Proceeding
PRO 89

Proceedings of the RILEM International workshop on performance-based specification and control of concrete durability

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11 - 13 June 2014

Zagreb, Croatia

REINFORCED CONCRETE CORROSION PERFORMANCE IN FLORIDA BRIDGES

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Abstract

Florida marine bridges are exposed to aggressive environments with high temperature, humidity and chloride exposure. Corrosion control guidelines for design at present emphasize the use of low permeability concrete by means of pozzolanic admixtures, high cementitious content, low water to cementitious ratio, and high clear concrete cover. Highlights of field evaluations are presented describing experience that led to discontinuing the use of epoxy coated rebar, verification of the adequate performance of newly formulated concretes, and assessment of the impact of cracking on durability. A brief update on corrosion issues in post tensioned systems is presented as well.

Keywords: corrosion, Florida, concrete, cracks, epoxy-coated rebar, chloride

1. INTRODUCTION

1.1 Service conditions and performance challenges

The State of Florida is the southernmost state of the United States (U.S) continental territory. According to the Köppen climate classification [1], the predominant climate is Humid Subtropical, while the southern region climate is categorized as Tropical. The annual average temperature and relative humidity are ~20ºC and ~70%, respectively, and both values tend to be appreciably higher during the summer. Most of the state is surrounded by waters from the Gulf of Mexico (West) and Atlantic Ocean (East) with a high chloride ion content of ~2.5%. Chloride content in water around estuarine bridges tends to be ~1% . Annual precipitation is ~1.5m with episodic severe weather in the form of tropical storms and hurricanes.

The Florida Department of Transportation (FDOT) owns approximately 6,500 bridges, about one half of which are exposed to salt water. About 3,000 other bridges on saltwater in Florida are owned by other public entities, and some of those structures are scheduled to become the responsibility of FDOT in the future. Newly designed FDOT structures must meet a minimum 75-year service life, per current guidelines by the American Association of State Highway Transportation Officials (AASHTO). [2]. This requirement represents a major issue in infrastructure reliability since the geographical location, climate conditions, and a densely developed coastline contribute to make the State of Florida one of the U.S. regions with the most severe liability for corrosion damage of embedded steel in concrete. Corrosion issues tend to be manifested earlier in Florida, and study of lessons learned there helps prepare for informed corrosion control elsewhere.

In Florida bridges concrete damage by reinforcing bar (rebar) or prestressed strand corrosion is the prevalent mode of concern in the substructure located in the tidal and splash-evaporation zones, where the content of chloride ions is high due to evaporative concentration. The chloride ions penetrate through the concrete cover and upon building to a critical threshold concentration (C_T) cause breakdown of the otherwise normal protective passive film that forms on the steel surface in contact with the highly alkaline pore water. The damage is commonly in the form of cracks, delaminations, or spalling caused by expansive steel corrosion products. Much of the FDOT research effort and specification development has been aimed at controlling marine substructure corrosion. 3-6]The external bridge superstructure and decks are at a much lower corrosion risk since seawater chloride contamination is minor at high elevations and deicing salts are not used in the State of Florida.

Under-water corrosion of reinforced concrete in pilings has not been yet a known issue of concern; however, there is ongoing research on the subject as the more demanding service life goals for bridges are being considered. Concrete carbonation induced corrosion has not been a major mode of deterioration in FDOT bridges so far. A study on that issue indicated however that it may merit attention if the service period of existing bridges were to be considerably extended [7].

During the last decade and a half a less frequent, but locally severe form of corrosion damage has affected a number of post-tensioned (PT) tendons in several major Florida bridges. The issue has received much attention in developing control guidelines as the tendons, used in both sub- and superstructure of the bridges, are critical structural components. The sources of the corrosion involve internal factors sometimes aggravated by external chloride intrusion. [8] This paper is a brief overview of FDOT corrosion control guidelines and specifications for design of durable reinforced concrete bridges, and of selected notable field experiences documenting major corrosion performance issues.

