

Experimental Investigation of FRCM-Concrete Interfacial Debonding

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Synopsis: This paper presents the results of an experimental study conducted to understand the stress-transfer mechanism of fiber reinforced concrete matrix (FRCM) composites externally bonded to a concrete substrate for strengthening applications. The FRCM composite was comprised of a polyparaphenylene benzobisoxazole (PBO) fiber net and polymer-modified cement-based mortar. Direct shear tests were conducted on specimens with composite strips bonded to concrete blocks. Parameters varied were composite bonded length and bonded width. Results were analyzed to understand the effective bonded length, which can be used to establish the load-carrying capacity of the interface to design the strengthening system. The normalized load carrying-capacity was plotted against the width of the composite strip to study the width effect. Finally, strain gage measurements along the bonded length were used to investigate the stress-transfer mechanism.

Keywords: FRCM, Concrete, Inorganic Matrix, Debonding.

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INTRODUCTION

Fiber-reinforced composite systems are increasingly used in civil engineering infrastructure applications for strengthening reinforced concrete structural members. A promising new type of composite comprised of fibers and an inorganic cementitious matrix presents several environmental, structural, and sustainability-related advantages over fiber-reinforced polymer (FRP) composites including better compatibility with the concrete substrate, improved freeze-thaw resistance, lesser influence of temperature and humidity on the composite performance, and simpler construction. Studies in the literature on the behavior of externally-bonded fiber reinforced cementitious matrix (FRCM) composites show that while they can be used successfully in strengthening applications¹⁻¹³, their performance is different from FRP composites due to differences in debonding failure mechanisms resulting from complex matrix-fiber bonding characteristics. Debonding failures are critical in strengthening applications since they are generally brittle in nature and usually control the overall performance of the system by triggering global member failure. With FRP composites, it is well known that debonding typically occurs within the concrete substrate. However, limited available research on debonding of FRCM composites suggests that debonding occurs within the matrix as a progressive process resulting in large slips at the matrix-fiber interface^{1,5,6,14-16}, which entails for increased ductility as compared to FRP composites¹⁴. In general, the matrix serves several critical purposes: it transmits and distributes shear forces between and along the fibers, and it bonds the composite to the concrete substrate, which is necessary for load sharing. A complete understanding of the mechanism of interfacial stress transfer of FRCM composites externally bonded to concrete is critical to design and has not yet been thoroughly examined. This paper presents the results of an experimental study conducted to understand the stress-transfer mechanism of FRCM composites externally bonded to a concrete substrate. The FRCM composite was comprised of poly(paraphenylene benzobisoxazole) (PBO) fiber net and polymer-modified cement-based mortar. Results from single-lap shear tests with different bonded lengths and bonded widths are presented and discussed.

EXPERIMENTAL PROGRAM

The experimental program included single-lap (direct) shear tests conducted on concrete block (prism) specimens with an externally-bonded FRCM composite strip. The parameters varied were the bonded length and bonded width of the composite strip. The classical push-pull configuration was adopted in which the composite fibers were pulled while the concrete prism was restrained (Figs. 1 and 2). The dimensions of the concrete prisms were 125 mm width x 125 mm depth x 375 mm length (5 in. x 5 in. x 15 in.). The composite material was comprised of a bidirectional PBO fiber net with longitudinal and transverse fiber bundles and cementitious matrix. The nominal width b^* and average thickness t^* of one longitudinal fiber bundle were 5 mm (0.2 in.) and 0.092 mm (0.0036 in.), respectively. Longitudinal fiber bundles are pointed out in Fig. 2. The matrix was applied only in the bonded region to embed the fibers and bond the composite to the concrete substrate. Fibers were bare outside the bonded area. Two aluminum plates were attached to the end of the fiber strip with a thermosetting epoxy to grip the fibers during testing (Fig. 1). A steel frame that was bolted to the testing machine base was used to restrain the concrete prism. A steel plate was inserted between the steel frame and the top of the prism to distribute the pressure provided by the frame restraint to the concrete prism. Dimensions of the frame are shown in Fig. 1. Tests were conducted under displacement control using a close-loop servo-hydraulic universal testing machine with a 556 kN (125 kip) force and +/- 150 mm (6 in.) stroke capacity. During testing the global slip, defined as the relative displacement between points on the composite strip just outside the bonded area and the concrete prism, was increased at a constant rate of 0.00084 mm/s (0.00033 in./s). Global slip was measured using two linear variable displacement transducers (LVDTs) that were attached to the concrete surface near the edge of the bonded region. The LVDTs reacted off of a thin aluminum Ω -

shape bent plate that was attached to the PBO transversal fiber bundle surface adjacent to the beginning of the bonded region as shown in Figs. 1 and 2. The average of the two LVDT measurements was used to control the rate.

The concrete prisms were constructed with normal-weight concrete with portland cement (Type 1) without admixtures. The maximum size of the aggregate was 9.5 mm (0.375 in.). Six 100 mm × 200 mm (4 in. x 8 in.) concrete cylinders were cast from the same batch of concrete to determine the concrete compressive strength and splitting tensile strength in accordance with ASTM C39¹⁷ and ASTM C496¹⁸. Material properties are provided in Table 1.

From the same batch of matrix used to cast the FRCM composite, ten 50 mm × 100 mm (2 in. x 4 in.) cylinders were cast to determine the compressive and tensile strengths of the matrix in accordance with ASTM C39¹⁷ and ASTM C496¹⁸. Results are provided in Table 1. Uniaxial tension tests were conducted on fiber samples. Samples with one, four, five, and seven longitudinal fiber bundles were tested, with at least three replicate samples. Uniaxial electrical resistance gages were mounted on the central fiber bundle of several specimens to measure the applied load-strain relation. The maximum force divided by the area of longitudinal fibers was similar irrespective of number of bundles. Table 1 reports the ultimate strength, ultimate strain, and elastic modulus taken as the average of the samples tested. Values of ultimate strength, ultimate strain, and elastic modulus of the PBO fiber reported by the manufacturer are 5.8 GPa (840 ksi), 0.025 and 270 GPa (39,000 ksi), respectively¹⁹. The values obtained from the tension tests were substantially lower than those reported by the manufacturer, although measured values were quite consistent. However, it should be noted that the methodology used by the manufacturer to test the mechanical properties was different from that used in this study²⁰. Thirty-three direct shear tests were performed to study the bond characteristics and stress-transfer mechanism of the FRCM composite. The parameters varied were the bonded length and bonded width of the composite. At least three replicates of each combination of parameters were tested. Specimens were named following the notation DS_X_Y(S)_Z, where X=bonded length (ℓ) in mm, Y=bonded width (b_1) in mm, S indicates that strain gages were mounted on the specimen, and Z=specimen number (Table 2). The number of longitudinal bundles n is indicated in Table 2.

The surface of the concrete prisms was sandblasted before applying the composite. A layer of cementitious matrix was then applied using molds to control the composite width and thickness. A single layer of PBO fiber net was then applied onto the matrix layer pushing the fibers delicately to assure proper impregnation. The fiber net strip was positioned such that it extended slightly beyond the end of the matrix at the unloaded end as shown in Fig. 2. A second layer of matrix was then applied over the PBO fibers. Each specimen was allowed to cure for at least one week before testing. The thickness of each of the two layers of matrix was 4 mm (0.15 in.) as recommended by the manufacturer¹⁹. The total thickness of composite t was 8 mm (0.3 in.) as indicated in Table 2.

Five specimens were instrumented with uniaxial electrical resistance strain gages (gage length = 1 mm [0.04 in.]) to study the axial strain distribution along the bonded length of composite. The positions of the strain gages are shown in Fig. 3a. Gages 4-7 were mounted to the fibers along the bonded length of the composite, and Gages 1-3 were mounted to the fibers outside the bonded length. For specimens DS_330_43_S_3, DS_330_43_S_4, and DS_330_43_S_5, Gages 1 and 3 were omitted. All gages were mounted to longitudinal fibers. Two different techniques were used to apply the strain gages to the fibers along the bonded length. For specimens DS_330_43_S_1 and DS_330_43_S_2, slots were created during the application of the top layer of matrix in the locations of the strain gages. Strain gages were then applied to the fibers after the composite set (Fig. 3b). For specimens DS_330_43_S_3, DS_330_43_S_4, and DS_330_43_S_5, strain gages were mounted to the fiber bundles and then embedded in the top layer of matrix (Fig. 3c).

DISCUSSION OF RESULTS

General behavior

Specimens were tested until one of the following conditions occurred: a sudden and drastic reduction in applied load, or considerable slippage between fibers and matrix. In general no damage was observed at the matrix-concrete interface except for specimens DS_100_34_1 and DS_100_34_2. The authors postulate that a Mode-I condition prevailed in these two tests due to the short bonded length adopted²¹. With the exception of specimens DS_100_34_1 and DS_100_34_2, debonding occurred at the matrix-fiber interface. As global slip increased, longitudinal fiber bundles were observed to gradually pull out of the composite at the loaded end of the bonded surface, and longitudinal fibers beyond the end of the bonded length advanced slowly into the matrix (position $y=0$

in Figure 3a). In many tests, the bare fiber net at the loaded end of the specimen exhibited nonuniform load-sharing among the longitudinal bundles with increasing slip. This was evidenced by global rotation of the Ω -shape bent plate, as well as by deformation observed in the transversal fiber bundles, which were orthogonal to the longitudinal fibers at the start of the test. This observation suggests that redistribution of stress was occurring between longitudinal fiber bundles throughout the test. Some specimens had preexisting shrinkage cracks on the composite surface, especially specimens with strain gages (see Fig. 3b); these cracks opened with increasing slip. The cracks eventually penetrated the thickness of the composite, as could be seen from the side of the specimens. The presence of through-thickness cracks resulted in a discontinuity in the stress transfer between fibers and matrix with consequent localized deformation at the crack locations along the composite bonded length. Cracks were not observed in specimens that did not have preexisting shrinkage cracks, which suggests a more uniform stress distribution along the composite bonded length, and that failure is controlled by slippage of fibers.

Maximum load and load-global slip response

The maximum load P^* is reported for each test specimen in Table 2. Scatter in the values of P^* can be explained in part by the non-uniform load-sharing among fiber bundles as discussed previously in the description of general behavior. Typical load P -global slip responses for different bonded lengths and widths are shown in Fig. 4. In general, a linear response is followed by a non-linear response up to the peak load. The descending post-peak response is characterized by slippage of the fibers with respect to the matrix. As mentioned previously, tests were terminated when considerable slippage between fibers and matrix was recorded. The stress-transfer mechanism for FRCM composites, including the role of the matrix on each side of the fiber net, is not yet understood. Because the application of strain gages introduced interruptions in the matrix top layer, two different applications were attempted, and the load response relations were compared. Load responses of the specimens with strain gages are reported in Fig. 5. The responses of specimens without strain gages, DS_330_43_1 and DS_330_43_5, are also plotted in Fig. 5 for comparison. The maximum applied load for specimens DS_330_43_S_1, DS_330_43_S_2, and DS_330_43_S_5 is consistent with the results previously discussed. However, in specimens DS_330_43_S_1 and DS_330_43_S_2, the non-linear pre-peak response appears to be more emphasized. It is possible that the slots used to mount the strain gages on specimens DS_330_43_S_1 and DS_330_43_S_2 induced a stress concentration at the gage locations or modified the restraining action of the matrix, which highlights the need to investigate the role of the top layer of matrix. The load response of specimen DS_330_43_S_4 exhibited a sharp decrease in applied load due to localized stretching of the fibers outside the bonded region. In this case the non-uniform distribution of load in the longitudinal bundles caused a localized stress peak leading to the failure of one or more bundles.

Influence of bonded length

Figure 6 shows the relation between maximum load P^* and bonded length ℓ of the composite for the series of test specimens with composite width b_1 of 34 mm (1.3 in.) (corresponding to $n=4$ bundles). An increasing trend can be seen between maximum load and bonded length, and similar to FRP-concrete joints, increasing bonded lengths result in a less than proportional increase in maximum load. For FRP-concrete joints, an effective bond length l_{eff} , defined as the minimum length of the bonded area in the direction of the fibers to fully establish the load-carrying capacity of the interface, can be determined from this type of relation as the length beyond which the maximum load remains constant. The maximum load associated with l_{eff} is the debonding force. If these same definitions and relations hold for FRCM-composite joints, results from Figure 6 suggest that the effective bond length l_{eff} is in the range of 250 to 350 mm (10 to 13 in.), if in fact it exists for this composite. Experimental results reported by D'Ambrisi et al.¹⁶ using the same composite tested with double-lap shear tests suggested that the effective bond length is approximately 250 to 300 mm (10 to 12 in.), although it should be noted that the longest bonded length tested in that study was 250 mm (10 in.). Further investigation of the effective bond length will be conducted by the authors in the near future including investigation of longer composite bonded lengths and the influence of other stress-transfer mechanisms such as friction. For comparison, the effective bond length l_{eff} was computed using the formulation provided in ACI 440.2R-08²² for the FRP-concrete interface. Using this approach, however, the computed effective bond length of the FRCM-concrete interface was found to be significantly underestimated. A possible reason is the different debonding mechanism between the FRP-concrete and FRCM-concrete interfaces.

Figure 7 depicts the maximum load P^* normalized with respect of the total width of the longitudinal fiber bundles nb^* versus bonded length ℓ of composite for all specimens with the exception of those with strain gages. Specimens with $n=4, 5$, and 7 bundles are plotted in the graph. Similar to Fig. 6, the normalized maximum load increases for the entire range of bonded lengths tested, and increasing bonded lengths result in a less than proportional increase in normalized maximum load.

Influence of bonded width

Figure 8 shows the relation between maximum load P^* normalized with respect of the total width of the longitudinal fiber bundles nb^* versus composite bonded width b_1 for test specimens with the same bonded length ($\ell=330$ mm [13 in.]). Specimens with strain gages were omitted from the graph. Three different bonded widths are shown, namely 34 mm (1.3 in.), 43 mm (1.7 in.), and 60 mm (2.4 in.). Results show that specimens with different bonded widths have a similar normalized maximum load. This observation is confirmed by results reported by D'Ambrisi et al.¹⁶ of test specimens with the same composite and with a bonded width of 100 mm tested with double-lap shear tests. These results suggest that a width effect does not exist for this type of composite. FRP composite, on the other hand, has been shown to exhibit a width effect^{23,24}. FRP and FRCM have several mechanical differences that might explain this difference in phenomenon, such as fiber layouts (sheets with continuous fibers across the width, versus net with discrete fiber bundles across the width) and bonding characteristics of the matrix. The limited data in Fig. 8 also exhibit greater scatter with smaller bonded width. This may be due to limited force redistribution capability in fiber nets with fewer than a certain critical number of bundles. As discussed previously, redistribution of forces was observed in the fiber bundles throughout the test. More data are needed to further investigate this phenomenon.

Measured strain

Axial strains along the direction of the longitudinal fibers ϵ_{yy} in the central and edge bundles of specimen DS_330_43_S_1 recorded by gages 1, 2, and 3, outside the bonded region, are reported in Fig. 9. Filled markers indicating the average values are shown in the figure, and coefficient of variation values are given in parentheses. Considering the average values of strain, Figure 9 shows that the applied load and the strain in the longitudinal bundles outside the bonded region is approximately linear. If the average stress of the bundles is computed, the results can be used to calculate the elastic modulus of the fibers. Values computed confirm the value determined from the tension tests discussed previously. Figure 9 also shows that a non-uniform strain distribution is observed among the three bundles that were instrumented with strain gages. A similar phenomenon is observed in FRP strips attached to concrete, and it is partially due to the local variation of the interfacial properties. In the case of discrete fiber bundles this phenomenon appears to be more pronounced. The non-uniform strain distribution may also be partially due to a slight eccentricity of the applied load. For load levels less than 50% P^* , it can be seen that the rate of change in strain with increasing applied load is approximately the same for the three bundles instrumented. For load levels higher than 50% P^* , the rate of change in strain is different. This behavior supports the visual observations of non-uniform load sharing of bundles discussed previously and suggests that load redistribution occurs among fiber bundles with increasing slip, even at load levels less than the peak load. The variation of the strain in specimen DS_330_43_S_1 at different locations along the bonded length for different values of the load is depicted in Fig. 10a. Location along bonded length y is defined in Fig. 3a. Note that strain gage 9 was damaged prior to testing, so it is not shown in the figure. Five values of the load, corresponding to five points (A1, B1, C1, D1, and E1) of the load response in Fig. 10b, were considered. The strain profiles of Fig. 10a resemble the profiles obtained from similar tests for FRP-concrete joints^{23,24}. This observation suggests that for FRCM-concrete interfaces a cohesive material interfacial law can be obtained. It should be noted that the limited points along the bonded length where strains were measured might lead to an erroneous interpretation of the readings. Additional measurements are planned for future tests to verify the strain profiles and determine if a cohesive material law similar to that used for the FRP-concrete interface can be adapted to the description of the matrix-fiber interface in FRCM composites.

