.						
98	P-98-3 rust stain on W face near top.						
99	P-99-1 rust stain on E face near top. P-99-3 rust stain on W face near top.						
100	P-100-1 popout on S face.						
101	P-101-1 rust stain on E face near top. P-101-2 rust stain on SE face near top.						
102	P-102-2 rust stain on S face near top.						
103	P-103-1 rust stain on E face near top.						
104	P-104-1 rust stain on E face near top.						
106	Tower bent. P-106-4 rust stain on SW face by top.						
107	P-107-1 rust stain on east face. P-107-2 rust stain on S face. P-107-3 rust stain on N face.						
108	P-108-2 crack near top of pile.						
109	P-109-1 rust stain on E face. P-109-4 rust stain on N and W face.						
110	P-110-2 rust stain on E face.	P-110-2 spiral wire corroded in rust stain 14"AHT	P110-2			P110-2	P110-2
112-145	Column piers.						
146	P-146-4 rust stain at W face in tidal zone						
147	P-147-2 rust stain on W face near top.						
148	P-148-1 rust stain on E face in tidal zone. P-148-3 repair patch on W face.						
149	P-149-1 rust stain on E and S face. P-149-2 rust stain on S face. P-149-3 rust stain on S face.	P-149-2 two large delam, ~30 and 42 in ² . Wire appears corroded.	P149-2				P149-2

Appendix I (continued)

Table A1 b (continued)

Bent	General Remarks	Inspection Notes	Potential	Resistivity	Continuity	IRH	Core Extraction
150	P-150-1 rust stain on E face in intermed. Elevation. P-150-3 rust stain in W face.						
151	Tower bent. P-152-1 rust stain on W face. P-152-6 rust stain on S face.						
152	P-152-1 rust stain on W face. P-152-2 repair patch on SW face.						
153	P-153-2 repair patch on N face, rust stain on W face.						
154	P-154-1 repair patch on E face. P-154-2 repair patch on E face. P-154-4 repair patch on W face.						
155	P-155-1 repair patch on E face.						
156	P-156-1 repair patch on E face intermediate elevation, rust stain on S face near pile top. P-156-2 rust stain on N face.						
157	P-157-1 rust stain on N and W face. P-157-3 repair patch on S face.						
158	P-158-1 rust stain on N face. Delaminated concrete removed ~12in ² .						
160	P-160-1 rust stain on E face. P-160-2 rust stain on W face. P-160-4 repair patch on W face.						
161	P-161-1 rust stain on S face at tidal zone. P-161-3 rust stain on S face near top.						
162	P-162-3 repair patch. P-162-4 repair patch on W face.						
163	P-163-1 repair patch on E face near top. P-163-3 repair patch. P-163-4 crack.						
164	P-164-1 repair patch on N face, rust stain on E face. P-164-2 rust stain on S face. P-164-4 repair patch on N face.						
165	P-165-1 repair patch on SE face near top. P-165-4 repair patch on W face.						
166	Tower bent. P-166-4 rust stain on N and E face.						

Appendix I (continued)

Table A1 b (continued)

Bent	General Remarks	Inspection Notes	Potential	Resistivity	Continuity	IRH	Core Extraction
167	P-167-1 rust stain on N E and S face. P-167-4 rust stain on W face close to patch.	P-167-1 hammer sounding does not indicate delamination on N face					
168	P-168-3 rust stain on E face. P-168-4 rust stain on E and W face.						
169	P-169-2 rust stain on N face. P-169-3 rust stain on N face. P-169-4 rust stain on NE face.						
170	P-170-1 rust stain on S face near top.						
171	P-171-1 rust stain on N face. P-171-3 rust stain on S face. P-171-4 rust stain on NE face.						
172	P-172-1 rust stain on E and N face. P-172-4 rust stain on north face under patch.						
173	P-173-2 rust stain on E face under patch. P-173-3 rust stain on N face. P-173-4 rust stain on W face under patch.						
174	P-174-1 rust stain on S face. P-174-2 rust stain on S face. P-174-3 repair patch on E face. P-174-4 repair patch on E face.						
175	P-175-1 rust stain on E face a intermed. Elev, rust stain on W face. P-175-3 rust stain on E face. P-175-4 rust stain on N face.						
176	P-176-2 rust stain on E and W face. P-174-4 rust stain on W face.						
177	P-177-1 rust stain on E and S face. P-177-3 rust stain on N and E face near top. P-177-4 rust stain on W face under patch.						
178	p-178-2 repair patch in intermed. Zone. P-178-4 rust between patches.						
180	P-180-1 rust stain on E face. P-180-3 rust stain on E and W face.						
181	Tower bent. P-181-1 repair patch on E face at intermed level.						
182	P-182-2 repair patch. P-182-4 repair patch W face intermed. Level.						