1.2 Florida Design Guidelines for Corrosion-Related Durability

The FDOT Structures Manual [9] is Volume I of the FDOT Structures Design Guidelines (SDG) for engineers who design structures, including bridges, for the State of Florida. The Manual follows the requirements of the AASHTO LRFD (Load and Resistance Factor Design)Bridge Design Specification as adapted to the State needs.

In the FDOT SDG a structure is divided in two main parts; superstructure conforming to all the bridge elements that are above the bearings, and substructure to the bridge sections located under the bearings. The latter corresponds to the bridge portions that are the most susceptible to chloride induced corrosion as noted earlier and receive the most attention in this paper. Structures can be classified as Marine or Non-Marine depending on its location. Structures submerged in water and/or soil or located 2,500 ft. (~760 m) away from a body of water with a Cl^- concentration greater than 2000 ppm are designated as marine structures. Depending on the chloride and sulfate ion content (as well as acidity if relevant), of the environment, surrounding the bridge, three environmental classifications are specified: Extremely Aggressive ($Cl^->2000$ ppm), Slightly Aggressive ($Cl^->500$ ppm) and Moderately

Aggressive (ranging between the other limit values). Only the chloride ion content limit information was provided for the environmental classification. For information on pH and sulfate content please refer to Ref. [9].

The FDOT design guidelines and procedures follow a prescriptive method, effectively a "Deemed-to-Satisfy" method per the *fib* (The International Federation for Structural Concrete) designations.[10] The aggressiveness of the environment and the bridge element location (superstructure and substructure) defines the concrete cover and structural concrete class requirements. High performance concretes of high cement content, low water-to-cement ratio, and cement replacements such as Fly Ash, Slag and Microsilica are specified to achieve long-term durability. Very low chloride diffusion coefficients, nearing values in the order of 1e-9 cm^2/sec, are routinely achieved using those formulations. As will be shown below, the approach to achieve long term durability does not at present rely on corrosion resistant reinforcement, so emphasis is placed on the use of highly impermeable concrete and adequate concrete cover.

FDOT guidelines have been a result of many years of investment in field and experimental assessments to establish the best approaches and practices to reduce reinforcement corrosion damage and provide a design basis to meet the 75-year service life requirement. Highlights of field experience and subsequent research are summarized next.

2. HIGHLIGHTS OF NOTABLE CORROSION PERFORMANCE ISSUES

2.1 30 Years Experience with Epoxy-Coated Rebar

Following the recommendations from FHWA guidelines at the time, numerous bridges constructed by the FDOT during the late 1970's and early 1980's were built using ECR. Many of these bridges are located in the Florida Keys, a region with yearlong high temperature and relative humidity. The concrete used in several of the major Florida Keys Bridges built with ECR had high permeability, as manifested by high chloride diffusion coefficients in the order of 1E-7 cm^2/s, placing much of the burden of corrosion control on the ECR.[11] That control was insufficient as corrosion-related spalls began to be observed in several of the bridges, designated by NIL, 7MI, LOK, INK and CH5 in Figure 1, at ages as early as 6 years. The damage progressed rapidly with time, with no indications of a slowdown; after 30 years the bridges showed on average multiple corrosion spalls per pier.[12]

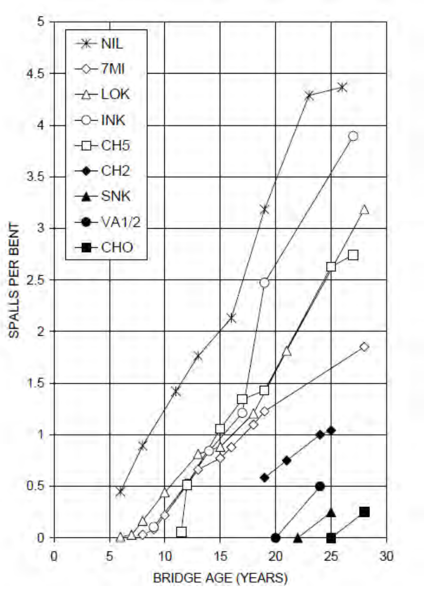

Figure 1: Damage progression in bridges with substructure build with ECR. Corrosion spall count is normalized by dividing by number of bents (pier) in each bridge. Structure age at the moment of evaluation per bridge code: NIL: 28 years, 7MI: 26 years, LOK: 26 years, INK: 27 years, CH5:26 years, CH2:27 years, SNK: 27 years, VA1/2: 26 years, CHO: 29 years. [11]