CONCLUSIONS

This paper describes the results of experimental research conducted to study the stress-transfer mechanism of fiber reinforced concrete matrix (FRCM) composites externally bonded to a concrete substrate. The FRCM composite was comprised of a polyparaphenylene benzobisoxazole (PBO) fiber net and polymer-modified cement-based mortar. Direct shear tests were conducted on specimens with composite strips bonded to concrete blocks. Parameters varied were composite bonded length and bonded width. Based on the results of this study, the following conclusions can be made:

1. Debonding of the FRCM composite occurred at the matrix-fiber interface rather than the matrix-concrete interface.
2. For the range of composite bonded lengths tested (100 to 330 mm), an increasing trend was observed between maximum load and bonded length. Similar to FRP-concrete joints, increasing bonded lengths resulted in a less than proportional increase in maximum load. Results obtained thus far suggest that the effective bond length is in the range of 250 to 350 mm (10 to 13 in.), if in fact it exists for this composite.

3. Although a width effect was not observed, specimens with smaller bonded widths exhibited greater scatter with respect to maximum load. This may be due to limited force redistribution capability in fiber sheets with fewer than a certain critical number of bundles.
4. The strain distribution along the bonded length resembles the strain distribution typical of FRP strips bonded to a concrete substrate. Further investigation is needed to determine the existence or value of an effective bond length for this composite based on the strain profiles.

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Table 1—Material Properties

Concrete Prism	
Compressive strength MPa (psi); <i>COV</i>	42.5 (6160) 0.013
Splitting tensile strength MPa (psi); <i>COV</i>	3.4 (490) 0.113
FRCM Composite	
Mortar	
Compressive strength MPa (psi); <i>COV</i>	27.9 (4050) 0.009
Splitting tensile strength MPa (psi); <i>COV</i>	3.6 (520) 0.072
PBO Fibers	
Ultimate strength GPa (ksi); <i>COV</i>	3.0 (430) 0.068
Ultimate strain <i>COV</i>	0.0145 0.104
Elastic modulus GPa (ksi); <i>COV</i>	206 (29,900) 0.065

Table 2—Test Specimens

Name	Composite Width b_1 mm (in.)	Number of Bundles n	Composite Length ℓ mm (in.)	Composite Thickness t mm (in.)	Maximum Load P^* kN (k)
DS 100 34 1	34 (1.3)	4	100 (4)	8 (0.3)	1.92 (0.43)
DS 100 34 2	34 (1.3)	4	100 (4)	8 (0.3)	0.97 (0.22)
DS 100 34 3	34 (1.3)	4	100 (4)	8 (0.3)	1.62 (0.36)
DS 150 34 1	34 (1.3)	4	150 (6)	8 (0.3)	2.22 (0.50)
DS 150 34 2	34 (1.3)	4	150 (6)	8 (0.3)	1.55 (0.35)
DS 150 34 3	34 (1.3)	4	150 (6)	8 (0.3)	2.87 (0.65)
DS 150 34 4	34 (1.3)	4	150 (6)	8 (0.3)	2.34 (0.53)
DS 200 34 1	34 (1.3)	4	200 (8)	8 (0.3)	3.05 (0.69)
DS 200 34 2	34 (1.3)	4	200 (8)	8 (0.3)	2.52 (0.57)
DS 200 34 3	34 (1.3)	4	200 (8)	8 (0.3)	3.44 (0.77)
DS 250 34 1	34 (1.3)	4	250 (10)	8 (0.3)	2.61 (0.59)
DS 250 34 2	34 (1.3)	4	250 (10)	8 (0.3)	2.11 (0.47)
DS 250 34 3	34 (1.3)	4	250 (10)	8 (0.3)	2.82 (0.63)
DS 330 34 1	34 (1.3)	4	330 (13)	8 (0.3)	3.00 (0.67)
DS 330 34 2	34 (1.3)	4	330 (13)	8 (0.3)	3.51 (0.79)
DS 330 34 7	34 (1.3)	4	330 (13)	8 (0.3)	4.07 (0.91)
DS 330 34 8	34 (1.3)	4	330 (13)	8 (0.3)	4.02 (0.90)
DS 330 34 9	34 (1.3)	4	330 (13)	8 (0.3)	3.44 (0.77)
DS 330 43 1	43 (1.7)	5	330 (13)	8 (0.3)	4.43 (1.00)
DS 330 43 2	43 (1.7)	5	330 (13)	8 (0.3)	5.25 (1.18)
DS 330 43 3	43 (1.7)	5	330 (13)	8 (0.3)	5.27 (1.18)
DS 330 43 5	43 (1.7)	5	330 (13)	8 (0.3)	4.79 (1.08)
DS 330 43 6	43 (1.7)	5	330 (13)	8 (0.3)	5.09 (1.14)
DS 330 43 S 1	43 (1.7)	5	330 (13)	8 (0.3)	4.48 (1.01)
DS 330 43 S 2	43 (1.7)	5	330 (13)	8 (0.3)	5.12 (1.15)
DS 330 43 S 3	43 (1.7)	5	330 (13)	8 (0.3)	3.03 (0.68)
DS 330 43 S 4	43 (1.7)	5	330 (13)	8 (0.3)	4.60 (1.03)
DS 330 43 S 5	43 (1.7)	5	330 (13)	8 (0.3)	4.03 (0.91)
DS 330 60 1	60 (2.4)	7	330 (13)	8 (0.3)	7.05 (1.59)
DS 330 60 2	60 (2.4)	7	330 (13)	8 (0.3)	6.56 (1.47)
DS 330 60 3	60 (2.4)	7	330 (13)	8 (0.3)	6.06 (1.36)
DS 330 60 4	60 (2.4)	7	330 (13)	8 (0.3)	6.50 (1.46)
DS 330 60 5	60 (2.4)	7	330 (13)	8 (0.3)	6.28 (1.41)

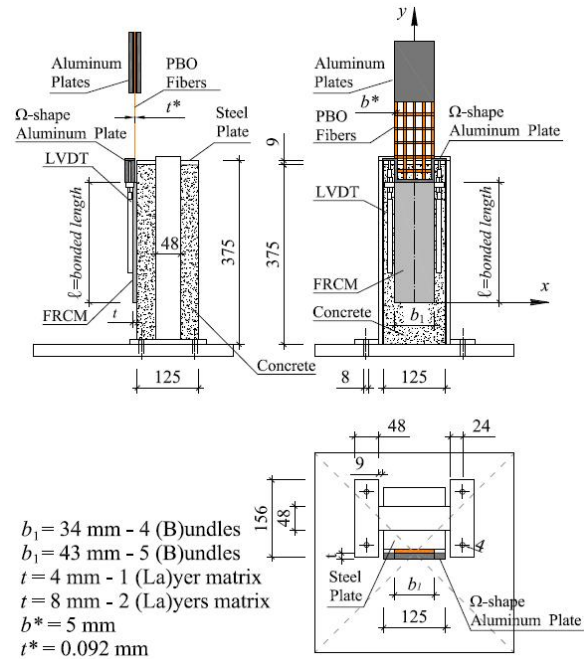


Figure 1 — Test setup (dimensions in mm; 1 mm = 0.0394 in.)