Appendix I (continued)

Table A1 b (continued)

Bent	General Remarks	Inspection Notes	Potential	Resistivity	Continuity	IRH	Core Extraction
183	P-183-3 repair patch on N face. P-184-4 rust stain on N face under patch. P-183-4 rust stain on N face.						
184	P-184-1 rust stain on E face under patch. P-184-2 rust stain on W face. P-184-3 rust stain on N face under patch. P-184-4 rust stain on N face under patch.						
185	P-185-2 rust stain on E face under patch. P-185-4 rust stain on N and W face.	P185-4 W face spall ~32" AHT, exposed wire corroded. P185-4 N face delaminated ~6in ² .	P185-4				P185-4
186	P-186-1 rust stain on E face. P-186-4 rust stain on S face.						
187	P-187-1 repair patch on E face. P-187-2 crack on repair patch. P-187-4 rust stain on N W and S face.						
188	P-188-1 patch on SE face. P-188-4 rust stain on W face.						
189	P-189-1 rust stain on SE face. P-189-3 patch on N face. P-189-5 rust stain on WS face under patch.						
190	P-190-1 patch on S face.						
191	P-191-1 severe rust stain on NE face.						
192	P-192-1 rust stain. P-192-3 rust stain on E face.	P-192-1 NE face hammer sounding does not indicate delamination.					
193	P-193-1 rust stain on E face. P-193-2 repaired crack.						
194	P-194-1 rust stain on SW face. P-194-2 rust stain on E face at low elev.						
195	P-195-2 rust stain on E face at intermed. Elev.						
196	Tower bent. P196-1 severe rust stain on E face. P196-2 rust stain on N face, rust stain on W face under patch. P196-6 rust stain on E face.	P196-1 E face hammer sounding does not indicate delam. Exposed reinforcement rust-free.					P196-1

Appendix I (continued)

Table A1 b (continued)

Bent	General Remarks	Inspection Notes	Potential	Resistivity	Continuity	IRH	Core Extraction
197	P-197-4 rust stain on W face.						
198	P-198-1 rust on NW S faces. P-198-3 patch on N face. P-198-4 rust stain betw. patches.						
199	P-199-1 rust stain on E face, patch. P-199-2 rust stain on N face. P-199-3 patch on W face.						
200	P-200-1 rust stain on E face. P-200-4 patch on E face						
201	P-201-3 rust stain on N face. P-201-4 rust stain on N face.						
202	P-202-1 patch on E face. P-202-4 rust stain on E face below patch.						
204	P-204-1 rust stain on S face.						
205	P-205-4 patch on NE face.						
206	P-206-1 patch on top of E face.						
207	P-207-1 rust stain on WS face. P-207-3 patch on NE face.						
209	P-209-2 patch on E face.						
210	P-210-1 rust stain on E face. P-210-4 rust stain on W face.						
211	Tower bent. P-211-2 rust stain on NE face. P-211-3 crack in N face.						
212	P-212-2 patch on E face.						
213	P-213-2 rust stain on E face. P-213-4 crack on W face.	P-213-4 crack length ~4', crack width ~0.3mm.	P213-4				P213-4
214	P-214-1 rust stain on NE face in tidal zone. P-214-2 patch south face.						
215	P-215-1 rust stain on E face at interm. Elev. P-215-4 rust stain on W and S face at tidal zone.						
216	P-216-1 patch on NE face. P-216-4 rust stain on W face.						
217	P-217-1 rust stain on E face. P-217-2 repaired crack. P-217-4 rust stain on north face.						
218	P-218-3 rust stain on N face at tidal elev.						

Appendix I (continued)

Table A1 b (continued)

Bent	General Remarks	Inspection Notes	Potential	Resistivity	Continuity	IRH	Core extraction
219	P-219-1 rust stain on S and W face in tidal zone. P-219-4 severe rust stain on west face.	P-219-1 rust stain on W face severely delam. Delam concrete removed. Exposed reinforcement rust-free.	P219-4				P219-4
220	P-220-1 rust stain on E face. P-220-4 patch on W face.						
221	P-221-1 rust stain on N face at tidal elev. P-221-2 patch on N face. P-221-4 rust stain on W face.						
222	P-222-4 severe rust stain on S W face.						
223	P-223-1 rust stain on N face. P-223-2 patch on N face.						
224	P-224-1 patch on E face. P-224-3 rust stain on E face. P-224-4 rust stain on N face.						
225	P-225-1 patch on E face, rust stain on N S face. P-225-4 rust stain on top of S face.						
226	Tower bent.						
227	P-227-1 rust stain on E face. P-227-4 rust stain on S face.						
228	P-228-4 rust stain on N face.						
229	P-229-1 rust stain on W face. P-229-3 rust stain on N face.						
230	P-230-4 rust stain on S face.						
231	P-231-1 rust on west face. P-231-3 rust on N face. P-231-4 rust on S face.						
232	P-232-2 rust on N E face.						
233	P-233-2 rust on W and S face.						
234	P-234-3 rust on N face.						
235	P-235-1 patch on E face. P-235-2 patch on E face, severe rusting. P-235-4 rust stain on NW face below patch.						
236	P-236-1 rust on E face, patch on W face. P-236-3 patch on W face.						