Research indicated a mechanism whereby coating disbondment began around preexisting coating damage while exposed to marine spray in the construction yard, followed by loss of coating after more than 5 years' service even in the absence of further chloride contamination.[12] On eventual chloride ingress, those factors promoted rapid corrosion in the steel-disbonded coating crevice, aggravated by efficient macrocell coupling with steel exposed at other coating defects. Solid corrosion products formed beneath the coating and caused concrete spalling damage. Based on this experience and findings FDOT discontinued in 1991 the specification of ECR for new construction projects. More recent work confirmed that damage was beginning to be manifested in other bridges built with ECR that had lower concrete permeability than in the first set (CH2, SNK, VA1/2, CHO in Figure 1). [11]

Surveys of other ECR bridges that had a thick rebar cover of highly impermeable concrete showed no ECR corrosion in areas of sound concrete, indicating that the corrosion avoidance was due primarily to the high quality of the concrete. Importantly, severe ECR corrosion was found in the substructure of another marine bridge built with otherwise high quality concrete and thick rebar cover, at locations where cracks ~1mm wide had intersected the rebar.[13,14] This observation further confirmed the vulnerability of ECR and served as an additional factor in leading to the FDOT corrosion control approach based on using conventional reinforcement combined with highly impermeable concrete and substantial clear concrete cover.

A mathematical model to forecast the service life of structures built with ECR was developed to adequately plan for future repair needs of these structures. The model projected that repair needs will continue in the long term to accrue in bridges with high and medium chloride permeability concrete. Damage projections are very low for the low permeability concrete structures, except for limited incidence at crack locations (addressed in the next section). [13,14]

2.2 Corrosion in locally deficient concrete

The newer concrete formulations and thick clear concrete covers used by FDOT have resulted in long corrosion-related durability projections, satisfying the 75-years design guidelines, for much of the substructure of recently built bridges. However, there was concern that local concrete deficiencies such as cracks could provide paths for fast chloride penetration with attendant early corrosion onset.

Surveys of the incidence of concrete cracks found cracks both from stress or shrinkage origin near the corrosion-prone tidal region in piles and footers, in both precast and cast in place structural elements. The median stress crack width was approximately 0.15 mm but several instances approaching 1 mm with were observed as well. The stress cracks regularly tended to extend all the way down to the rebar depth form creating a path towards the steel bar, whereas shrinkage cracks tended to be typically superficial. Stress cracks were encountered on average every 3 m along the waterline perimeter of partially submerged structural elements.[13]

Concrete cores were extracted in pairs near the tidal region, one core centered on a preexisting crack and the other ~15 cm away at the same elevation but on sound concrete, and chloride penetration profiles were obtained for both. Figures 2a and 2b show the results obtained for three highly permeable and highly impermeable concrete bridges respectively. The profiles for sound and cracked concrete in the highly permeable concrete bridges are quite comparable, indicating that crack transport on those cases is not strongly preferential to sound concrete transport. That lack of differentiation is not surprising since the bulk transport is very fast already in the sound concrete and the crack does not represent an important relative addition. In contrast, there was strong preferential penetration at cracked concrete compared to that at sound concrete in the highly impermeable concrete bridges. That strong differentiation reflects the role of the crack in those less permeable concretes as the only important chloride transport path. [13,14] The effect was still relatively important at higher elevations.