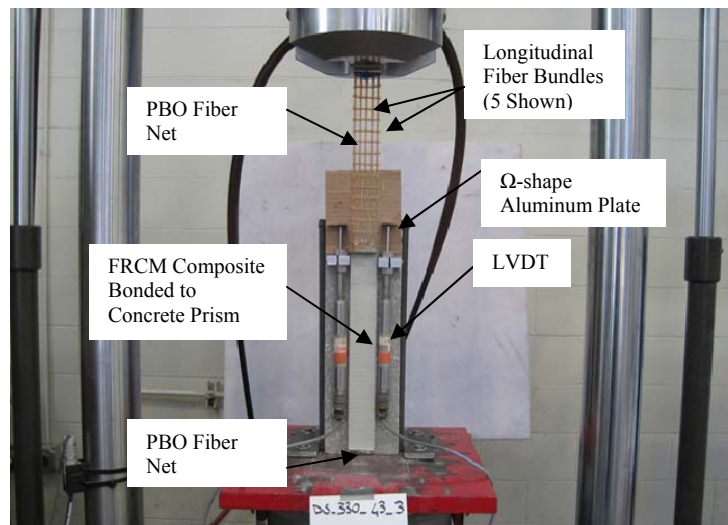


Figure 2 — Photo of specimen DS_330_43_3

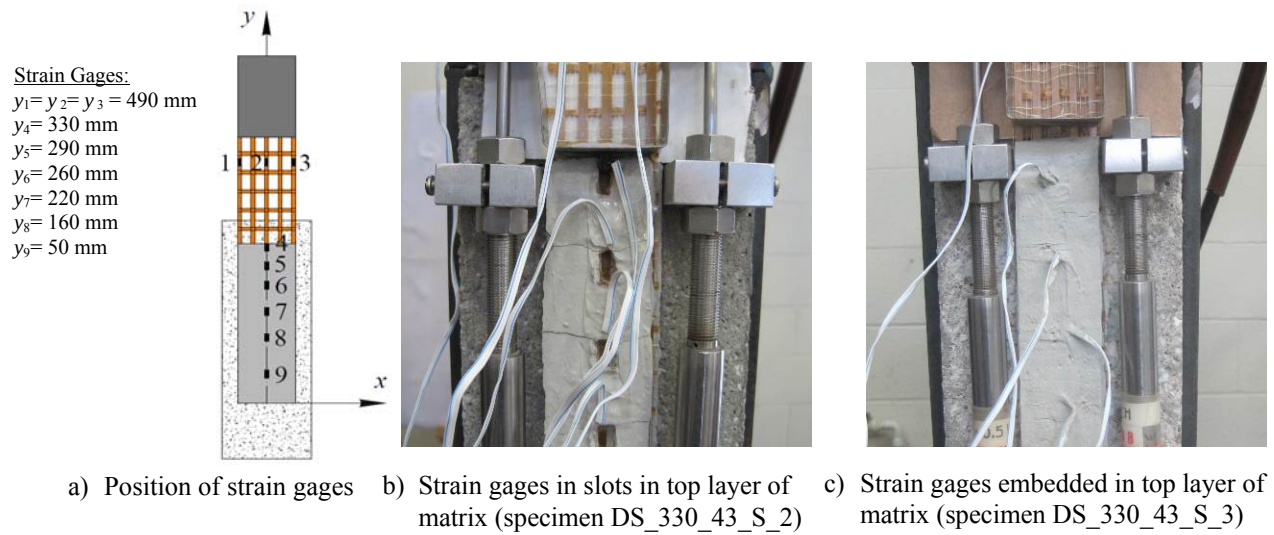


Figure 3 — Strain gage position and application

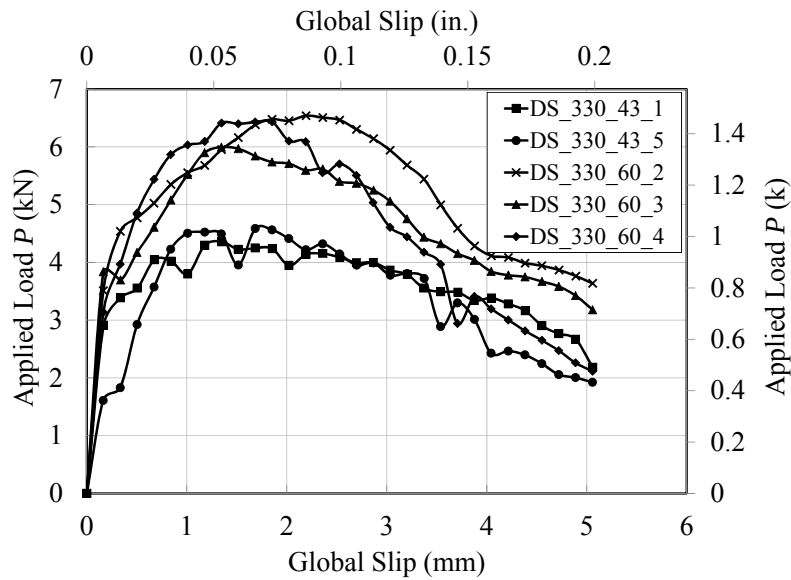


Figure 4 — Typical load-slip responses

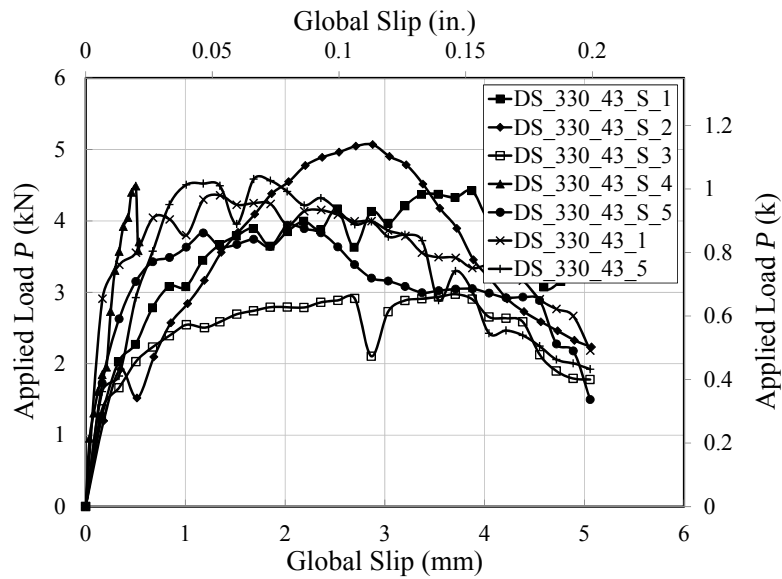


Figure 5 — Load slip responses for specimens with strain gages

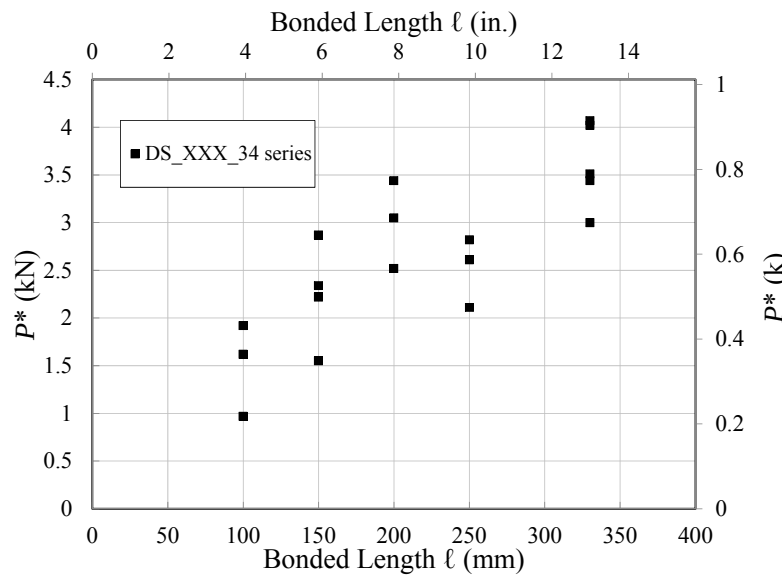


Figure 6 — Load versus bonded length

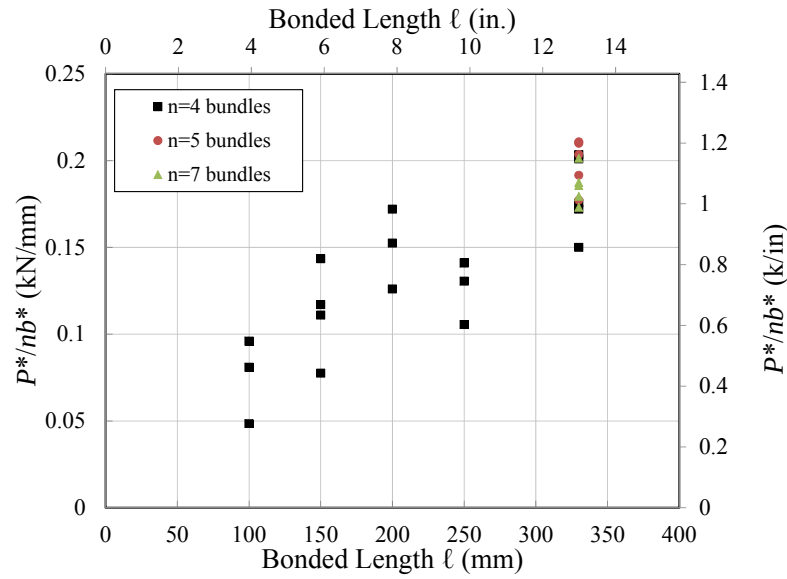


Figure 7 — Normalized load versus bonded length

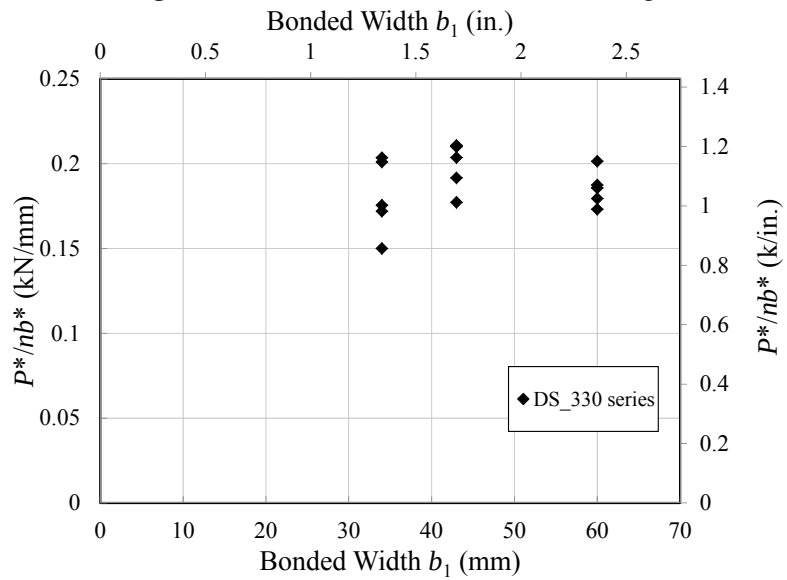


Figure 8 — Normalized load versus bonded width

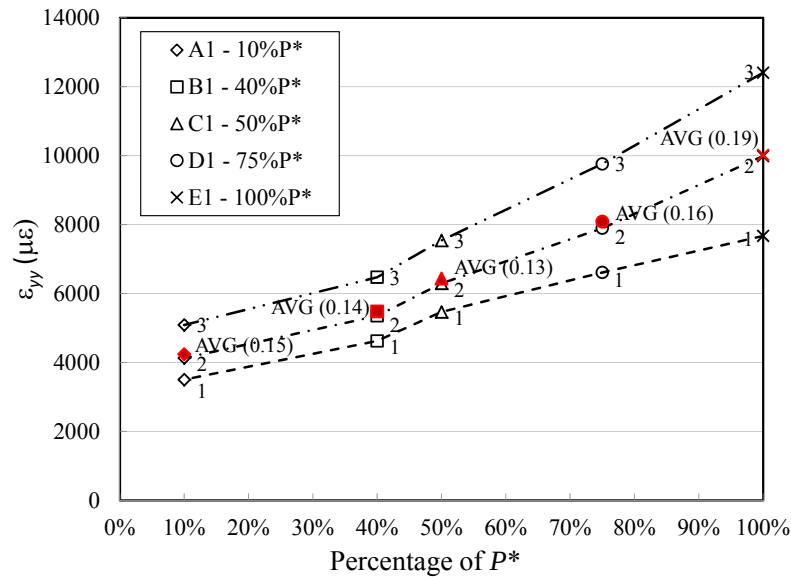


Figure 9 — Strain in central and edge bundles outside the bonded region of specimen DS_330_43_S_1

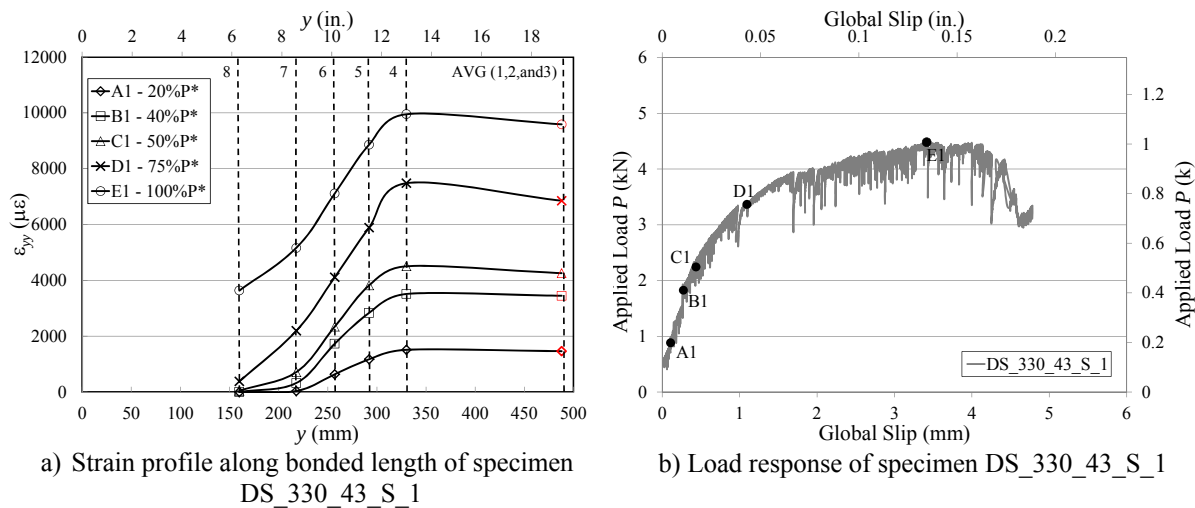


Figure 10 — Strain profile along bonded length of specimen DS_330_43_S_1

Acoustic Emission Corrosion Monitoring of Prestressed Concrete Bridge Members

William Vélez, Mohamed K. ElBatanouny, Fabio Matta, and Paul H. Ziehl

Synopsis: Corrosion of prestressing steel strands in concrete bridge members may lead to the rupture of single strands and catastrophic collapse before visual inspection uncovers evident signs of damage, and well before the end of the design life. Recognizing corrosion in its early stage is critical to plan maintenance and repairs and prevent premature failures. The acoustic emission (AE) technique is a rational means to develop structural health monitoring and prognosis systems for the early detection and location of corrosion in prestressed concrete. It is sensitive to micro- and macro-damage, non-intrusive, and suitable for remote monitoring. However, there is little understanding of the correlation between AE and the extent of early damage in prestressing strands. This paper presents recent advances in AE monitoring of corrosion for prestressing strands embedded in concrete. The state of the art is reviewed, and results from recent research efforts are reported, in which prestressed concrete specimens representative of scaled bridge girders and piles were exposed to salt water through wet/dry cycles. The acoustic emission activity resulting from the early corrosion of strands was studied by evaluating AE data vis-à-vis electrochemical measurements and evidence from scanning electron microscopy.

Keywords: acoustic emission, corrosion, prestressed concrete, structural health monitoring

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INTRODUCTION

Corrosion of prestressing strands is one of the main factors contributing to premature deterioration of prestressed concrete (PC) bridges, and poses structural safety issues. For instance, when corrosion takes place in a PC girder, strand fracture may occur before any visual signs can be detected. If the problem is not identified and addressed in a timely manner, the girder may collapse due to strand fracture, as evidenced by well-known cases such as the collapses of a fascia girder at the I-70 overpass in Pennsylvania and an entire deck failure of the Ynys-y-Gwas bridge in the UK¹. Acoustic emission (AE) monitoring is a non-destructive testing technique that can be effective in detecting the corrosion of prestressed steel strands in PC. Corrosion of steel in concrete has been shown to be a source of elastic waves that can be detected and located through AE monitoring. While the mechanisms producing acoustic emission at early stages of corrosion are not clear, experimental evidence suggests that the accumulation of corrosion byproducts at the interface between steel and concrete and micro-crack nucleation produce the most likely AE sources². AE monitoring of concrete bridges has been the subject of several studies, most of which focused on damage quantification during load tests and corrosion detection in reinforced concrete (RC) structures. Corrosion of prestressing steel, on the other hand, has received less attention. The detection of strand fracture using AE has been the subject of a few studies. Yuyama et al.³ conducted laboratory and field tests in which detection and location of strand fracture were attempted. In laboratory tests, three types of specimens were artificially corroded via impressed electric current: unbonded (without grout), partially grouted, and fully grouted. In the field, a T-beam bridge and a box-girder bridge were monitored for 24 days. AE activity was generated from artificial strand fracture in small post-tensioned specimens attached to the bridge in the parallel and perpendicular direction with respect to the girders. Laboratory results showed that source location was successful in unbonded and partially grouted specimens, whereas full grout was observed to affect source location accuracy. In the field, most strand breaks were identified. Linear source location on T-beams was successful, whereas it was not achieved in the box girders due to wave attenuation and complex wave paths resulting from the attached specimens being at the bottom of the box girder instead of on the sides, where the AE sensors were mounted. AE signals from traffic and hammer blows were found to be discernible from strand break signals. In particular, traffic-induced signals were observed to have lower amplitude and longer duration. Youn et al.⁴ subjected a post-tensioned concrete beam with grouted tendons to corrosion by drilling a hole through the concrete, creating a corrosive reservoir using NaCl and NaOH. Anodic polarization was imparted using a potentiostat, and the corrosion process was monitored with four AE sensors and a vibrating wire gauge. Strand fracture was detected from AE signals and confirmed by visual inspection. Fricker and Vogel⁵ described the implementation and testing of a monitoring system installed on a PC bridge in Switzerland where piezoelectric accelerometers were used to detect wire breaks. The performance was assessed by conducting blind tests where two artificial strand breaks were induced using a corrosion cell with salt water and hydrochloric acid, and powered with a car battery. The outcome from the system was compared with results from destructive inspection. The system detected all the artificial wire breaks as well as spontaneous ones. Among 11 spontaneous breaks, three were confirmed through invasive inspection. The spontaneous breaks were located by the system with errors between 3.9 and 23.6 in (100 and 600 mm). The two artificial breaks were detected and located with an accuracy of 6.3 and 26.0 in (160 and 660 mm), respectively.

The feasibility of deploying AE sensors to detect corrosion at early stages, and before strand rupture occurs, was explored under laboratory conditions. Perrin et al.⁶ subjected a medium scale post-tensioned concrete beam with one reinforcing strand to corrosion in ammonium thiocyanate (NH_4SCN) at 50°C . AE activity was monitored using sensors located on the strand near an exposed area, at the anchorages, strand ends, and on the concrete surface. Four stages in the damage process were identified using the AE sensor located near the exposed area, namely: production of hydrogen bubbles, crack initiation, hydrogen embrittlement, and steel crack growth till failure. A sensor placed on one end of the strand identified the four stages as well, and a sensor on one of the anchorage heads identified the crack propagation stage. Ramadan et al.⁷ exposed a tensioned strand to corrosion in a simulated pore solution contaminated with chloride, sulphate and thiocyanate ions. AE sensors were placed on the strand and on the anchorages, and the AE activity was recorded until fracture of the strand occurred. Although the onset of corrosion (i.e., steel depassivation) was not detected, AE amplitude, frequency content, and signal energy were related to three stages of the corrosion process: crack initiation due to local corrosion, crack growth and propagation, and fracture. This paper presents research on AE sensing aimed at monitoring early corrosion of prestressing strands in PC. An experimental program comprising corrosion tests on PC specimens representative of scaled bridge girders and piles is presented. The evidence discussed includes results from AE monitoring, benchmark electrochemical measurements, and visuals from photos and scanning electron microscopy (SEM) micrographs from strands that were removed from the specimens. The application of acoustic emission Intensity Analysis (IA) for corrosion assessment is also discussed.

METHODOLOGY

Specimens

Two prestressed and two non-prestressed pile specimens were included in this study. The specimens were designed to have material properties and dimensions that would allow for stresses to be similar to those of actual piles, and for corrosion to result from the same mechanisms that cause corrosion at the splashing zone. The following notations are used to identify the specimens: P1 and P2 are similarly designed and built prestressed specimens, and N1 and N2 are also similar specimens having an embedded non-prestressed strand. Figures 1(a) and 1(b) show the geometry of the specimens. Each had an embedded Grade 270 steel seven-wire strand with a 13 mm (0.5 in) nominal diameter. The strands in the two PC specimens, P1 and P2, were prestressed to 70% of their breaking strength. Stirrups were not provided so that corrosion of strands could be isolated, as the characteristics of AE data from corroding strands are not well understood. Type I Portland cement concrete with a 0.48 w/c ratio and an average 28-day cylinder compressive strength of 6238 psi (43 MPa) was used on prestressed and non-prestressed specimens. To expedite the onset of corrosion, each strand was placed eccentrically with respect to the centroid of the cross section to reduce the concrete cover. A 1.0 in (25 mm) wide groove was cut along the 16 in (406 mm) length to be exposed to a NaCl solution, leaving a 0.5 in (25 mm) cover. Details of the design for strength and time-to-corrosion (chloride penetration) are provided elsewhere⁸. The scaled PC girder specimens included two T-section beams with a length of 16 ft and 4 in (5.0 m), as illustrated in Figures 2(a) and 2(b). Type I Portland cement concrete was used, having a 0.4 w/c ratio and a 28-day cylinder compressive strength of 4200 psi (29 MPa). Two Grade 270 steel strands were used in each specimen, with a clear cover of 2 in (51 mm). The strands were prestressed to 75% of their breaking strength. The strands were slightly pre-corroded with a mass loss of 0.35 oz/ft (33 g/m). To accelerate the corrosion process when exposed to chlorides, the girders were pre-cracked at the mid-section, imposing a maximum crack width of 0.016 and 0.032 in (0.4 and 0.8 mm), respectively, using plastic inserts, as detailed elsewhere⁹.

Test setup and instrumentation

Figures 1(c) and 1(d) show the test setup for the pile specimens. Acrylic sheets were mounted between the specimens to form a pool, which was partially filled with salt water with 5% NaCl concentration. The specimens were subjected to three-day wet/four-day dry cycles by raising and lowering the water level between 8 and 15 in (203 and 381 mm). A 1.0 in (25 mm) thick clay layer was attached to the side and outer face of each specimen to simulate continuity and minimize the effect of AE wave reflections. A vibrating wire strain gauge was installed on each strand prior to casting to monitor prestress loss. Figures 2(b) and 2(c) show the test setup for the beam specimens. Acrylic sheets were used to build a dike on the mid-section where the cracks were located. The specimens were subjected to three-day wet/four-day dry cycles using salt water with 3% NaCl concentration. Acoustic emission was monitored continuously using piezoelectric AE sensors (preamplified at 40 dB and with an operating frequency range of 40-100 kHz). Two pile specimens were instrumented with two sensors each as shown in Figure 1(c), and two with one sensor each 2 in (51 mm) away from the maximum water level. The beams were monitored with four AE sensors, as shown in Figures 2(b) and 2(c), which were attached using an epoxy adhesive. For all specimens, background noise was recorded for one week before starting the tests by exposing them to the

NaCl solution. Based on the observed behavior, the threshold for acquisition was set at 40 dB. Relevant AE acquisition parameters are listed in Table 1. For all specimens, half-cell potential measurements per ASTM-C876¹⁰ and linear polarization resistance (LPR) tests were routinely performed using a 263A potentiostat/galvanostat (Model 263A, Princeton Applied Research, Oak Ridge, TN). The steel strands acted as the working electrodes, while a copper plate immersed in the salt water (electrolyte) and a Cu/CuSO₄ electrode were used as the counter and reference electrode, respectively. A sweep rate of 0.1 mV/s was used for the LPR tests¹¹. The electric connection to the steel strand in the pile specimens was made through an opening close to the polarized area to minimize the likelihood of electrical discontinuity. In the beam specimens, a copper rod protruding from the concrete and connected to the strands near the polarized area was used for the electric connection. The electrochemical tests were carried out at the end of the wet cycles to ensure good conductivity throughout the concrete cover.