Appendix I (continued)

Table A1 b (continued)

Bent	General Remarks	Inspection Notes	Potential	Resistivity	Continuity	IRH	Core Extraction
237	P-237-1 severe rust stain on E face intermed. Elevation. P-237-2 rust stain on E face	P237-1 delam in bottom ~20in ² . Exposed reinforcement rust-free.	P237-1				P237-1
238	P-238-1 patch on E face. P-238-4 rust stain on N W face near patch.						
239	P-239-1 patch on E face. P-239-4 rust on W face, patch on W face.						
240	P-240-2 patch on E face. P-240-3 rust on E face. P-240-4 rust on N face.						
241	Tower bent. P-241-1 patch on E face. P-241-2 patch on E and W face. P-241-3 patch on E face. P-241-4 on W face.						
242	P-242-2 patch on E face. P-242-4 patch on W face						
243	P-243-2 patch on N face. P-243-3 patch on N face. P-243-4 rust on N W face.						
244	P-244-1 patch on E face.						
245	P-245-1 rust on N face. P-245-2 rust on N face near patch.						
246	P-246-3 rust on E and S face.						
247	P-247-1 patch on E face. P-247-2 rust on W face. P-247-3 rust on W face.						
248	P-248-2 patch on N face. P-248-4 spall on W face.						
249	P-249-3 rust stain on E face intermed. Elev, rust stain on W face high elevation.	P249-4 Spall ~4in ² .					
250	P-250-1 patch on W face. P-250-2 rust on S face. P-250-4 rust on N face under patch.						
251	P-251-2 patch on E face. P-251-3 patch on N W face.						
252	P-252-1 rust stain on E face intermed. Elev. P-252-2 rust stain on W face high elev. P-252-4 repair patch on E face.	P-252-1 severe delamination 3'AHT. Area ~36in ² .	P252-1				P252-1
253	P-253-1 rust stain on N face. P-231-4 patch on E						
254	P-254-2 rust stain on W face.						
255	Rust stain on N face P-255-1 & E face P-255-2						

Appendix I (continued)

Table A1c Brooks Bridge Survey 3/13/03

Bent	General Remarks	Inspection Notes	Potential	Resistivity	Continuity	IRH	Core Extraction
1	End Abutment						
2	Ten 36" diameter cylinder piles. Piles on shore						
3	Piles in shallow water. Br-3-2 fine cracks 0.2mm.	Br-3-2 Crack from top of pile section. Core extraction upon cracks. Br-3-7, 8, and 9 cracks. Br-3-4 and 8 rust stain. Br-3-2-5 exposed wire shows rust.	Br-3-2				Br-3-2
4	Br-4-1 rust stain on E face. Br-4-6 fine crack. Br-4-9 concrete spall.	Br-4-1 no indication of delamination. Br-4-5, 7, and 8 fine crack. Br-4-9 crack ~0.3mm.					
5	Column piers.						
6	Column piers.						
7	Column piers.						
8	Column piers.						
9	Br-9-1 concrete spall. Br-9-6 concrete spall.	Br-9-5, 6, and 9 cracks. Br-9-10 small rust stain.					
10	Piles onshore.						
11	Piles on shore.						
12	Piles on shore.						
13	Piles onshore.						
14	End Abutment.						

Appendix I (continued)

Table A1d St George Island Survey 3/2/04

Bent	General Remarks	Inspection Notes	Potential	Resistivity	Continuity	IRH	Core Extraction
73	73EN superficial rust stain. 73ES rust stain. 73CS longitudinal cracking.						
133	133ES rust stain and longitudinal crack.						
139	139EN longitudinal crack. 139ES longitudinal cracking. 139CS rust stain.						
145	145ES rust stain and longitudinal crack. 145CS longitudinal crack. 145WS longitudinal crack.						
146	146ES longitudinal crack. 146CN longitudinal crack.						
152	Test pile on west side of pile bent.		152-TP-N	152-TP-N			152-TP-N
162	162WS longitudinal crack. 4 th bent from Eastpoint approach.		162WS		162WS		162WS
163	163WS longitudinal crack. 3 rd bent from Eastpoint approach.		163WS		163WS		163WS

Appendix II Bridge Survey Diagrams.

Figure A1 Core Extraction Diagrams (Not to scale. Units in inches unless otherwise noted.

Appendix II (continued)

Figure A1 (continued)

Figure A1 (continued)

Figure A1 (continued)

Appendix II (continued)

Figure A1 (continued)

Appendix II (continued)

Figure A2 Cumulative fraction of clear cover.

Table A2 Direct Clear Cover Measurements. (cm / in)

Bridge	Pile diameter (in.)	Steel Type								
		Spiral Wire			Spacing Bar			Strand		
		Min.	Avg.	Max.	Min.	Avg.	Max.	Min.	Avg.	Max.
Hathaway	54	2.5 0.98	3.06 1.20	3.5 1.38	2.5 0.98	2.5 0.98	2.5 0.98	4.4 1.73	4.7 1.85	5.0 1.97
Pensacola Bay	54	1.0 0.39	2.33 0.92	3.9 1.54	1.6 0.63	2.7 1.06	3.8 1.50	4.7 1.85	5.48 2.16	6.4 2.52
Brooks	36	2.6 1.02	2.9 1.14	3.2 1.26	3.8 1.50	3.8 1.50	3.8 1.50	N.A.		
Escambia Bay [2]	54	1.9 0.75	2.64 1.04	3.17 1.25	N.A.			1.27 0.5	3.51 1.38	5.08 2.0
	36	1.9 0.75	2.84 1.12	5.08 2.0				2.54 1.0	4.04 1.59	5.71 2.25
SGI	54	7.2 2.83	8.08 3.18	8.9 3.50				N.A.		

Appendix II (continued)

a)

b)

Figure A3 Pensacola Bay Bridge cumulative count of concrete deterioration features. a) rust stain cumulative count. b) spalls, cracks, and severe staining cumulative count.