Careful examinations of steel condition at crack-rebar intersections were conducted at several bridges built with ECR given the propensity for early corrosion observed for that system. Observations at five bridges having moderate to low sound concrete chloride permeability showed no signs of corrosion aggravation at cracks that reached to the rebar depth in four of the bridges, aged 19 to 27 years. However, severe corrosion (noted in the previous section) was found in the remaining bridge, aged 17 years. This observation together with the enhanced chloride penetration evidence emphasizes the need for continuing monitoring of crack locations to allow for timely remedial action. [13]

A specialized model for corrosion forecasting at crack locations was developed. Damage forecasts were dominated by the incidence of cracks in the structures. While repairs are expected to be necessary at the crack locations, the projected fractional impact of corrosion at cracks on total maintenance needs was generally small unless a high incidence of cracks (e.g. several cracks per meter of waterline perimeter, an unusually high rate) took place.

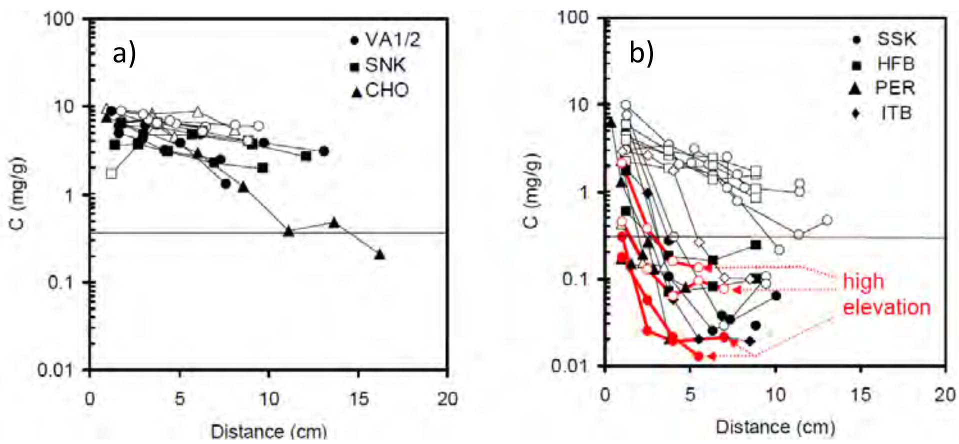

Figure 2: Progress of concrete corrosion damage as function of time. Structure age at the moment of evaluation per bridge code: VA1/2:26 years, SNK: 27 years, CHO: 29 years, SSK: 22 years, HFB: 17 years, PER: 27 years, ITB: 19 years. [13] Open symbols: sound concrete; Closed symbols: cracked concrete. Horizontal line: conservatively assumed initiation C_T.

2.3 Corrosion in post tensioned tendons

Failures involving full tendon separation took place in multistrand external grout-bonded tendons at early bridge ages, ranging from 4 to 20 years.[8] The failures, often in or near the anchorage region were associated with void spaces (hence leaving the steel unprotected) from deficient grouting and grout materials that promoted bleed water formation. After the first manifestations of this issue in Florida, comparable incidents were found elsewhere in the U.S. These incidents led to the FDOT and other agencies issuing updated and stricter specifications for PT practice, and to the development by industry of very low bleed grout formulations that were put in use subsequently. However, another set of early tendon corrosion failures (starting at 8 years bridge age) emerged recently in another major Florida PT bridge even though it was built using the new generation grout. [15] Work in progress indicates that other modalities of grouting deficiency including segregation and insufficient consolidation may have been corrosion triggering factors. Notably, high concentrations of sulfate ions in the grout pore water appear to have been a corrosion triggering factor even though the pore water pH remained high. The renewed incidence of PT corrosion is an important issue of concern and is being addressed by continuing investigation at present.