RESULTS AND DISCUSSION

Pile specimens

Figure 3(a) shows the half-cell potential measurements, E , as a function of time during 300 days of exposure. In this period, specimens P1 (prestressed) and N2 (non-prestressed) showed signs of active corrosion according to ASTM C876 at 209 days and 161 days, respectively. Figure 3(b) shows the measured polarization resistance, R_p , as a function of time. It can be seen that the potential shift coincided with a steep decrease in polarization resistance at or below 500 Ω . Conversely, E from specimens N1 and P2 remained above the ASTM C876 threshold for active corrosion throughout the test, and R_p did not show any sustained drop, indicating that no depassivation occurred. It is not surprising that not all specimens experienced corrosion under nominally identical conditions, as it has been recognized that significant variability in critical chloride concentrations, and therefore in time to corrosion, should be expected due to the influence of random microscopic and macroscopic characteristics of the steel-concrete interface¹², as well as imperfections on strand surfaces¹³. Figure 4 shows the relation between measured prestress losses and age of the specimens. Most of the losses took place during the first 30 days and reached 2.9% at 210 days. The tests started when further losses had become negligible, with unlikely effects on significant AE data. The strands from specimens P1 and N2 were removed for visual inspection at four and 12 weeks, respectively, after the first reading indicating depassivation. Minor signs of corrosion were visible on the strand removed from specimen N2. More evident visual evidence of early corrosion was noted for specimen P1 as shown in Figure 5. Figure 5(a) shows a non-uniform distribution of corrosion on the strand. In addition, Figure 5(b) shows that corrosion occurred at the impingement sites in the core wire, indicating the relevance of crevice corrosion in the damage process. An SEM micrograph showing a representative corrosion pit that developed on the inner surface of the strand is provided in Figure 5(c). The morphology of the pit suggests that the tendency is for it to develop as a larger and relatively shallow crater instead of a localized deeper pit. Deep pits are stress raisers, and their formation can result in a stress corrosion cracking mechanism (SCC)¹⁴, whereas shallow craters are less likely to induce SCC. Only corrosion sites similar to that on Figure 5(c) were observed, also in agreement with previous experimental evidence where no SCC was observed on corroded prestressing steel¹⁴.

AE data were analyzed by means of intensity analysis (IA). This method has been used for damage quantification in concrete structures during load tests¹⁵. The method is based on the computation of the Historic Index, $H(t)$, and the Severity Index, S_r . The Historic Index is defined as

$$H(t) = \frac{\sum_{i=N-K+1}^N \frac{S_i}{K}}{\sum_{i=1}^N \frac{S_i}{N}} \quad (1)$$

that is, the ratio of the average signal strength of the last K hits to the average signal strength of all recorded (N) hits up to time t , where S_i indicates the signal strength of the i -th event. The Severity Index is defined as

$$S_r = \sum_{i=1}^J \frac{S_i}{J} \quad (2)$$

that is, the average signal strength for the J hits exhibiting the largest signal strength up to time t . Different K and J values have been reported in the literature. Irrespective of the selected values, it is required that the number of hits be sufficiently large to prevent the computed indexes being dominated by isolated hits, and sufficiently small to

enable the indexes being affected by meaningful AE hits. In the present study, the IA parameters were calculated using $K = 20$ and $J = 35$. IA is best interpreted when plotting $H(t)$ and S_r as Cartesian coordinates and comparing the result with empirically defined assessment criteria. Mangual et al.¹⁶ developed a set of such criteria for corrosion assessment. In their study, eleven cracked concrete prisms with an embedded strand were subjected to accelerated corrosion through exposure to NaCl solution and application of a constant voltage. Upon testing, $H(t)$ and S_r were calculated and plotted as indicated above. It was observed that specimens showing similar section losses, half-cell potentials and crack patterns produced IA coordinates that grouped in four distinct zones of the plot. An assessment chart was then proposed by drawing boundaries in the $H(t) - S_r$ plot, hence dividing it in four areas, each indicative of a different condition: no corrosion, depassivation, cracking, and severe damage. Figure 6 shows the $H(t)$ - S_r coordinates for each pile specimen on the assessment chart. The coordinates were calculated taking t equal to the maximum exposure time of each specimen, i.e. $t = 189$ days for N2 and $t = 300$ days for the remaining ones. It is noted that relatively higher historic and severity indexes are associated with corrosion activity (specimens N2 and P1). In addition, the degree of damage is in good agreement with the qualitative descriptions of the IA boundaries. In fact, specimen N2 is plotted in the “No corrosion” area but close to the “Depassivation” area, consistent with the mild corrosion that was observed on the steel strand. Specimen P1 is plotted in the “Depassivation” area approaching the “Cracking” area, consistent with the visual evidence provided in Figure 5.

To analyze the relation between IA results and the time at which corrosion was detected by electrochemical measurements, $H(t)$ and S_r from corroding specimens are plotted in Figure 7 as functions of time. The dashed line indicates the onset of corrosion as indicated by the electrochemical measurements. Specimen P1 shows a slight increase in the rate of S_r at 150 days, possibly due to depassivation, and a significant increase in both parameters at 257 days, which can be attributed to micro-crack nucleation. Specimen N2 reached the maximum $H(t)$ and S_r at 123 days, 38 days before corrosion was indicated by electrochemical measurements. In both specimens, relevant changes in IA indicators were observed earlier than electrochemical changes. It is so because corrosion products begin to build up at the steel-concrete interface shortly after depassivation, thereby generating AE that is detected by the sensors. In contrast, the electrochemical measurements can be insensitive to localized corrosion¹⁷, especially at early stages of the process, when damage is limited to a small area. Table 2 presents a summary of AE results from the pile specimen tests. The largest amplitudes and signal strengths were observed on corroding specimens, and the number of recorded hits was higher in non-corroding specimens. This shows that the number of hits can be a misleading measure of corrosion damage, as noise and AE produced by sources other than corrosion can account for a large portion of the total recorded hits, especially during long monitoring periods. In contrast, intensity analysis takes into account recent and past magnitudes of signal strength and relies on arithmetic ratios that mitigate the effect of high numbers of hits, thereby minimizing spurious alarms.

Beam specimens

The corrosion tests were halted after 140 days as the LPR measurements indicated that the specimen with the larger crack width had a relatively high corrosion rate compared with the specimen with the smaller crack width, as shown in Table 3 and detailed elsewhere¹⁸. The strands were then removed for visual inspection and SEM analysis. Figure 8 shows SEM micrographs that illustrate the extent of the damage induced during the tests. Figure 8(a) shows the damage pattern prevalent on the beam with the smaller pre-existing cracks, which consists of spread, non-uniform section loss. Similar corrosion morphology was observed on the beam with the largest pre-existing crack. However, deep craters were also observed, as illustrated in Figure 8(b). Figure 9 shows IA results from the beams. IA parameters were calculated by taking K as a function of the number of hits, as proposed by Nair and Cai¹³, and J equal to 50. IA coordinates for both beams are located in the areas of greater damage, consistent with the observed damage and the estimated corrosion rates. This and the results from the tests on pile specimens demonstrates the feasibility of the IA chart approach for corrosion assessment and suggests that similar charts can be used for specimens of different sizes. Table 4 presents a summary of AE monitoring results from beam specimens. Maximum signal strength, maximum amplitude, and number of hits are larger in the specimen with the larger cracks. Notice that the maximum signal strengths attained are of the same order of magnitude as those of piles specimens. This illustrates the adequacy of signal strength based approaches –such as IA– for the development of universal assessment criteria.

CONCLUSIONS

The literature on acoustic emission monitoring of corrosion in PC reports on efforts that focused primarily on the detection of corrosion-induced strand fracture. A few laboratory studies have shown, however, that detection of corrosion at earlier stages is feasible. In this paper, the implementation of AE monitoring to detect early corrosion in

PC piles and beams was illustrated. Intensity analysis was enlisted to classify corrosion damage. The following conclusions are drawn:

1. Early corrosion in both uncracked and cracked PC members can be recognized using AE intensity analysis.
2. The values of severity and historic index and the observed damage were in good agreement with the damage classification criteria based on IA boundaries that were recently developed based on evidence from small scale reinforced concrete prisms subjected to accelerated corrosion¹⁴. This suggests that scaling effects do not significantly affect the IA analysis results for dimensions limited to those in this study, and for tests conducted under laboratory conditions.
3. Visual inspection of corroded strands suggests that the localized pitting and crevice corrosion are the main corrosion mechanisms in PC exposed to chlorides. In addition, the observed morphology of the pits suggests they tend to develop into shallow craters instead of localized deeper pits, thereby reducing the likelihood of strand failure due to stress corrosion cracking effects.

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Table 1—AE parameters

Specimen Type	AE Threshold [dB]	Peak definition time [μ s]	Hit definition time [μ s]	Hit lock-out time [μ s]
Piles	40	50	200	300
Beams	40	200	400	200

Table 2—Summary of AE monitoring results for pile specimens

Specimen	Test duration [days]	Number of recorded hits	Maximum amplitude [dB]	Maximum signal strength [pV-s]
P1	300	1353	84	2.4×10^6
P2	300	4020	71	4.0×10^5
N1	300	2865	80	4.2×10^5
N2	186	7400	87	1.1×10^6

Table 3—Electrochemical measurements on beam specimens

Crack width [in]	R_p^* [Ω]	i_{corr} [$\mu\text{A}/\text{cm}^2$]	CR [$\mu\text{m}/\text{yr}$]	Corrosion level
0.016	378	0.48	5.7	Moderate
0.032	30	5.96	70.4	High

* Polarization resistance averaged over duration of test (Metric (SI) conversion factors: 1 in = 25.4 mm)

Table 4—Summary of AE monitoring results for beam specimens

Specimen pre-crack size [in]	Test duration [days]	Number of recorded hits	Maximum amplitude [dB]	Maximum signal strength [pV-s]
0.016	140	20659	90	3.6×10^6
0.032	140	33328	94	6.2×10^6

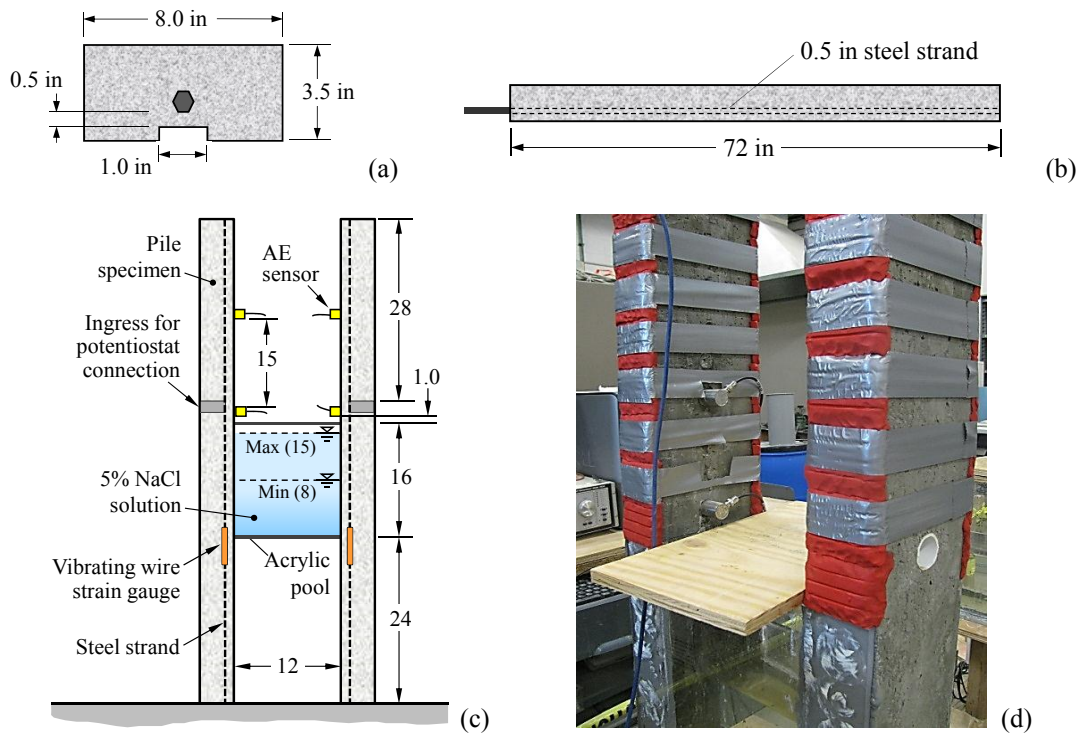


Figure 1—Pile specimens: (a) cross section; (b) side view; (c) schematic of test setup; and (d) photo of test setup. Dimensions in in (1.0 in = 25.4 mm)

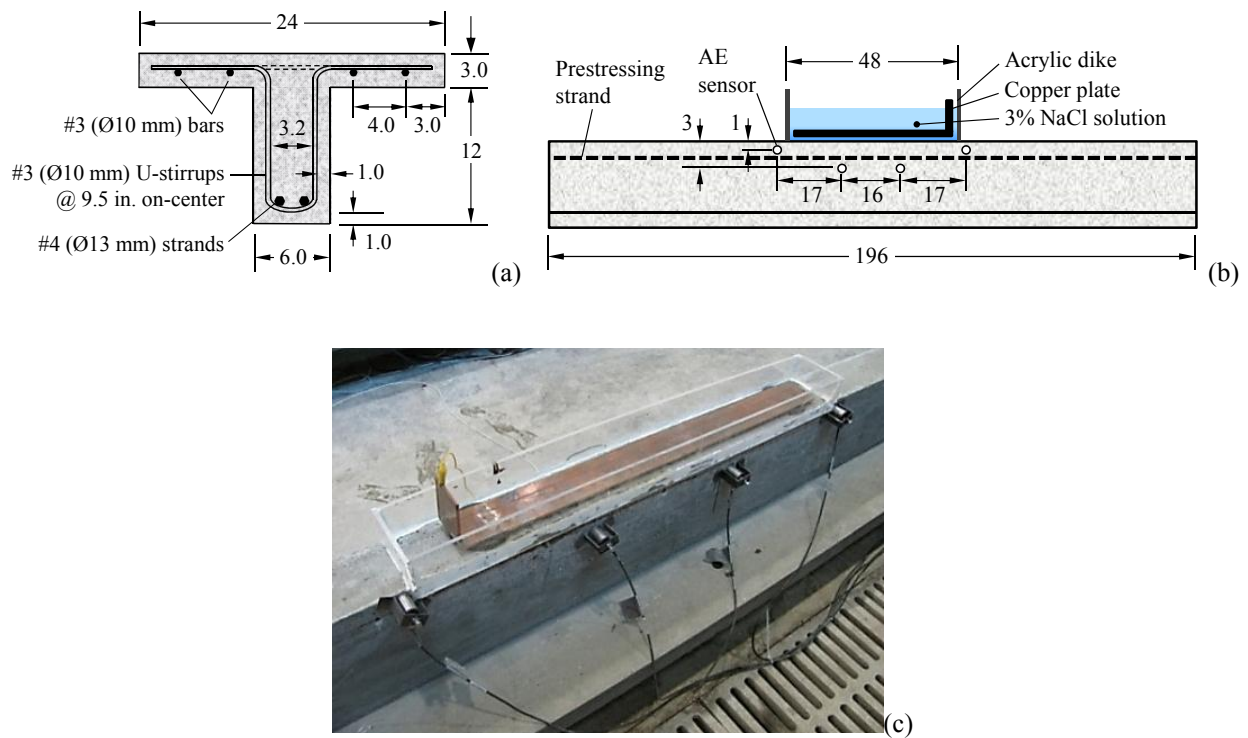


Figure 2—Beam specimens: (a) cross section; (b) schematic of test setup; and (c) close-up photo of monitored area. Dimensions in in (1.0 in = 25.4 mm)

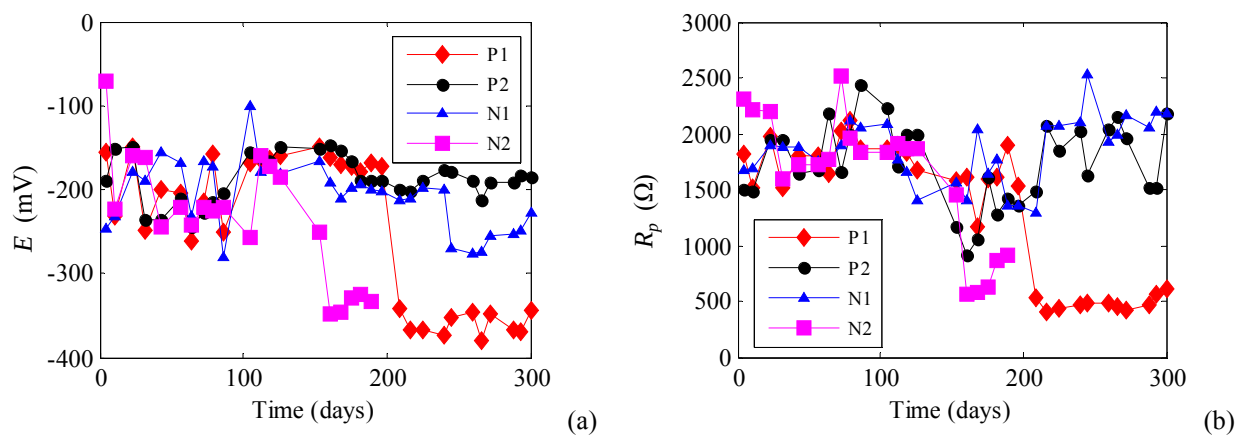


Figure 3—Electrochemical measurements on pile specimens: (a) half-cell potential; and (b) polarization resistance