Appendix II (continued)

a)

b)

c)

Figure A4 Examples of corrosion ratings. a) Example of spiral wire with No Corrosion (NC) rating. Hathaway Bridge, bottom of core hole, Pile H29-3. b) Example of spiral wire with Light Corrosion (LC) rating. Arrow indicates spot showing minor rust discoloration. Pensacola Bay Bridge, bottom of core hole, Pile P110-2. c) Example of severe corrosion (SC) of $\frac{1}{4}$ in (0.64 cm) diameter spiral wire from Pensacola Bay Bridge (P149-2) showing appreciable localized loss of cross section at the right end of the wire segment cut out by coring.

Appendix III Data Tables

Table A3 Core Properties, Chloride Diffusion Parameters, Projected Chloride Concentration, and Steel Condition

Bridge	Core	Elevation AHT (m)	Crack/ Spall	Crack Width/ Spall Area	Cs (mg/g _{dry})	D (m ² /s)	Projected C (mg/g) ^a		Cover Xc (cm) and Steel Condition ^b		
							Spiral	Bar	Spiral	Bar	Strand
Hathaway	H-8-2-1	0.15			c	c	0.19		3.3	NC	-
	H-8-2-2	0.11			11.6	5.92E-14			-	-	-
	H-8-2-3	-0.14			9.75	2.86E-13			-	-	-
	H-8-2-4	-0.14			9.6	9.20E-14	1.16		2.5	NC	-
	H-8-2-5	1.52			7.37	3.02E-14	0.08		3.3	NC	-
	H-8-2-6	1.52			c	c	0.10		2.7	NC	-
	H-29-3-1	0.15			c	c			-	-	5.0 NC
	H-29-3-2	0.15			8.36	8.41E-14	0.25		3.5 ^e	NC	-
	H-29-3-3	1.52	Crack	0.3 mm	4.69	7.41E-14		0.44	-	2.5 LC	4.4 LC
	H-29-3-4	1.52			c	c	0.14		3.5	NC	-
Pensacola Bay	P-4-1-1	0.32			7.4	2.40E-13	3.42		1.9	NC	-
	P-4-1-2	0.32	Crack	0.3 mm	12.8	4.38E-14	3.14 ^g		2	NC	-
	P-4-1-3	0.76			8.64	7.79E-14	0.81		2.5	NC	-
	P-4-1-4	0.79			c	c	0.93		2.4	NC	5.1 LC
	P-110-2	0.30	Spall	100cm ²	g	g	1.87		2.7 ^e	LC	-
	P-110-2-1	0.38			c	c	1.26		2.7 ^e	NC	-
	P-110-2-2	0.38			8.35	1.25E-13	1.26		2.7	NC	6.4 LC
	P-110-2-4	1.52			9.38	6.07E-14			-	-	6.4 NC

Appendix III (continued)

Table A3 (continued)

Bridge	Core	Elevation AHT (m)	Crack/ Spall	Crack Width/ Spall Area	Cs (mg _{cl} /g _{dry})	D (m ² /s)	Projected C (mg/g) ^a		Cover Xc (cm) and Steel Condition ^b		
							Spiral	Bar	Spiral	Bar	Strand
Pensacola Bay	P-149-2	0.66	Spall	270cm ²	g	g	1.90		1.4 ^f	SC	-
	P-185-4	0.81	Spall	100cm ²	g	g	2.04	1.15 ^d	2.6 ^e	SC	3.2 ^e NC
	P-185-4-1	0.48			14.7	9.6E-14	1.0		3	NC	-
	P-185-4-2	0.48			12.3	5.52E-14	1.13		2.1	NC	-
	P-196-1-1	0.33			g	g	2.4		2.4	NC	-
	P-196-1-2	0.33			g	g	2.22		2.5	NC	-
	P-213-4-1	1.42			6.05	8.66E-14	0.10 ^g		3.9	NC	-
	P-213-4-2	1.42	Crack	0.3 mm	5.0	6.81E-14		0.11	-	3.8	NC
	P-219-4-1	0.29			9.47	6.38E-14	0.84		2.3	NC	-
	P-219-4-2	0.29			c	c	1.92		1.7	NC	-
	P-237-1-1	0.28			7.82	3.12E-13	4.07 ^g		2.6	NC	-
	P-237-1-2	0.28			c	c	3.25		2.4	NC	4.8 NC
	P-252-1-1	0.30			c	c	5.88	4.77 ^d	1.0	NC	4.7 NC
	P-252-1-2	0.28			7.89	3.43E-13	4.08		1.9	NC	-
Brooks	Br-3-2-3	0.62	Crack	0.2 mm	3.76	2.81E-13			-	-	-
	Br-3-2-4	0.62			3.66	2.46E-13			-	-	-
	Br-3-2-5	0.15			4.69	1.97E-13	1.18	0.47	2.6	LC	3.8 LC
	Br-3-2-6	0.15	Crack	0.2 mm	5.33	2.88E-13	1.28		3.2	NC	-

Appendix III (continued)

Table A3 (continued)

Bridge	Core	Elevation AHT (m)	Crack/ Spall	Crack width/ Spall Area	Cs (mg _{cl} /g _{dry})	D (m ² /s)	Projected C (mg/g) ^a		Cover Xc (cm) and Steel Condition ^b		
							Spiral	Bar	Spiral	Bar	Strand
St. George Island	162ws1	0.15	Crack	< 0.05 mm	8.96	7.84E-14	0.08		7.9	NC	-
	162ws2	0.15			8.22	5.02E-14	0.08		8	NC	-
	162ws3	0.30	Crack	< 0.05 mm	6.70	4.39E-14	0.08		8.3	NC	-
	162ws4	0.30			6.97	2.87E-14	0.08		8	NC	-
	163ws1	0.15	Crack	< 0.05 mm	4.33	5.02E-13	0.08		8.9	NC	-
	163ws2	0.15			4.08	3.09E-13	0.08		8.1	NC	-
	163ws3	0.30	Crack	< 0.05 mm	4.11	1.74E-13	0.08		7.2	NC	-
	163ws4	0.30			4.24	2.40E-13	0.08		8.3	NC	-