2.1 Corrosion modeling and design

Simplified deterministic calculations, based on straightforward initiation-propagation stage duration calculations that assume diffusional chloride transport, are used routinely by FDOT to compare the relative consequences of implementing changes in rebar cover and concrete quality in alternative design scenarios. For bridges that have been in service for one decade, chloride diffusion coefficients for the aggressive low elevation substructure regime are estimated using the following empirical relationship based on field surveys of Florida bridges:

$$D_{estimated} = 0.0645 \cdot \left(1 + \frac{w/ct - 0.32}{0.09}\right) \cdot \left(1 + \frac{444 - CF}{56}\right) \cdot F1 \cdot cm^2/y \qquad \text{Eq(1)[16]}$$

where CF is the cementitious factor in kg/m^3, w/ct is the water to cementitious ratio and F1 =1 if pozzolanic admixtures or blast furnace slag are present exceeding specified values, otherwise F1=3. See Ref. [16] for ranges of application. For more detailed forecasts of special cases, an approach with full probabilistic methodology is used by FDOT, assuming distributed values of chloride surface concentration and C_T, diffusion coefficient and concrete cover [6,12,13,17]. Normally, the average diffusion coefficient is conservatively assumed to be time-invariant fixed at its 10-yearr age value as provided by Eq (1).

FDOT is developing a new specialized probabilistic durability forecasting model that is expected to be implemented in the near future. That model will incorporate among other factors allowance for dependence of C_T on the prior activation history of the steel assembly, a feature not normally implemented in durability models. That feature is a result of evidence showing that C_T increases when the potential of passive steel is cathodically polarized, for example by coupling with other components of the rebar assembly that have experienced earlier corrosion initiation. [18-20] In such cases it has been shown that

$$log_{10}\left(\frac{C_T}{C_{To}}\right) \sim \frac{E_{To} - E}{\beta_{C_T}} \qquad \text{Eq. (2)[20]}$$

where E is the potential of the passive steel bar, β_{CT} is the cathodic prevention slope, and E_{TO} and C_{TO} are constants. Incorporating that feature in predictive models addresses the mutual corrosion aggravation and corrosion prevention effects due to macrocell coupling between active and passive steel components, which can be important in situations like marine bridge substructure. Recent corrosion-damage prediction model calculations using Eq.(2) have shown that those effects, if neglected, can result in overestimated projections of corrosion damage with consequently overly conservative design [20-23].

The highly aggressive service environment in Florida requires establishing demanding concrete materials and building specifications for corrosion control. Corrosion modes not seen elsewhere tend to be manifested early in this environment, as illustrated by the case histories noted for ECR, preferential chloride ingress at cracks and PT tendons. These occurrences create challenges but also the opportunity for development of information on the deterioration mechanisms, and advances in testing, specification and model-influenced design to be applied to advantage in the state and elsewhere.

ACKNOWLEDGEMENTS

This work was supported by the State of Florida Department of Transportation and the U.S. Department of Transportation. The findings and opinions presented here are those of the authors and not necessarily those of the supporting organizations.

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Edited by Dubravka Bjegović, Hans Beushausen and Marijana Serdar

RILEM Proceedings PRO 89
ISBN: 978-2-35158-135-3
e-ISBN: 978-2-35158-136-0
2013 Edition

RILEM TC 230-PSC was constituted in 2008 and grew to a membership of more than 45 experts from around the world. Several meetings took place over the years in Italy, France, the Netherlands, Germany, South Africa, Switzerland, and Croatia. The subject matter of the TC 230-PSC was to establish guidelines for the specification of the penetrability and thickness of the concrete cover, as a function of the exposure conditions and service life design, and for compliance control through suitable site and/or core testing. The development of guidelines for the application of suitable performance based approaches for concrete durability is an onerous task that will require further research into the calibration of suitable test methods and service life prediction models.

This conference represents the final event of the Technical Committee. It aims at bringing together experts from around the world, including researchers, practitioners, and infrastructure managers. It is anticipated that ongoing discussions and knowledge exchange will aid in developing guidelines for the application of suitable performance approaches for design, specification and quality control of concrete durability.

The editors wish to thank the International Scientific Advisory Board and authors for their efforts at producing and delivering papers of high standard. We are sure that the Proceedings will be a valued reference for many working in this important field and that they will form a suitable base for discussion and provide suggestions for future development and research.

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