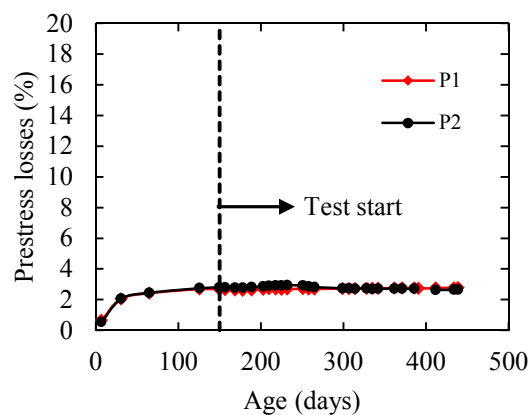


Figure 4—Prestress loss over time

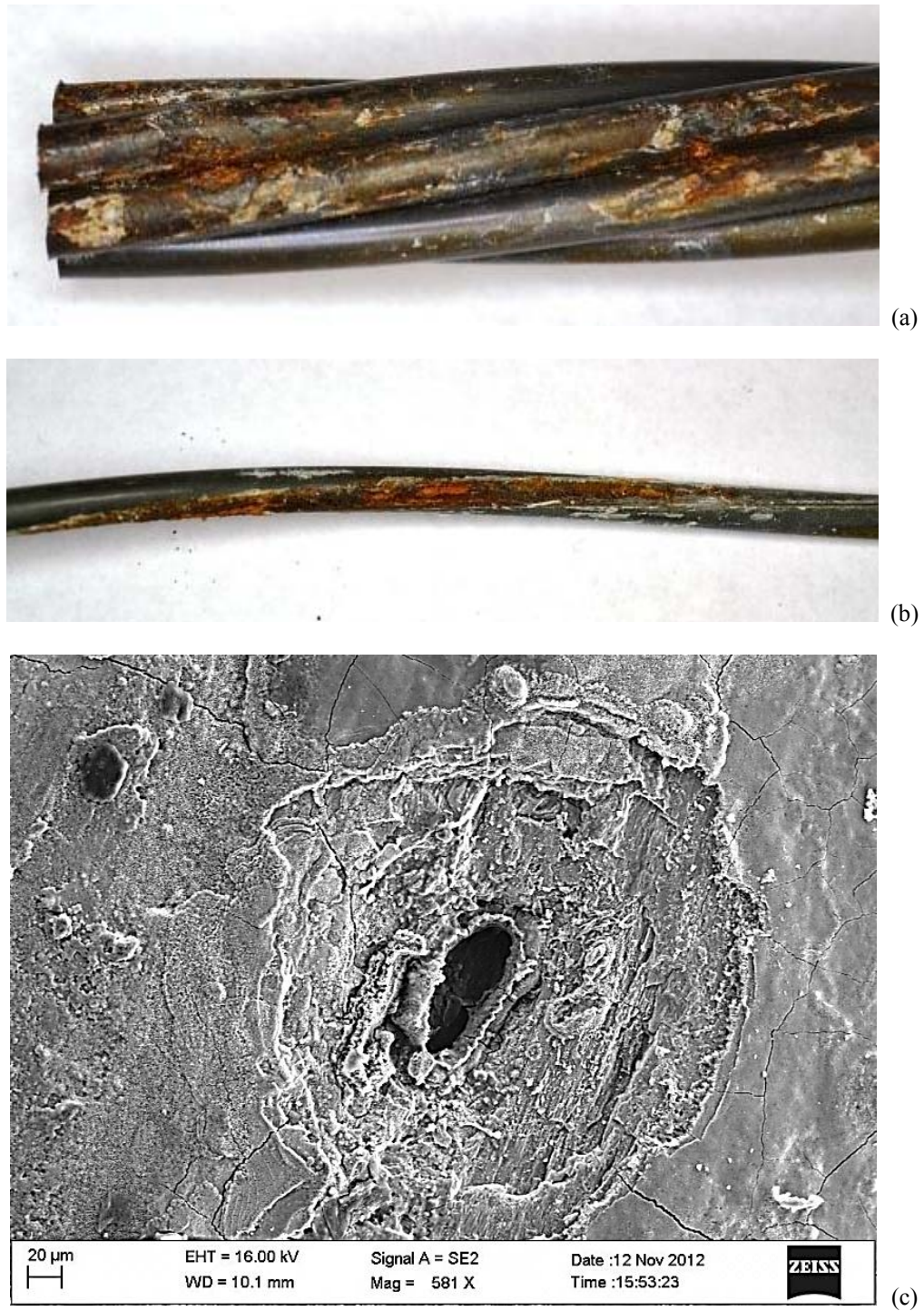


Figure 5—Visual evidence of corrosion in specimen P1: (a) photograph of corroded strand; (b) photograph of crevice corrosion in core wire; and (c) SEM micrograph showing corrosion pit on strand surface

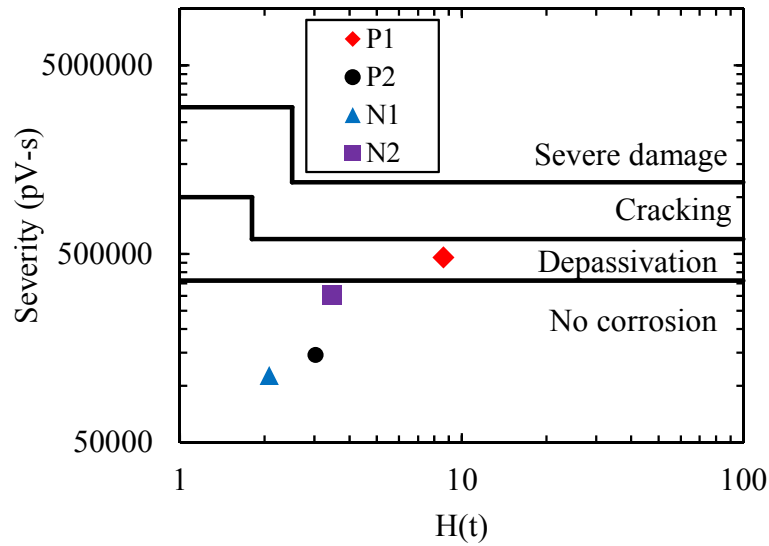


Figure 6—Intensity analysis chart for pile specimens after 300 days of exposure

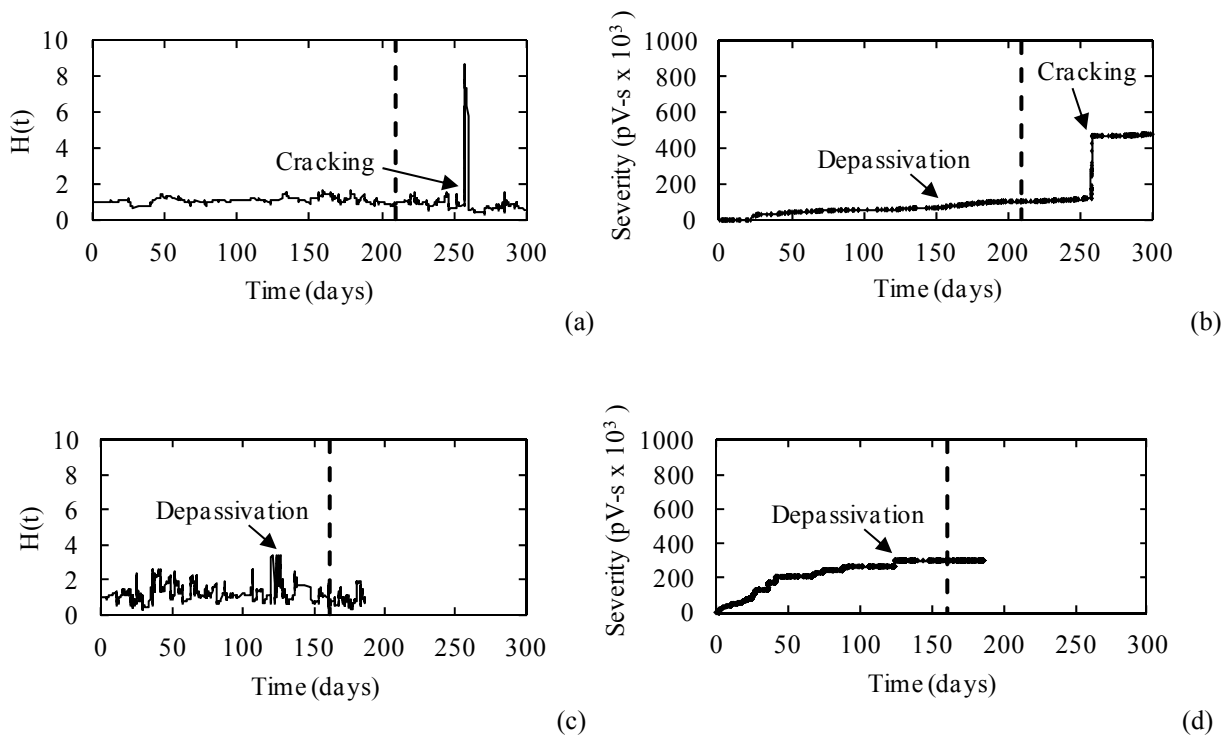


Figure 7—Variation of IA parameters over time: (a-b) P1, and (c-d) N2

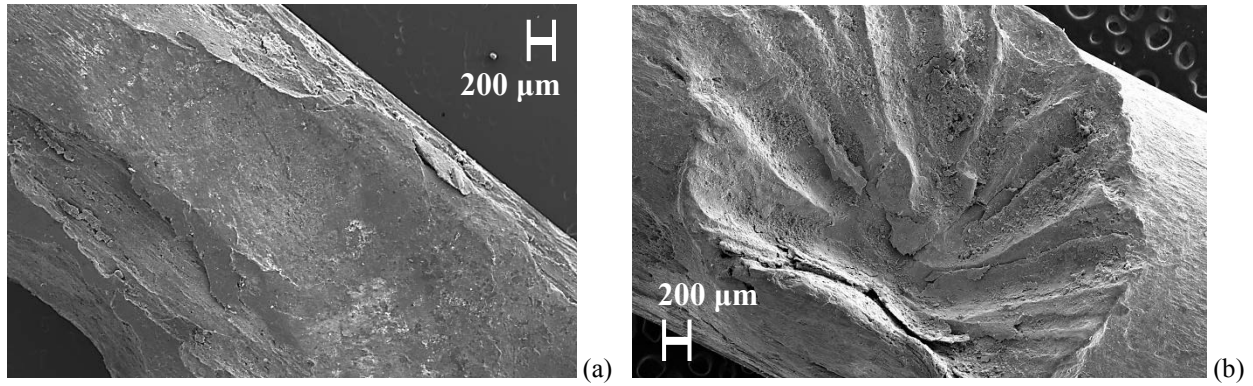


Figure 8—Visual evidence of corrosion in beam specimens: SEM micrographs showing corrosion pit on strand surface for case of (a) 0.016 in (0.4 mm) and (b) 0.032 in (0.8 mm) crack.

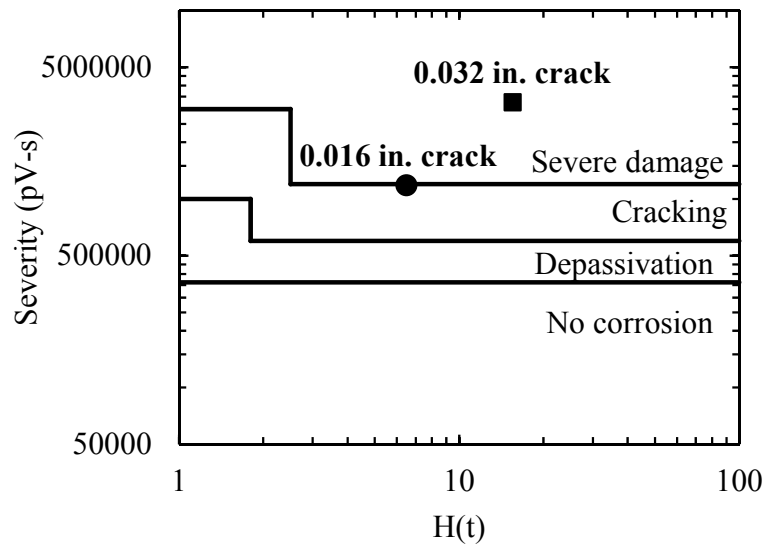


Figure 9—Intensity analysis chart for beam specimens after 140 days of exposure

Swedish Recommendations for Steel Fiber Concrete Overlays

Johan Silfwerbrand

Synopsis: Despite that steel fiber concrete (SFC) has been used in concrete structures during more than 50 years there is still a lack of practical recommendations. In Sweden, SFC has been used in concrete overlays on bridges during more than 25 years but the usage frequency is fairly low. The current Swedish guidelines for SFC overlays are based solely on empirically determined relationships between the amount of conventional reinforcement for crack control and amount of fibers. In a recent study at the Swedish Cement and Concrete Research Institute, the current recommendations have been critically reviewed. The experience shows that this empirical recommendation works fairly well for traditional concrete mixes and for the most frequently used fibers but there is a need of recommendations that promote the development of better concrete mixes and more efficient fibers. In order to reach this goal, guidelines which are much broader in their application field need to be developed. A novel approach taking the mechanical properties of the actual SFC into account has been proposed. This approach may also be used for concrete with synthetic fibers if their mechanical and long-term properties are proven to be sufficient.

Keywords: Bridge decks, crack control, fiber concrete, overlays, recommendations.

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INTRODUCTION

The bridge deck of a road bridge has several functions: (a) evenness for traffic comfort, (b) prescribed evenness and inclination for water drainage, (c) friction between tire and bridge surface, (d) protection of the structural concrete against water, moisture, and de-icing salt, (e) wear resistance and resistance against deformation, and (f) durability. Furthermore, (g) minimum noise when trafficked and (h) visibility are additional but valuable functions. In Sweden, there are two competing systems: (i) multi-layer system and (ii) cast-in-place wear layer concrete (Fig. 1). The multi-layer system may consist of the following layers: structural concrete, waterproofing system, protective concrete, bituminous binder course, and bituminous wear course. The waterproofing system is also a multi-layer system consisting of a bituminous sheet, mastic asphalt, and a liquid that could be composed by, e.g., epoxy, polyester, poly-urea, or polyurethane, (Edwards 2011). Solutions without the protective concrete layer are becoming more frequent. The wear layer concrete is cast-in-place as a bonded overlay to the structural concrete. Both systems have their advantages and disadvantages (Table 1). The main advantage with the multi-layer system is its evenness. The most important advantage with the wear layer concrete system is its resistance against wear and deformation and its durability. The most important disadvantages are damages in the sealing that are difficult to detect and difficulties to obtain evenness, respectively. The wear layer concrete could be plain, reinforced or composed by fiber concrete. This paper is focused on the third type and especially steel fiber concrete (SFC). The abbreviation SFC is used in this paper instead of the longer term steel fiber reinforced concrete (SFRC). The reason is that the role of the fiber in fiber concrete is different from the role of the steel bar in conventionally reinforced concrete.

AIM

The aim of the project was to develop a proposal to guidelines for designing and executing cast-in-place wear layers in fiber concrete, especially steel fiber concrete (SFC). Especially, the following questions ought to be answered:

- Which fiber content is needed?
- Are there other suitable fiber materials than steel?
- Which thickness is needed?
- Which requirements are valid for the concrete?
- Which requirements are valid for the execution?
- How to warrantee the quality of the wear layer concrete?

RESEARCH SIGNIFICANCE

SFC is a versatile alternative for the concrete overlay on bridge decks. Correctly designed and constructed it can replace a series of layers constituting the multi-layer system that is more frequently used to fulfill the various requirements on the bridge deck. The Swedish guidelines have dealt with the recommended fiber content in a very approximate way. In order to increase the application field for the SFC overlay and admit the use of more advanced fiber and other fiber materials than steel there is an urgent need of recommendations that are based on mechanical behavior and not solely on experience.

CURRENT SWEDISH RECOMMENDATION

General

The current Swedish recommendation on cast-in-place wear layers of SFC is given by the Swedish Road Administration (SRA 2004). (After a merge with the Swedish Rail Administration in 2010, the road authority is a part of the Swedish Transport Administration.) It deals with the concrete material, the fiber content, the thickness, the execution, the conformity control, and its limitations. Due to space limitation, this paper is focused on fiber content and conformity control.