Table A3 Notes

- Except as indicated, value listed is projected chloride concentration at depth of spiral wire and/or spacing bar (Xc) at time of inspection using equation (1), listed diffusion parameters, and assuming Co=0.08mg/g. In cores where direct comparison with data in Table A3 was possible, projected value was found to be between those listed for slices immediately above and below steel position. Exceptions are noted under (g).
- NC- no corrosion. LC- light surface rust. SC- severe corrosion. See Section 3.
- Diffusion parameter values of companion core extracted at the same elevation. See Figure A1.
- Cover data taken from companion core.
- Cover value estimated using spall and core fragments.
- Assumed typical diffusion parameter values for cores at low elevation. D=1.5e-13m²/s, Cs=10mg/g.
- Listed value is by linear interpolation from concentrations of slices immediately above and below steel position as listed in Table A4.

Appendix III (continued)

Table A4 Chloride Concentration Profile Data and Analysis Results

Bridge	Core	Time in service (yrs)	C (mg/g)	Midpoint (cm)	Thickness (cm)	C _s (mg/g)	D (m ² /s)
Hathaway	H8-2-2	43	11.05	0.10	0.20	11.6	5.92e-14
			7.53	0.55	0.70		
			3.33	1.41	0.64		
			0.90	2.47	1.12		
			0.16	4.22	1.00		
			0.148	5.26	1.69		
			8.8	0.15	0.3		
			8.41	0.57	0.54		
	H8-2-3	43	7.24	1.24	0.44	9.75	2.86e-13
			3.92	2.19	1.07		
			1.57	3.85	1.88		
			9.05	0.15	0.30		
			6.62	0.57	0.54		
			4.60	1.24	0.44		
	H8-2-4	43	1.46	2.19	1.07	9.6	9.20e-14
			0.27	4.00	2.18		
			6.33	0.13	0.25		
			4.76	0.41	0.32		
			1.31	1.21	0.45		
			0.50	1.86	0.92		
	H8-2-5	43	0.64	3.12	1.21	7.37	3.02e-14
			11.46*	0.10	0.20		
			6.07	0.50	0.61		
			3.72	1.27	0.54		
			1.52	2.23	1.01		
			0.01	3.67	1.48		
			4.02	.27	.55		
			2.29	1.07	.66		
	H29-3-2	43	0.31	2.63	.78	8.36	8.41e-14
			0.12	3.48	1.22		
	H29-3	43				4.69	7.41e-14

Appendix III (continued)

Table A4 (continued)

Bridge	Core	Time in service (yrs)	C (mg/g)	Midpoint (cm)	Thickness (cm)	C _s (mg/g)	D (m ² /s)
Pensacola Bay	P4-1-1	43	7.05	0.18	0.35	7.4	2.40e-13
			5.69	0.71	0.72		
			3.89	1.51	0.48		
			2.25	3.00	2.12		
			0.43	4.71	1.31		
			0.18	6.14	1.17		
			12.16	0.10	0.20		
	P4-1-2	43	8.22	0.47	0.54	12.8	4.38e-14
			4.09	1.14	0.44		
			0.99	3.96	2.30		
			8.39	0.10	0.20		
	P4-1-3	43	6.08	0.47	0.19	8.64	7.79e-14
			3.64	1.14	0.45		
			1.48	2.40	1.72		
			0.08	3.98	1.08		
			0.07	5.83	2.25		
	P110-2-2	43	13.09	0.14	0.28	8.35	1.25e-13
			6.48	0.49	0.43		
			4.45	1.23	0.46		
			1.69	2.41	1.73		
			0.25	3.88	1.20		
			0.12	5.38	1.41		
	P110-2-4	43	6.34	0.57	1.13	9.38	6.07e-14
			2.57	1.52	0.78		
			0.18	2.64	1.08		
			0.11	4.71	0.87		
			7.91	0.18	0.35		
	P185-4-1	43	10.45	0.62	0.54	14.7	9.6e-14
			6.39	1.29	0.45		
			1.74	2.70	1.99		
			0.03	4.39	1.40		

Appendix III (continued)

Table A4 (continued)

Bridge	Core	Time in service (yrs)	C (mg/g)	Midpoint (cm)	Thickness (cm)	C _s (mg/g)	D (m ² /s)
Pensacola Bay	P185-4-2	43	10.73	0.20	0.40	12.3	5.52e-14
			7.66	0.59	0.38		
			0.38	3.31	1.73		
			0.16	4.90	1.07		
			0.12	6.00	0.75		
			3.75	0.79	1.58		
	P213-4-1	43	1.30	2.04	0.91	6.05	8.66e-14
			0.11	3.22	1.07		
			0.09	4.48	1.07		
			4.23	0.32	0.63		
	P213-4-2	43	1.98	1.10	0.94	5.0	6.81e-14
			0.87	1.98	0.44		
			0.45	2.60	0.44		
			0.33	3.23	0.44		
			7.87	0.30	0.60		
			4.73	0.87	0.54		
	P219-4-1	43	2.65	1.54	0.44	9.44	6.61e-14
			0.19	2.80	1.71		
			0.13	4.91	2.13		
			6.99	0.30	0.60		
			5.85	0.87	0.54		
			5.00	1.54	0.44		
	P237-1-1	43	3.33	2.80	1.71	7.86	3.12e-13
			0.59	4.38	1.07		
			0.16	6.25	2.29		
			6.31	0.23	0.45		
			6.80	0.72	0.54		
			4.51	1.39	0.45		
	P252-1-2	43	3.79	2.34	1.08	7.89	3.43e-13
			2.03	3.61	1.08		
			0.54	5.71	2.73		