Fiber content

The required fiber content V_f (in percentage) is given by the following equation:

$$V_f = 15400 \cdot \frac{\phi^2}{s \cdot (\phi + 70)} \cdot \frac{d_f}{l_f} \quad (1)$$

where,

- ϕ = bar diameter of the reinforcement that is replaced with steel fibers (mm)
- s = spacing of the substituted reinforcement bars (mm)
- d_f = fiber diameter (mm)
- l_f = fiber length (mm)

SRA recommends that the conventional reinforcement, if any, is placed in two layers in each direction (Fig. 2). Eq. (1) may be rewritten in the following way:

$$V_f = 15400 \cdot \frac{2 \cdot h \cdot \rho_s}{\pi \cdot (\phi + 70)} \cdot \frac{1}{\lambda_f} \quad (2)$$

where,

- h = overlay thickness
- ρ_s = reinforcement ratio
- λ_f = fiber slenderness = l_f/d_f

Eq. (1) is based on the following two assumptions:

1. A wear layer of conventionally reinforced concrete should be reinforced with a reinforcement ratio $\rho_s \geq 0.33\%$ in both directions.
2. The conventional reinforcement according to assumption No. 1 could be replaced by a fiber volume according to Eqs. (1) or (2).

Both assumptions can be criticized. The given lower limit of the reinforcement ratio seems theoretically to be too small to control cracking despite the absence of reported cracking. In order to obtain multiple cracking, the tension capacity of the steel reinforcement has to exceed the tensile capacity of the concrete cross section. For centrally placed reinforcement, this leads to the condition $\rho_s \geq f_{ct}/f_{st}$, where f_{ct} = concrete tensile strength and f_{st} = yield strength of steel. Frequent strength values are $f_{ct} = 3$ MPa (435 psi) and $f_{st} = 500$ MPa (72.5 ksi), indicating that $\rho_s \geq 0.6\%$. The SRA value of $\rho_s \geq 0.33\%$ is therefore too small for most cases. The second assumption regarding the transformation from conventional reinforcement to fiber reinforcement seems to be based on considerations and not on research. Finally, there are no known tests or theories behind the coefficient 15400. Analyzing Eq. (2), it may be noted that the fiber content has to be enhanced with increasing thickness h . Fibers with higher slenderness λ_f are more beneficial (the fiber content may be reduced).

CONFORMITY CONTROL

Conformity control

According to SRA (2004), the pull-off strength has to be checked if the wear layer SFC is cast against a mature concrete surface. The following condition has to be fulfilled:

$$m \geq f_v + 1.4 \cdot s \quad (3)$$

where, f_v = required (tensile) bond strength (= 1.0 MPa [145 psi]), s = standard deviation for all measuring values ($s \geq 0.36$ MPa [52 psi]) and m = average of all measuring values. The minimum value of s (= 0.36 MPa [52 psi]) implies that the average value m always has to exceed $m \geq 1.504$ MPa (218 psi). Furthermore, for a single measuring value x the requirement says that $x \geq 0.80 f_v$ (= 0.8 MPa [116 psi]).

NEW GUIDELINES

General

On the commission of SRA, the author has developed a proposal to new guidelines (Silfwerbrand 2009a). They are summarized in the following Sections.

Assumptions

The new guidelines are based on the following assumptions:

1. The previous Swedish guidelines have performed well for the concrete overlay thickness, fiber material, type, and content, concrete quality, and execution methods frequently used. However, the guidelines have not allowed any great deviation from more or less standardized values.
2. The bond between the structural concrete acting as substrate and the overlay has to be sufficient for crack distribution purposes.
3. The crack spacing is proportional to the overlay thickness and the coefficient of proportionality is approximately equal to 3.
4. The crack width can be estimated as the product of crack spacing and estimated free shrinkage also taking the beneficial effect of the fibers into account.

The first assumption implies that the outcome of the design using the new guidelines ought to be similar to the outcome from the previous guidelines for a case with frequently used parameter values. It may be underlined that the aim of the new guidelines was not to change the amount of traditional steel fibers for more or less standardized solutions but to allow the use of different overlay thicknesses, other concrete mixes, and other types of fibers. Good bond is necessary for crack control. Tests by, e.g., Carlswärd (2006) clearly show that the crack width is reduced with 90 % if a case without bond is compared with one with bond. By using water-jet technology, decent bond strength is easily obtained if cleaning, casting, and curing are conducted with good workmanship. The fourth assumption is based on experience and laboratory tests (Laurence et al. 2000). Bonded plain concrete overlays will crack with spacing approximately equal to three times the overlay thickness (Fig. 3). In the proposed design formula (see Equation [5b] in section “Residual strength factor” below), the spacing has been increased with 50 % (i.e., the factor 3 is enhanced to 4.5) for conservative reasons. In plain concrete, the stress in the crack vanishes. Crack spacing will be dependent on the length (transfer length) needed for developing a stress equal to the cracking strength. In fiber concrete, the stress is dependent on the stress after cracking. For strain-softening fiber concrete, the ratio (denoted R) between stress after cracking and cracking strength is somewhere between 0 and 100 percent. That means that a smaller transfer length for developing stresses of such magnitude that a second crack can develop. This length reduction has been assumed to be proportional to $(1 - R/100)$.

Thickness

The thickness of the SFC wear layer ought to be $h \geq 70$ mm (2.8 in.). The reason for this rather high value is to enable use a concrete with maximum aggregate size $d_{\max} = 18$ mm (0.7 in.) in order to achieve a wear resistant concrete with limited shrinkage. The proposed thickness also enables grinding to an extent of 25 mm (1.0 in.) that may be necessary after 15-25 years to remove rutting due to the use of studded tires.

Fiber content

The necessary fiber content is dependent on fiber type and concrete mix why a general value cannot be given. Required fiber content has to be determined by pre-testing the candidate concrete mix in order to harmonize with the required performance, especially required maximum crack width. The pre-testing should be carried out through beam tests, see section “Residual strength factor” below.

Concrete

The selected concrete mix has to fulfill adequate requirements on durability, wear resistance, stiffness, limited shrinkage, friction, vulnerability to frost slippery, evenness, noise, and brightness. Essential for the wear resistance are high stone content, maximum possible $d_{\max}$, high compressive strength, and selected aggregates with high wear resistance.

Residual strength factor

In practice, the residual strength factor R for fiber concrete is equal to the ratio between the concrete strength after cracking (defined for a given displacement in a flexural test) and the concrete flexural strength at cracking. The residual strength factor is usually expressed as a percentage. The residual strength factor R can be determined through flexural tests on beams according to the Swedish method (Fig. 4, SCA 1997), but the European method is also possible to use since there is a conversion method available (Silfwerbrand 2008). The SCA method was in turn a further development of the American standard ASTM C1018 (1992). The residual strength factor used in the new guidelines has to fulfill both general and specific requirements. The general requirement aims at warranting a minimum ductility, the specific requirement aims at warranting crack control. The general requirement is given by the following condition:

$$R \geq 50 \% \quad (4)$$

The specific requirement can be fulfilled in two ways:

$$R \geq 70 \% \quad (5a)$$

$$w = 4.5 \cdot (1 - R/100) \cdot h \cdot \varepsilon_{cs} < w_{req} \quad (5b)$$

where,

- w = crack width (mm)
- h = wear layer thickness (mm)
- ε_{cs} = free concrete shrinkage
- w_{req} = requirement on maximum crack width (mm), often $w_{req} = 0.05$ mm (0.002 in.).

Eq. (4) has always to be fulfilled. Furthermore, either Eq. (5a) or Eq. (5b) has to be fulfilled. That means that $R = 65$ % might be sufficient, cf., the following example. For $R = 65$ %, $h = 70$ mm (2.8 in.), $\varepsilon_{cs} = 0.45$ mm/m (450 μ strain), and $w_{req} = 0.050$ mm (0.002 in.), we obtain $w = 4.5 \cdot (1 - 0.65) \cdot 70 \cdot 0.45 \cdot 10^{-3} = 0.0496$ mm (0.00195 in.) < 0.050 mm (0.00197 in.) = $w_{req} \Rightarrow$ OK.

Alternative fibers

The market is dominated by fibers of conventional steel, but there exist also fibers of galvanized steel, stainless steel, glass, carbon, basalt, and of various polymer materials (usually called synthetic fibers). There is no general hindrance for their use. Before using alternative fibers, the following items have to be considered:

- Various fibers have markedly different properties (e.g., density, strength, stiffness, length, slenderness, shape, anchorage devices) and they also perform differently in the concrete. One fiber type cannot be replaced by another one without extensive testing and probable change in fiber content.
- All fibers are not durable in concrete. Conventional steel fibers will corrode if the moisture content is sufficiently high, but due to the high steel quality used, they are more durable than conventional reinforcing steel. Without a specific protection, zinc in the galvanization will react chemically immediately in the fresh concrete. Glass fibers are not durable in the alkali environment that characterizes concrete.
- The long term properties are different. Concrete with synthetic fibers has been considered to creep more than SFC. However, there are recent tests contradicting this assumption (Kanstad & Zirculis, 2012). Anyhow, creep must not necessarily be a problem in the wear layer, but certainly long-term behavior has to be considered.

Execution

The recommendation according to SRA deals with two different execution methods, (i) casting wet-on-wet and (ii) casting on a hard substrate. The difficulty to obtain good bond is larger in the second method. The most important conditions are absence of micro-cracking in the substrate, cleanliness, compaction, and curing. The reader is referred to Silfwerbrand (2009b). As stated above, selecting water-jetting for concrete removal will provide great possibilities to obtain good bond.

Conformity control

The author has criticized the SRA recommendation for conformity control because they are too demanding in common cases and since the complicated analysis of the pull-off test results is not suitable for statistical evaluation. The failure may be located in the substrate, in the interface, in the overlay, or in the adhesive that connects the pull-off test device to the cylindrical concrete test sample. Also mixed failures are common. Results obtained for different failure modes cannot be placed in the same statistical population. In the new guidelines, the following condition is proposed:

$$0.9 \cdot m_{\text{lowest } 1/3} \geq f_v \quad (6)$$

where,

f_v = required bond strength (MPa)

$m_{\text{lowest } 1/3}$ = average of the lowest third of the measuring values (MPa)

If the conformity tests are conducted through conventional pull-off tests, the required tensile bond strength ought to be set to $f_v = 0.8$ MPa (116 psi). If the conformity tests are conducted through torsion tests (Silfwerbrand 2003), the required shear bond strength ought to be set to $f_v = 1.5$ MPa (218 psi).

COMPARISON BETWEEN OLD AND NEW METHOD

Computations

According to SRA (2004), the fiber content V_f may be computed with Eq. (1). The new guidelines do not provide such a simple formula. Instead, there is a need to determine residual strength factors for the actual concrete mix. In order to make comparisons possible, some assumptions are needed. Eq. (5) leads to the following condition:

$$R \geq \max \left\{ \begin{array}{l} 50 \\ 100 \cdot \left(1 - \frac{w_{\text{req}}}{4.5 \cdot h \cdot \varepsilon_{cs}} \right) \end{array} \right. \quad (7)$$

Please, note that R does not have to exceed $R = 70$ %. Let us study the conditions for $w_{\text{req}} = 0.05$ mm (0.002 in.) and $\varepsilon_{cs} = 0.30$ mm/m (300 μ strain). Since different fibers are differently efficient and since they work differently in different concrete mixes, it is not possible to provide a true relationship between fiber volume and residual strength factor. However, experimental results (Fig. 5, Silfwerbrand 2009a), show that for common fibers and concrete mixes this relationship is located in the limits given by the following equations:

$$R(\%) \approx 80 \cdot V_f(\%) \quad (8a)$$

$$R(\%) \approx 80 \cdot V_f(\%) + 30 \quad (8b)$$

A SFC that fulfills Eq. (8a) is a low quality SFC whereas one that fulfills Eq. (8b) is a high quality SFC.

Comments

The graph in Fig. 6 shows comparisons between required fiber content for the two sets of recommendations. Using the old recommendations, concrete with three fiber types with three different slenderness ratios have been studied. The graph clearly shows that increased fiber slenderness was assumed to have a very beneficial effect of the performance. These three lines have been compared with the new recommendations. Here, two lines describe two significantly different SFC mixes, one for a low quality SFC and one for a high quality SFC. The comparison shows that the difference between the old and new recommendations is limited. The new recommendation will not dramatically change the fiber content. This was also one of the requirements for the new guidelines. However, a high quality SFC is no longer solely dependent on fiber slenderness ratio. It may be observed that all curves have a positive inclination, i.e., the fiber content has to be increased with increasing wear layer thickness. According to the old recommendations, the relationship is linear. Using the new recommendations, the inclination diminishes (Fig. 6). This means that maximum fiber content V_f is obtained for a certain thickness. Up to this value, the fiber content is directly proportional to the residual strength factor R according to Eq. (8) that is based on test results shown in Fig.

5. The crack control requirement of R according to Eq. (7) increases with increasing h but is limited by $R = 70\%$. For the selected values of w_{req} ($= 0.05\text{ mm}$ [0.002 in.]) and ϵ_{cs} (0.30 mm/m [$300\text{ }\mu\text{strain}$]), this value is obtained for $h = 125\text{ mm}$ (4.9 in.) ($V_f = 0.5\%$ and $V_f = 0.875\%$ for the high quality SFC and low quality SFC, respectively). A never ending fiber content increase due to increasing wear layer thickness, as stipulated by the old recommendations, is hardly reasonable. In a practical case, an increasing wear layer thickness will lead to a higher relative wear layer thickness (the ratio between the wear layer thickness and the total thickness including the structural concrete). Such an increase leads to a diminished restraint which in turn reduces the restrained stresses in the wear layer (Silfwerbrand 2009a).

CONCLUDING REMARKS

The SFC wear layer constitutes an interesting alternative to the traditional multi-layer system. This alternative provides a durable solution with high resistance against wear and deformation and is not relying on a sealing which condition is difficult to assess. The comparison between the old and new recommendations shows that the difference in required fiber content is limited. This means that there will neither be any substantial economical savings nor any increased crack risk if the old recommendations are replaced with the new ones. However, there are three clear advantages with the new recommendations:

1. There is a method that is based on the properties of the SFC and not solely on fiber content and fiber slenderness. According to the new recommendation, the SFC properties have to be tested. The anchorage of the fiber in the surrounding concrete is a factor that has a substantial impact on the ductility of the SFC. So far, it has been neglected.
2. Only a method that is based on the SFC properties can promote the development of even better SFC and in turn improved wear layers.
3. The proposed method – that is based on a material independent residual strength factor – may be applied on concrete not only with steel fibers but also with, e.g., synthetic or basalt fibers.

Furthermore, the new recommendations contain a proposal for a new method for conformity control. This proposal will be simpler and fairer to use than present method.

ACKNOWLEDGEMENT

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Table 1—Advantages and disadvantages for the two different alternatives for concrete bridge decks

Alt 1: Wear layer concrete system		Alt 2: Multi-layer system	
Advantages	Disadvantages	Advantages	Disadvantages
High wear resistance	Difficulties to obtain good evenness without expensive grinding	Good evenness	The water-proofing layer has a limited lifespan (< 25 years)
High resistance against deformations under high temperature conditions	Many contractors lack the experience	Many contractors with long experience	Damages in the water-proofing system are difficult to detect
Excellent durability	If SFC is used, risk of protruding fibers		Fairly high risk of air bubble development in the water-proofing system
			Vulnerable for execution in cold weather
			Several interfaces with a potential risk of water reservoir
			If epoxy is selected, expensive and need of precaution due to health problems

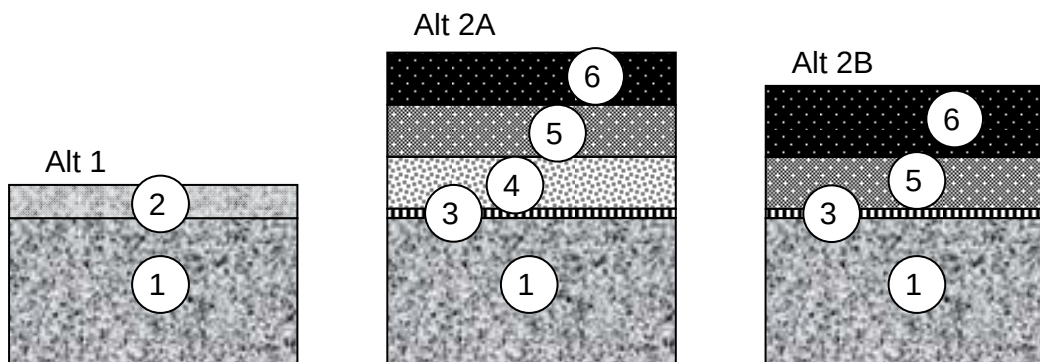


Fig. 1— The alternatives for the bridge deck. 1 = structural concrete, 2 = wear layer concrete, 3 = waterproofing system, 4 = protective concrete, 5 = bituminous binder course, 6 = bituminous wear course

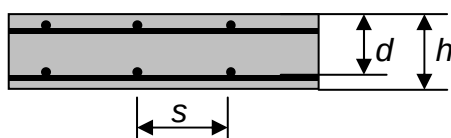


Fig. 2—Conventionally reinforced concrete overlay (with reinforcement in two layers) showing the definitions of thickness h , effective depth d , and spacing between rebars s .

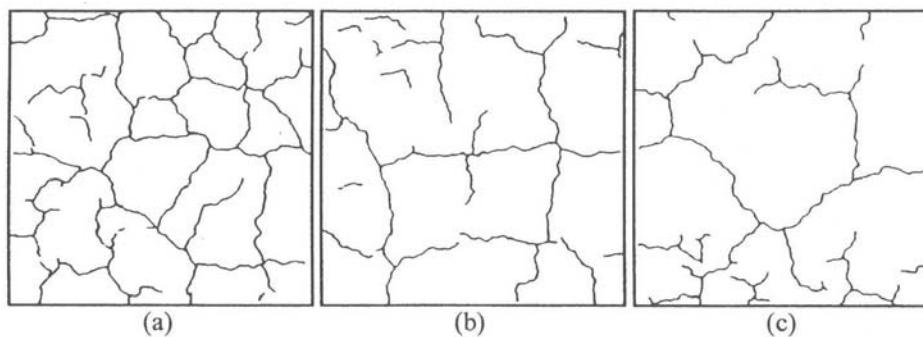


Fig. 3—Tests on mortar overlays showing increasing crack spacing for increasing overlay thickness. (a) Thickness = 25 mm (1 in.), (b) 50 mm (2 in.), (c) 75 mm (3 in.). (Laurence et al. 2000).

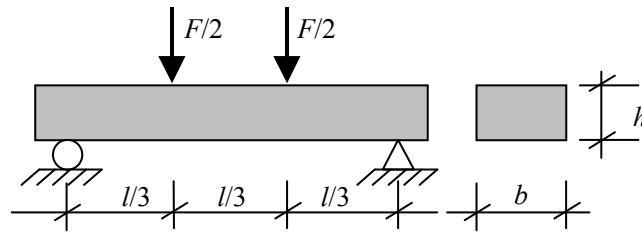


Fig. 4—Swedish flexural strength tests. $l/h = 6$. Recommended values: $b = 125$ mm (4.9 in.), $h = 75$ mm (3.0 in.), $l = 450$ mm (17.7 in.). (SCA 1997, Silfwerbrand 2008).

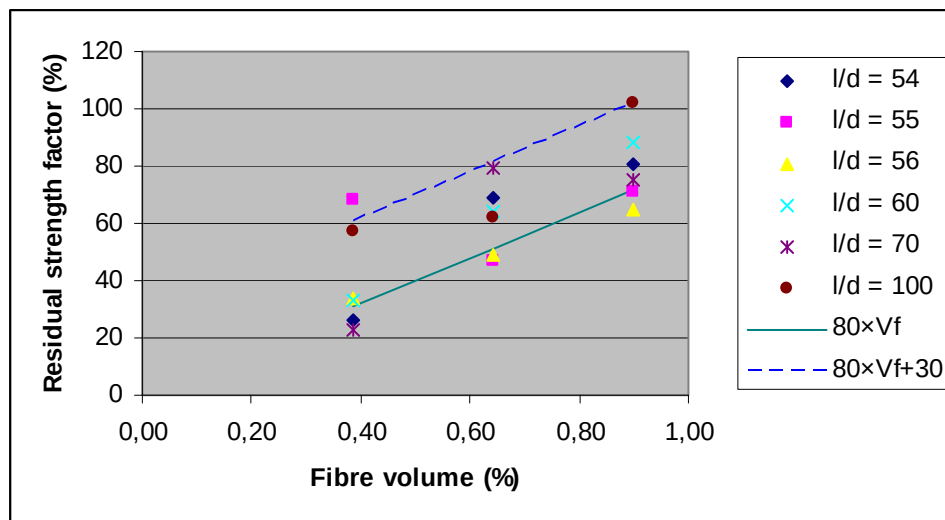


Fig. 5—Experimental data showing the relationship between fibre volume V_f and residual strength factor R for concrete (with compressive strength ≈ 30 MPa [4350 psi]) with six different fibre types with different slenderness ratios $\square_f = l_f/d_f$. The lines are given by Eqs. (8a) and (8b).

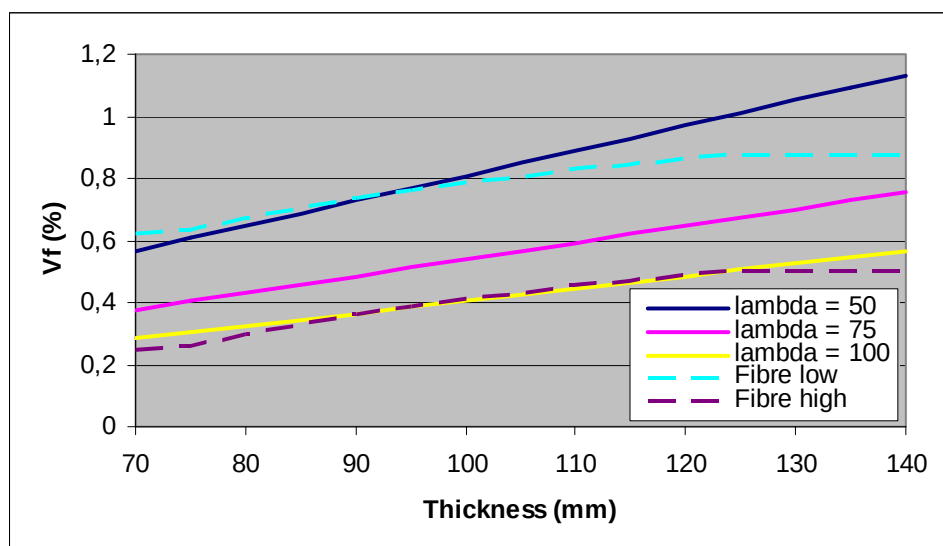


Fig. 6—Comparison between old and new recommendation showing the necessary fibre content as a function of wear layer thickness. The solid curves are computed using the old recommendation with three different slenderness ratios λ_f . The dotted lines are computed for the new recommendation for a low and a high quality SFC, respectively. The recommended overlay thickness should not be less than 70 mm (2.8 in.).

Evaluation of the Impermeability of Bridge Deck Overlays using Embedded Wireless Moisture Sensors

Brian M. Pailes, Michael C. Brown, Andrew J. Foden, and Nenad Gucunski

Synopsis: Overlays are installed on concrete bridge decks to improve ride quality, and in the case of impermeable overlays, also protect the deck from exposure to moisture and chlorides. Moisture and chlorides can penetrate over time into reinforced concrete, allowing for the initiation and progression of corrosion, which shorten the service life of a structure. To evaluate whether impermeable overlays are truly keeping moisture from penetrating into the concrete deck, researchers have implemented wireless moisture sensors in several bridge decks to monitor the moisture content of the deck below the overlay. In this study, the four overlays that are being monitored are a hot-mix asphalt wearing surface with a thermoplastic additive, an epoxy polymer concrete overlay, a fabric-reinforced liquid membrane with asphalt wearing surface, and a thin-set urethane membrane with an asphalt wearing surface. The moisture sensors have been installed at various locations in each deck including near the bridge joints, overlay construction joints, drainage paths, and under wheel paths. Results indicate that the hot-mix asphalt wearing surface with thermoplastic additive overlay only has moisture penetrating in regions that are near the joints. Measurements also indicate that the polymer concrete overlay has been effective at preventing the penetration of moisture. The latter two overlays, a fabric-reinforced asphalt membrane and a thin-set urethane, were recently installed and some preliminary conclusions may be offered about their effectiveness based upon early results.

Keywords: overlay, moisture, bridge deck, sensor

Evaluation of the Impermeability of Bridge Deck Overlays using Embedded Wireless Moisture Sensors

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INTRODUCTION

Overlays are placed on bridge decks for two primary reasons: to improve ride quality and to protect the deck from deterioration due to the intrusion of chlorides and moisture (1). Depending on the overlay's objective, various different types of overlays can be implemented. Impermeable overlays are installed to protect reinforced concrete decks from chlorides and moisture that can penetrate into the concrete and create an environment conducive to reinforcement corrosion. Corrosion of reinforced concrete diminishes the service life of the structure and increases the amount of maintenance dollars needed to keep the structure in proper service (2). Through the use of impermeable overlays, moisture and chlorides are prevented from penetrating into the deck, thereby extending the service life. Chlorides which can diffuse into the concrete matrix, lower the concrete's pH allowing for the depassivation of the reinforcing steel. Water along with oxygen are necessary ingredients in the corrosion reaction to create ferric oxide and hydroxide. Due to the importance of water and chlorides in the corrosion process, it can improve the service life of a structure if they are prevented from entering the concrete matrix by using an impermeable overlay. To evaluate the effectiveness of impermeable overlays, researchers have installed wireless sensors in the decks of four bridge structures prior to the installation of an impermeable overlay. Each bridge has a unique overlay; however, each overlay is marketed as impermeable. It is the goal of this research to evaluate the effectiveness of each overlay and monitor the overlays effectiveness over the long term.

METHODS AND MATERIALS

Moisture Detection System

The researchers utilized, HydroTracker, a moisture sensing system based on electrical resistivity principles to monitor the moisture content in the concrete bridge decks. The system implements embedded wireless sensors that are interrogated using a portable scanning device. The sensors are approximately 2 in. (5.08 cm) in diameter and ¼ in. (6.35 mm) thick (Figure 1). The sensors contain an electrical circuit of known parameters in conjunction with two exterior metal tabs that are electrically connected to the concrete through a bedding mortar. The exterior metal tabs in combination with the surrounding concrete act as a resistor in the sensor's internal electrical circuit, wherein the moisture in the surrounding concrete behaves as an electrolyte. The resistance of concrete is affected by several factors including moisture content, chloride content, and temperature (3). To take measurements, the portable scanning device (Figure 2) is placed on the surface of the deck above the embedded sensor. Once inductively coupled with the sensor, the measurement device sends a varying frequency electrical signal to the sensor, to which

the sensor responds based on the parameters of its circuit and the resistance of the concrete. The variation in the return signal is attributed to the resistance of the concrete. The scanner indicates the strength (amplitude) of the return signal along with the resonant frequency of the sensor. These values, along with known calibration values for the bedding mortar employed, are used to calculate the moisture content of the concrete. Calibration curves have been predetermined by the manufacturer for the sensors and the specified mortar bedding material used during installation. Along with the sensors that measure the localized moisture content, there are also control sensors that are embedded into the deck. The control sensors contain the same internal circuit as the moisture sensors; however, there are no external tabs to create the concrete resistor (Figure 3). Without the external metal tabs the control sensor is not affected by the changing moisture content of the surrounding concrete. The control sensor is used as a reference in the deck to provide a correction factor for temperature and general concrete properties. It is expected that the control sensor will provide limited correction for ion content. This will come from the attenuation effect the free ions have on the moisture meter signal which should be corrected for by the control sensor. Chloride will also affect the resistance of the mortar around the moisture sensor for which the control sensor will not be able to correct for. However, it will take several years for chloride to diffuse into the chloride-free patch material around the moisture sensors. When chloride level due increase in the patch material it will have an effect on the measurements. To determine the moisture content, the system uses the resistance of the surrounding concrete along with established concrete patch calibration values and a correction factor supplied by the control sensors. The patch concrete resistance is affected by the amount of moisture that is present in the pore structure of the concrete.

It is necessary to drill holes into the concrete to install the sensors in an existing deck. Installation of the sensors takes place prior to the installation of the overlay (Figure 4), such that no cuts are made into the new overlay that would create pathways for moisture to penetrate. Using a hammer drill, shallow holes are bored into the deck at each of the desired sensor locations. The sensors are patched using a mortar that has been preselected by the manufacturer of the moisture detection device. Moisture calibrations have been performed by the manufacturer on this patch material so that the sensor measurement can be properly converted into moisture content. The mortar is a typical sand and Portland cement mix that can be purchased at most hardware stores. The sensor locations are determined by reviewing the overlay construction documents to identify features such as drainage paths, construction joints, wheel paths, and structure joints, locations where risk of water infiltration may be high. Sensors are also installed at locations without identified weaknesses in the overlay as a control and to verify the impermeability of just the overlay material. The sensors are typically installed as close to the time of overlay installation as possible to limit or prevent exposure of the patch mortar to direct traffic loading.

New Jersey Bridge

The first bridge in which the sensors were installed was in Upper Freehold Township, New Jersey. The bridge carries east bound I-195 over Sharon Station Road. This bridge will be hereafter referred to as the New Jersey Bridge. The New Jersey Bridge is a single-span, simply supported steel girder bridge. The deck is reinforced cast-in-place concrete with stay-in-place steel forms and is 8 inches (20.32 cm) thick. The bridge was constructed in 1969 and received a rehabilitative latex-modified concrete (LMC) overlay in 1996. A more recent asphalt overlay, along with the moisture sensors, were installed in August 2010 over the existing LMC overlay. The recently installed overlay is a hot-mix asphalt wearing surface that contains a concentrated thermoplastic additive. In documentation provided by the overlay manufacturer, the overlay provides a “waterproofing/wearing course.”

To monitor the effectiveness of the overlay on the New Jersey Bridge, twenty moisture sensors were installed in the deck along with two control sensors (Figure 5). The sensors were placed strategically in the deck to monitor areas in the overlay that the researchers identified as potential weak points in the impermeability of the overlay; at the bridge joints, construction joints, places of significant wheel loading, and drainage paths. As a control in the experiment, researchers also placed sensors in areas of the deck that were not indicated as weak points. Sensors placed transversely at 1 ft. (0.31 m) from the south parapet (Sensors 1, 4, 11, 15, and 18) are intended to monitor moisture along the gutter line of the deck. Sensors at 11 ft. (3.35 m) transversely (Sensors 2, 5, 16, and 19) are at the lane line between the travel lane and shoulder, which is an area not expected to have any weaknesses in the overlay. Sensors at 14 ft. (4.27 m) transversely (Sensors 3, 6, 9, 12, and 14) are in the wheel path of the travel lane. The sensors at 18 ft. (5.49 m) transversely (Sensors 7, 10, 17, and 20) are placed between wheel paths, an area that is expected not to have weaknesses in the overlay. At 23 ft. (7.01 m) transversely (Sensors 8 and 13) the sensors are located at the overlay construction joint. At each end of the bridge sensors were placed along each of the expansion joints. At the west expansion joint a single row of sensors was placed at 1 ft. (0.305 m) from the centerline of the expansion joint (Sensors 1, 2, and 3). At the east expansion joint two rows of sensors were placed at 6 in. (15.24 cm) (Sensors 15, 16, and 17) and 18 in. (45.72 cm) (Sensors 18, 19, and 20) from the centerline of the expansion joint. Sensor

placement was denser at the expansion joints, since this area is expected to be the most susceptible to moisture penetration. The New Jersey Bridge was the first application of the moisture detection system by the researchers, so there was interest in seeing how a variation in sensor embedment depth would affect the readings. The thickness of the overlay was specified to be 2 in. (5.08 cm). Most of the sensors were placed with ½ in. (1.27 cm) of cover into the concrete deck to ensure that after placement of the overlay they would be within the detection range of the scanner, which is 4 in. (10.16 cm). Three other sensors were placed with 1½ in. (3.81 cm) of cover, which provided a sensor depth, after overlay installation, near the limit of the device to determine if the scanner would be able to measure sensors at this depth. The control sensors were placed at a depth of ½ in. (1.27 cm) to be at the same depth as the standard moisture sensors.