Appendix III (continued)

Table A4 (continued)

Bridge	Core	Time in service (yrs)	C (mg/g)	Midpoint (cm)	Thickness (cm)	C _s (mg/g)	D (m ² /s)
Brooks	Br3-2-3	39	5.84*	0.14	0.27	3.76	2.81e-13
			3.40	0.54	0.54		
			2.12	1.21	0.44		
			1.59	2.16	1.07		
			0.86	3.84	1.92		
	Br3-2-4	39	6.83*	0.10	0.20	3.66	2.46e-13
			3.20	0.39	0.39		
			2.46	1.09	0.63		
			1.46	2.07	0.95		
			0.66	3.73	1.99		
	Br3-2-5	39	10.22*	0.10	0.20	4.69	1.97e-13
			4.05	0.47	0.43		
			2.52	1.15	0.50		
			1.54	2.43	1.47		
			8.77*	0.13	0.25		
	Br3-2-6	39	4.45	0.54	0.58	5.33	2.88e-13
			3.51	1.27	0.50		
			1.44	2.97	2.54		
			0.59	4.89	1.30		
			9.48	0.06	0.11		
St. George Island	152TPN-2	2.18	8.32	0.16	0.10	10.3	2.26e-13
			6.67	0.28	0.13		
			7.75	0.04	0.08		
			6.38	0.13	0.09		
			4.76	0.23	0.11		
	162WS-1	2.6	2.81	0.33	0.10	8.96	7.84e-14
			1.57	0.42	0.09		
			1.18	0.58	0.23		
			0.65	1.08	0.61		

Appendix III (continued)

Table A4 (continued)

Bridge	Core	Time in service (yrs)	C (mg/g)	Midpoint (cm)	Thickness (cm)	C _s (mg/g)	D (m ² /s)
St. George Island	162WS-2	2.6	7.00	0.05	0.10	8.22	5.02e-14
			5.15	0.15	0.09		
			3.37	0.24	0.09		
			1.90	0.33	0.08		
			0.61	0.46	0.20		
			0.25	0.93	0.56		
			5.83	0.05	0.09		
			3.87	0.14	0.10		
	162WS-3	2.6	2.33	0.24	0.10	6.70	4.39e-14
			1.28	0.34	0.09		
			1.01	0.44	0.12		
			0.48	0.61	0.21		
			0.52	1.18	0.56		
			5.58	0.05	0.10		
			3.94	0.15	0.10		
			1.67	0.25	0.10		
	162WS-4	2.6	0.75	0.35	0.10	6.97	2.87e-14
			0.27	0.49	0.17		
			0.11	1.09	0.51		
			4.03	0.07	0.13		
			3.81	0.18	0.09		
			3.25	0.29	0.12		
			2.79	0.41	0.12		
			1.82	0.73	0.33		
	163WS-1	2.14	0.45	1.20	0.25	4.33	5.02e-13
			0.09	8.47	0.40		
			3.76	0.05	0.10		
			3.28	0.17	0.14		
			2.89	0.31	0.14		
			2.01	0.46	0.10		
			0.82	0.62	0.23		
			0.30	1.09	0.36		
	163WS-2	2.14	0.08	8.12	0.30	4.08	3.09e-13

Appendix III (continued)

Table A4 (continued)

Bridge	Core	Time in service (yrs)	C (mg/g)	Midpoint (cm)	Thickness (cm)	C _s (mg/g)	D (m ² /s)
St. George Island	163WS-3	2.14	3.77	0.06	0.12	4.11	1.74e-13
			3.34	0.17	0.12		
			2.41	0.28	0.10		
			1.77	0.38	0.10		
			1.23	0.55	0.24		
			0.54	1.00	0.32		
			0.14	8.30	0.29		
			3.90	0.05	0.09		
	163WS-4	2.14	3.57	0.13	0.07	4.24	2.4e-13
			2.75	0.22	0.11		
			2.26	0.33	0.10		
			1.66	0.48	0.20		
			0.99	0.67	0.19		
			0.68	0.87	0.21		
			0.35	1.07	0.20		
			0.08	8.29	0.25		
	1404-3	1.83	5.45	0.052	0.10	5.95	1.44e-13
			4.19	0.15	0.08		
			3.19	0.23	0.09		
			2.49	0.32	0.08		
	1404-4	1.80	8.50	0.03	0.07	9.53	1.09e-13
			6.48	0.13	0.12		
			4.770	0.21	0.12		
			4.33	0.28	0.09		
			2.72	0.39	0.14		
	943-3	1.89	7.39	0.077	0.94	8.37	4.03e-13
			7.14	1.45	1.03		
			6.04	2.49	0.75		
			5.04	3.47	1.21		
			4.36	4.55	1.25		

* Data for sample closest to surface not used to fit chloride concentration and depth to Eq. 1 (see Section 4.3).