Seabreeze Bridge

Sensors were installed in the deck of the Seabreeze Bridge, located in Daytona Beach, Florida. The Seabreeze Bridge carries west bound State Route 430 (Seabreeze Blvd) over the Intracoastal Waterway. The bridge is a ten-span, post-tensioned concrete adjacent box-girder structure. The bridge was constructed in 1997 and a thin polymer overlay was installed in May 2011. The overlay is an epoxy multi-layer rapid-set, skid-resistant polymer concrete. In the documentation provided by the manufacture, the overlay provides “impermeable waterproofing.” To monitor the effectiveness of the overlay installed on the Seabreeze Bridge, nine moisture sensors along with one control were installed in span three of the bridge prior to installation of the overlay (Figure 6). Sensors placed at 1 ft. (0.31 m) transversely from the north parapet wall (Sensors 1, 4, and 7) are intended to monitor the moisture content along the gutter line of the bridge. The sensors at 5 ft. (1.52 m) and 16 ft. (4.88 m) (Sensors 4, 5, and 8) are in areas where there are no discontinuities or areas of anticipated excessive moisture exposure. The sensor at 13 ft. (3.96 m) transversely (Sensor 2) is located in the wheel path of the travel lane. The sensors at 20 ft. (9.14 m) transversely (Sensors 3, 6, 9) are located at the overlay construction joint. All of the sensors were placed approximately 1 in. (2.54 cm) below the surface of the deck prior to the installation of the overlay, which was approximately $\frac{3}{16}$ in. (4 mm) thick. On the Seabreeze Bridge, it is important to note that the sensors were not patched with the mortar material recommended by the sensor manufacturer. The recommended patch material did not meet the minimum strength requirements required by the overlay manufacturer and would void the overlay warranty. To meet the minimum requirements, a substitute material was used for which a calibration curve had not yet been developed; therefore direct moisture contents could not be determined. However, the relative moisture content at each sensor location in relation to the other Seabreeze Bridge sensors and changes over time can be determined. The researchers will, at a minimum, be able to know if the moisture content is rising or lowering and which locations have more moisture than others. Work is currently ongoing to develop calibration curves for the new patch material.

Natural Bridge

Sensors have been recently installed in a bridge located in Natural Bridge, Virginia. The bridge carries Virginia State Route 11 over Interstate 81 and will be hereinafter referred to as the Natural Bridge site. The structure at Natural Bridge is a four-span, continuous steel girder bridge. The deck is cast-in-place reinforced concrete. Natural Bridge was constructed in 1966 and an asphalt membrane and overlay were installed in August 2011. The overlay installed on the Natural Bridge is a 2 in. (5.8 cm) thick fabric-reinforced liquid asphalt membrane with a hot-mix asphalt wearing surface. The manufacturer of this overlay indicates that this overlay is “impermeable to moisture.” Figure 7 provides a layout of the moisture sensors that were installed in the Natural Bridge structure. Several of the sensors were installed at locations in the bridge that are potential weak points to the impermeability of the overlay, including near the expansion joints (sensors 29 and 30), gutter line (sensors 27, 25, and 21), and overlay construction joint (sensors 26 and 22). As a contrast to sensors at potential weak points, several sensors (23, 24 and 28) were installed at locations away from such features. Sensor 1 is the control sensor and was placed in the middle of the second span away from any abnormal features. The sensors installed in the Natural Bridge were installed with 1/2 in. (1.27 cm) concrete cover.

Weyers Cave Bridge

Moisture sensors were also installed in the deck of Virginia State Route 773 over Broad Run in Weyers Cave, Virginia. The Weyers Cave Bridge is a single simple-span precast/prestressed structural slab constructed through the combination of precast panels. The precast panels were fabricated off site, then placed as a replacement superstructure on existing abutments and post-tensioned together to form the Weyers Cave Bridge. The Weyers Cave Bridge along with the overlay was installed in May 2012. The Weyers Cave Bridge is the only bridge in this study in which the overlay was installed immediately after the bridge was built. The other three bridges were in service for a significant amount of time before the overlays were installed. The overlay on the Weyers Cave Bridge

is a thin-set urethane membrane with a hot-mix asphalt wearing surface approximately 3/16 in. (4 mm) thick. The manufacturer of this overlay indicates that this overlay is “impermeable to moisture.” Ten sensors along with a control were installed into the Weyers Cave Bridge precast panels while the panels were still at the fabrication yard. The sensors were placed approximately 1 in. (2.54 cm) below the surface of the precast panels. As shown in Figure 8, the sensors were located near the panel joints since it was suspected that if a failure in the overlay were to occur, it would happen near the panel joints due to the possibility of differential deflection of the panels under wheel loading. The bridge did not include curbs to guide drainage, and the membrane was turned down at the external faces of the precast slabs on all sides, so edges were not anticipated to affect waterproofing. The number of sensors for each bridge varied depending on the size of bridge, the amount and size of the identified weaknesses in the overlay. The New Jersey Bridge had the most sensors just because it was the first bridge to implement these sensors and a more thorough approach was taken.

RESULTS AND DISCUSSION

The moisture sensors have been monitored in the New Jersey Bridge for two years and the Seabreeze Bridge for just over a year. The recent additions to this work, the Natural Bridge structure and the Weyers Cave Bridge are in the preliminary stages, and some preliminary results will be provided.

New Jersey Bridge

Sensors embedded in the New Jersey Bridge deck have been measured five times since their installation, on 9/15/10, 12/22/10, 9/27/11, 12/28/11, and 6/12/12. The first reading on 9/15/10 was right after the installation of the overlay and verified that the sensors were working. Over the course of the study, sensors 18, 19, and 20 could not be scanned by the system. These sensors were installed 6 in. (15.24 cm) from the centerline of the east expansion joint. The expansion joint of the overlay incorporates a large steel plate, which inhibits the scanning device from identifying and taking readings from the sensors that are in close proximity to the plate. Researchers anticipated that this might occur, which is why two rows of sensors were placed at the east joint, one set at 6 in. (15.24 cm) and another at 18 in. (46 cm). The overlay expansion joint was not installed until after the second round of data collection on the New Jersey Bridge, allowing for some initial data to be collected from the sensors. Also, on some of the data surveys, the sensors that were installed deeper into the deck have been difficult to interrogate. If the overlay has high moisture content, it can significantly attenuate the scanner signal. This makes getting readings from deeper sensors very difficult and sometimes impossible. Table 1 provides an overview of the moisture sensor results from each test collection date. When the sensors were measured on 9/15/2001, only one day had passed after installation, thus the reported moisture content represents that of the newly patched mortar surrounding the sensor and not the actual condition of the deck. For later test dates, the mortar was expected to have reached equilibrium with the surrounding deck concrete. For this reason, trends in the data will be identified without regard to the first data set; however, the results from this test are still included in all provided tables and charts for completeness. Figure 9 plots the results of the sensors that have been placed along the gutter line. Looking at the trend in this data, since 12/22/10, there is a trend of increasing moisture content over time. The moisture contents in 12/22/10 along the gutter line were below average, however, in future readings the moisture contents are all above average and show an increasing trend. In the latest reading they are all about average, however, the average did increase significantly from previous averages as well. Figure 10 provides the results of the sensors that were installed along the lane line between the travel lane and shoulder. Sensor 19 only has two data points since this sensor is in close proximity to the steel plate in the expansion joint and now is undetectable. Sensors 5 and 16 remain at relatively constant moisture content and are above the average moisture content, except for the latest data collection. Sensor 5 and 16 indicate relatively constant moisture contents with no indication of wetting or drying. Since the lane line should not impact the integrity of the overlay, these sensors verify that the overlay is under no distress and is performing adequately. However, sensor 2 has a moisture content that is increasing with time. Sensor 2 is located along the lane line but it is also 1 ft. (30.48 cm) from the west expansion joint. This indicates that the expansion joint is of concern with regard to moisture penetration. Sensor 1 in Figure 9, located at the west expansion joint along the gutter line, also shows a large increase in moisture content compared to the other sensors in the gutter line (Figure 9). This indicates that the expansion joint is more susceptible to moisture penetration than the overlay itself. Figure 11 is a plot of the sensors that were installed 18 ft. (5.49 m) from the parapet between the wheel paths, in an area of the pavement that did not have any anticipated weakness to water penetration. It is important to note that there is no data for the 6/12/12 data collection. It is not fully understood at this time, but the three sensors could not be read by the scanner during the 6/12/12 collection. Sensor 20 was lost due to the expansion joint and sensor 10 is one of the sensors at considerable depth which could be to blame for the recent laps in data. Besides the missing data there is clear trend of decreasing moisture content at these locations. An interesting note is that sensor 17 is

located near the expansion joint. While the pavement above sensor 17 is not considered a weak point, its proximity near the expansion joint, 18 in. (45.72 cm), does present a possibility for moisture penetration. The results of sensors 16 and 17 indicate that a distance of 18 in. (45.72 cm) may be beyond the distance of moisture penetration from the expansion joint. Sensor 15 while at the same distance from the expansion joint is also located in the gutter line which explains the increase in moisture at that location. Figure 12 are the results of the sensors that are located in the wheel path of the travel lane. There is a lot of variability in the sensors that were installed in the wheel path of the overlay. Overall the sensors show a general decrease/constant moisture content. The one exception would be for the latest data point of sensor 14 which shows extremely high moisture content in the recent survey, indicating a possible error. Sensor 6 shows relatively constant moisture content and sensor 3 indicates a moisture content that is similar to the average moisture content. Sensor 3 is also near the west expansion joint and therefore is affected by more than just the wheel loading. These results indicate that the impermeability of the overlay in the wheel path does not degrade, despite significant loading. This data represents a short period of time in the life of the overlay and the overlay will experience significantly more loading in the coming years. The sensors will be continually monitored to establish if the impermeability of the overlay changes as the overlay experiences continued loading. Figure 13 is a plot of the sensors that were installed at the construction joint in the overlay. The sensors indicate two distinct trends. Sensor 8 shows steadily increasing moisture trend, while the trend for sensor 13 is higher but parallel to that of the average moisture content for all sensors in the deck. These results indicate that the construction joint of the overlay may not be adequately preventing the penetration of moisture.

Seabreeze Bridge

Sensor readings on the Seabreeze Bridge have been made three times since the installation of the sensors. These readings have occurred on 6/14/11, 1/4/12, and 7/10/12. Researchers observed that during measurement of the embedded sensors, the initial calibration of the scanner was more difficult than on other bridges. It is believed the reason for the difficulty in calibration of the system is because the Seabreeze Bridge carries utilities, including power lines. These power lines generate electromagnetic fields that can affect the scanner. With careful calibration and zeroing of the scanner, these effects can be accounted for and zeroed out. It is hard to tell at this stage to know how much effect the power line has on the results. The moisture meter has a zeroing control to make sure the meter is properly calibrated. Ideally the zeroing of the meter factors out the effects of the power line, but until more data is collected to know for sure, the data collected on the Seabreeze Bridge must be used cautiously. Table 2 provides an overview of the moisture data that was collected on the Seabreeze Bridge. It is important to remember that because of the alternate bedding mortar employed, the moisture contents presented are relative values and do not indicate the actual moisture percentage of the deck. Nonetheless, comparisons among sensors on this bridge and trends can still be analyzed. Calibration of the patch material used on the Seabreeze Bridge has not yet been completed. Figure 14 plots the average moisture content of each sensor group based on location. The primary observation made in Figure 14 is that all of the curves follow a very similar path. There is a sharp drop in moisture after the initial reading and then a slight increase. Also looking at the standard deviation of the sensor readings in Table 2 indicates that the measurements are closer together in the second two measurements than in the first. This could be an indication of the decks' moisture content reaching equilibrium. It is of interest to note that the moisture in the wheel path is low and the sensors located away from areas of possible moisture penetration indicate the highest moisture content. However, the separation between the control area and wheel path is minor, less than 1% in the last two data collections.

Natural Bridge

The overall results of the two surveys of the Natural Bridge sensors (Table 3) indicate that the moisture content of the bridge has slightly dropped. There are ten sensors in the Natural Bridge and eight of them indicated a drop in moisture. Although this drop is minor and there have been only two surveys of the bridge, it suggests that the overlay is generally working. Sensors 24 and 26 were the only sensors that indicated an increase in moisture content. Sensor 24 went from 6.7% to 6.9% and sensor 26 went from 5.9% to 7.0%. Sensor 24 is in a control area and measured a very small increase; sensor 26 is located at a construction joint in the overlay and showed a significant increase in moisture. It is still early to give a definitive conclusion regarding the Natural Bridge; however, the construction joint has showed an early sign of being a weak point in the overlay. Anecdotal visual evidence obtained during a recent visit to the bridge revealed that there are a small number of cracks visible in the bottom of the deck. Several such cracks show active water seepage, as evidenced by the accumulation of efflorescence on the underside of the deck in the period from September 28 to December 4, 2012. A more detailed survey will be required to determine how the distribution of actively leaking cracks related to features of the deck and overlay.

Weyers Cave Bridge

The sensors in the Weyers Cave Bridge have only been measured once since their installation. The sensors all gave relatively similar moisture contents in all of the panels (see Table 4). Continued monitoring of this bridge will be required to identify if moisture is penetrating the overlay.

CONCLUSIONS

The location of the sensors with regard to steel in the deck or overlay is important, as this can affect the ability to take readings from the sensors. Since some bridges carry utilities, like power lines, it is important to understand where they are and what utilities are being carried since they can affect the measuring device. Electromagnetic fields generated by these utilities can make calibration of the moisture scanner difficult.

New Jersey Bridge

Since the New Jersey Bridge has an “impermeable” overlay and stay-in-place forms, the cast-in-place soffit overhangs (without SIP forms) at the sides of the bridge are the only area that will provide an area of escape for moisture trapped in the concrete deck. This may account for the increase in moisture at the gutter line sensors. Moisture could also be penetrating between the parapet and the overlay joint along the gutter line. Continued monitoring of the sensors is needed to identify the cause of the moisture increase at the gutter line. Several of the sensors along the transverse expansion joint of the bridge show very significant increases in moisture content over the course of the study. It is a strong indication that the expansion joint is allowing moisture to penetrate to the concrete deck. As of the latest survey the penetration of moisture from the expansion joint is limited to less than 18 in. (45.72 cm) transversely from the joint. The overlay construction joint allows penetration of moisture. Sensors along the joint have indicated above average and increasing moisture contents. The integrity of the overlay does not appear to be affected by the constant loading from vehicles during its early life. The moisture content of the deck below the wheel path indicates constant or drying moisture content over time. The overlay is still young and continued monitoring of this area is required as the wheel loading continues over the life of the overlay. The locations of the overlay that are not preventing the penetration of moisture appear to be the result of seams in the overlay. These locations are at the overlay construction joint, expansion joints, and border with the parapet. These locations are weak points in the overlay and the overlay cannot sustain its impermeability at these locations. It indicates that the overlay itself is impermeable, however, the details at joints and seams are not; these need to be better developed to fully protect the bridge from moisture. At this point in the study the only definitive conclusion regarding the deeper sensors is that at times they can be difficult to identify and collect readings since they are at the limit of the sensor range. Further work will be needed to understand how this depth is affecting the measured moisture content.

Seabreeze Bridge

Due to the limited data on the Seabreeze Bridge, it is difficult to provide complete conclusions at this time. However, based on the similar changes in all of the sensors and only minor shifts in moisture content during the last data collection, the overlay appears to be adequately performing its job of keeping moisture from penetrating into the deck.

Natural Bridge and Weyers Cave Bridge

It is still too early to make any definitive conclusions regarding the effectiveness of the overlays installed on the Natural and Weyers Cave Bridges. Visual evidence of efflorescence suggests the membrane at Natural Bridge may not be fully effective, but needs to be further evaluated. Nonetheless, the sensors are working properly and continued monitoring of the bridges should reveal information regarding the impermeability of the overlays.

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Table 1—New Jersey Bridge moisture sensor results overview

Moisture Content (%)	Date Measured				
	9/15/2010	12/22/2010	9/27/2011	12/28/2011	6/12/2012
Average	8.38	7.80	7.86	6.83	9.23
Standard Deviation	1.99	1.87	2.72	2.64	2.05
Minimum	3.78	5.03	1.65	2.27	6.75
Maximum	11.47	12.05	10.9	9.79	14.98

Table 2—Seabreeze Bridge results overview

Moisture Content (%)	Date Measured		
	6/14/2011	1/4/2012	7/10/2012
Average	2.83	1.39	1.78
Standard Deviation	0.60	0.22	0.31
Minimum	1.86	1.06	1.23
Maximum	3.65	1.71	2.19

Table 3—Natural Bridge data

Moisture Content (%)	Date Measured	
	8/6/2012	10/10/2012
Average	6.89	6.56
Standard Deviation	1.03	1.21
Minimum	5.47	4.45
Maximum	8.74	8.61

Table 4—Weyers Cave Bridge Data

Moisture Content (%)	Reading
	9/15/2012
Average	5.58
Standard Deviation	0.88
Minimum	4.40
Maximum	7.68

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Figure 1—Moisture sensor



Figure 2—Scanning device A) Top view B) Profile view