Appendix III (continued)

Table A5 Field Potential Mapping

Bridge	Pile	Reinforcement	Elevation AHT (ft)	E vs CSE (mV)		Remarks
Hathaway	H-8-2-1	wire	Water	-361	-376	
			0	-151	-167	
			1	-143	-160	
			2	-40	-68	
			3	73	38	
			4	172	141	
	H-29-3-1	strand	5	191	160	
			6	180	147	
			Water	-656	-625	
			0	-426	-389	
			1	-328	-312	
			2	-251	-240	
	H-29-3-2	wire	3	-154	-151	
			4	-111	-100	
			5	-103	-100	
			6	-90	-82	
			Water	-425	-425	
			0	-245	-238	
	H-29-3-3	spacing rebar	1	-179	-194	Minor rusting on rebar. Coincident with crack.
			2	-116	-113	
			3	-14	-11	
			4	9	9	
			5	40	40	
			6	72	55	
			Water	-520	-524	
			0	-376	-382	
			1	-264	-260	
			2	-161	-161	
			3	-96	-140	
			4	-99	-96	
			5	-60	-68	
			6	-3	-12	

Appendix III (continued)

Table A5 (continued)

Bridge	Pile	Reinforcement	Elevation AHT (ft)	E vs CSE (mV)		Remarks
Hathaway	H-29-3-4	wire	Water	-530	-508	
			0	-364	-348	
			1	-240	-244	
			2	-181	-187	
			3	-120	-127	
			4	-111	-108	
			5	-88	-69	
			6	-43	-40	
			0	-535		
			1	-499		
Pensacola Bay	P-4-1-1	wire	2	-420		
			3	-276		
			4	-240		
			0	-379		
	P-4-1-2	strand	1	-306		
			2	-260		
			3	-167		
			4	-112		
	P-4-1-3	wire	0	-302	-331	
			1	-282	-282	
			2	-269	-248	
			3	-157	-150	
	P-4-1-4	strand	4	-75	-119	
			0	-380		
			1	-299		
			2	-285		
	P-110-2	wire	3	-148		Core extracted from spalled concrete. Corroded wire.
			4	-130		
			0	-270	-250	
			1	-301	-256	
			2	-182	-160	
			3	-134	-79	
			4	-100	-54	
			5	-5	7	

Appendix III (continued)

Table A5 (continued)

Bridge	Pile	Reinforcement	Elevation AHT (ft)	E vs CSE (mV)	Remarks
Pensacola Bay	P-110-2-1	wire	0	-263	
			1	-283	
			2	-129	
			3	-70	
			4	-84	
			5	3	
			0	-270	
			1	-179	
			2	-134	
			3	-87	
	P-110-2-2	wire	4	-84	
			5	-52	
			0	-333	
			1	-232	
			2	-73	
			3	-17	
			4	-38	
			5	-109	
			Water	-456	
			0	-325	
	P-149-2	strand	1	-230	Core extracted from spalled concrete. Corroded wire.
			2	-214	
			3	-160	
			4	-69	
			5	-85	
			Water	-590	
			0	-402	
			1	-343	
			2	-420	
			3	-238	
Pensacola Bay	P-185-4	spiral	4	-334	Core extracted from spalled concrete. Corroded wire.
			5	-187	
				-434	
				-396	
				-462	

Appendix III (continued)

Table A5 (continued)

Bridge	Pile	Reinforcement	Elevation AHT (ft)	E vs CSE (mV)		Remarks
Pensacola Bay	P-185-4	Strand	Water	-585		Core extracted from spalled concrete. Corroded wire.
			0	-404	-401	
			1	-352	-342	
			2	-404	-396	
			3	-241	-248	
			4	-335	-312	
			5	-197	-196	
			Water	-530		
			0	-448		
			1	-290		
	P-213-4-1	wire	2	-269		
			3	-205		
			4	-207		
			5	-178		
			6	-154		
			Water	-530		
	P-213-4-2	strand	0	-444		
			1	-305		
			2	-275		
			3	-262		
			4	-227		
			5	-187		
	P-219-4-1	wire	6	-181		
			0	-409		
			1	-381		
			2	-298		
			3	-260		
			4	-240		
	P-237-1-1	wire	Water	-490	-489	
			0	-410	-380	
			1	-290	-302	
			2	-196	-214	
			3	-138	-145	
			4	-115	-135	

Appendix III (continued)

Table A5 (continued)

Bridge	Pile	Reinforcement	Elevation AHT (ft)	E vs CSE (mV)		Remarks
Pensacola Bay	P-252-1-1	wire	Water	-460		
			0	-417		
			1	-351		
			2	-180		
			3	-150		
			4	-109		
			0	-365		
			1	-287		
	P-252-1-1	spacing rebar	2	-70		
			3	-109		
			4	-119		
			Water	-500		
	P252-1-1	strand	0	-42		
			1	-362		
			2	-194		
			3	-37		
			4	-212		
			Water	-499		
			0	-324		
			1	-160		
Brooks	P-252-1-2	wire	2	-84		
			3	-153		
			4	-44		
			Water	-684	-693	
			0	-638	-645	
			1	-359	-395	
			2	-109	-154	
			3	-116	-153	
			4	-69	-92	
	Br-3-2-5	spiral	Water	-670	-693	
			0	-624	-643	
			1	-376	-378	
			2	-162	-153	
			3	-140	-125	
			4	-129	-95	
			Water	-670	-693	
			0	-624	-643	
			1	-376	-378	
			2	-162	-153	
			3	-140	-125	
			4	-129	-95	
	Br-3-2-5	spacing rebar	Water	-670	-693	
			0	-624	-643	
			1	-376	-378	
			2	-162	-153	

Appendix III (continued)

Table A5 (continued)

Bridge	Pile	Reinforcement	Elevation AHT (ft)	E vs CSE (mV)		Remarks
Brooks	Br-3-2-6	spiral	Water	-640	-643	
			0	-584	-549	
			1	-337	-300	
			2	-93	-81	
			3	-50	-26	
			4	-30	-8	
			Water	-489		
			0	-360		
			1	-241		
			2	-168		
St. George Island	152TPN-2	wire	3	-68		
			4	-55		
			5	-72		
			Water	-567		
			0	-415		
			1	-267		
	162WS-1	wire	2	-159		
			3	-110		
			4	-153		
			5	-155		
			Water	-568		
			0	-382		
	162WS-4	wire	1	-241		
			2	-134		
			3	-134		
			4	-163		
			5	-174		
			Water	-410		
	163WS-3	wire	0	-382		
			1	-275		
			2	-232		
			3	-131		
			4	-68		
			5	-57		

Appendix III (continued)

Table A5 (continued)

Bridge	Pile	Reinforcement	Elevation AHT (ft)	E vs CSE (mV)	Remarks
St. George Island	163WS-4	wire	Water	-407	
			0	-405	
			1	-321	
			2	-148	
			3	-136	
			4	-42	
			5	-42	

Appendix III (continued)

Table A6 Field IRH Measurements

Bridge	Pile	Elevation AHT (ft)	Time	IRH (%)		Temperature (°C)	Remarks
				Measured	Calibrated		
Hathaway	H-8-2	0.44	10:30	82.8	84.7	15	Sealed probe into fitting.
			10:31	78.9	80.6	16.5	
			10:33	76.3	77.9	17.2	
			10:39	74.6	76.1	17	
			10:47	73.9	75.4	16.7	
			10:52	73.5	75.0	16.6	
			11:00	72.9	74.4	16.5	
			11:12	72.1	73.5	16.4	
			11:19	71.8	73.2	16.4	
			11:29	71.4	72.8	16.3	Ambient conditions 85%RH 15°C
			10:31	83.5	75.1	15.1	Sealed probe into fitting.
			10:33	82.3	73.7	15.1	
	H-8-2	5	10:39	81.1	72.4	15.2	
			10:47	79.9	72.1	15.1	
			10:52	79.6	71.7	14.9	
			11:00	79.2	71.1	14.9	
			11:12	78.7	71.0	15	
			11:19	78.6	70.7	15	
			11:29	78.3	70.6	15	Ambient conditions 85%RH 15°C
			11:41	74.9	76.4	14.9	Sealed probe into fitting.
	H-29-3	0.5	11:42	83.6	85.6	14.8	
			11:45	84	86.0	14.5	
			11:50	84.6	86.6	14.3	
			12:00	84.9	86.9	14.3	
			12:05	84.8	86.8	14.4	
			12:10	84.8	86.8	14.5	
			12:15	84.8	86.8	14.5	
			12:20	84.7	86.7	14.6	
			12:26	84.7	86.7	14.6	
			12:30	84.7	86.7	14.7	
			12:35	84.6	86.6	14.7	

Appendix III (continued)

Table A6 (continued)

Bridge	Pile	Elevation AHT (ft)	Time	IRH (%)		Temperature (°C)	Remarks
				Measured	Calibrated		
Hathaway	H-29-3	5	11:41	85.1	79.0	15.7	Sealed probe into fitting.
			11:42	83.5	76.4	15.1	
			11:45	82.6	75.4	14.7	
			11:50	81.2	73.9	14.7	
			12:00	80.5	73.9	13.0	
			12:05	79.9	73.3	14.7	
			12:10	79.4	72.7	14.8	
			12:15	79.3	72.6	15.0	
			12:20	79	72.3	15.0	
			12:26	78.5	71.7	15.1	
			12:30	78.5	71.7	15.1	
			12:35	78.4	71.6	15.3	
Pensacola Bay	P110-2	1.17	4:00	-	-	-	Sealed probe into fitting.
			4:02	71	66.7	18.7	
			4:04	71.6	67.4	18.8	
			4:06	72.2	68.1	18.7	
			4:10	73.2	69.3	18.8	
			4:26	73.5	69.6	19.3	
			4:33	73.7	69.9	19.4	
			4:41	73.7	69.9	19.4	
			4:46	73.8	70.0	19.4	
			4:51	73.9	70.1	19.4	Ambient 63.7%RH 19.6°C
			4:02	-	-	-	Sealed probe into fitting.
			4:04	68.6	64.2	19.7	
			4:06	69.4	65.2	19.6	
	P110-2	5	4:10	70.3	66.3	19.3	
			4:26	71.1	67.2	19.4	
			4:33	71.3	67.5	19.4	
			4:41	71.4	67.6	19.5	
			4:46	71.5	67.7	19.5	
			4:51	71.5	67.7	19.4	Ambient 63.7%RH 19.6°C