Field Service Life Performance of Deep Polymer Impregnation as a Method for Protecting Bridge Decks from Corrosion

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Two sound concrete bridge decks were impregnated with methyl methacrylate to a depth of 76 to 102 mm (3 to 4 in.) and polymerized in situ. Both bridges were in sound condition with no surface spalling at the time of impregnation. After 15 and 5 years, respectively, the small- and full-scale field trial sections were evaluated for their corrosion abatement performance. The evaluation program included delamination survey, chloride content, corrosion potentials, petrographic analysis of core section, and corrosion rate measurements. The small-scale field trial has undergone dramatic differences in performance between the control and impregnated sections, with spalling and very high corrosion rates occurring in the control section. One year after evaluation (bridge impregnation age is now 15 years), the bridge deck containing the small-scale section was replaced. The impregnated section and a nonimpregnated section were salvaged for continuous monitoring. The full-scale field trial was reevaluated after 8 years of service. The control section for the full-scale field trial shows higher chloride contents and corrosion rates than the impregnated sections.

In 1972 NCHRP sponsored a research project with the objective of developing the deep polymer impregnation process for bridge decks. The deep impregnation process consists of drying the concrete, impregnating the concrete with a monomer to a depth of 102 mm (4 in.) in order to encapsulate the upper rebar mat, and polymerizing the monomer in situ. The deep impregnation process should abate or arrest the corrosion of the rebar by replacing the corrosion cell electrolyte (concrete pore water) with a dielectric polymer, immobilizing the existing chloride and reducing the ingress of further chlorides, water, and oxygen. The research project culminated in the impregnation of a test section 1.1 × 3.5 m (3.5 × 11.5 ft) of bridge deck in Bethlehem, Pennsylvania. In 1985 the Pennsylvania Department of Transportation sponsored a research project to demonstrate the technical and economical feasibility of full-scale impregnation of bridge decks. Approximately one-half—18.0 × 13.4 m (60 × 44 ft)—of a center span 39.9 m (131 ft) long of a bridge deck in Boalsburg, Pennsylvania, was impregnated to a design depth of 102 mm (4 in.) using the grooving technique in May and June 1985. In both cases, the in situ polymerization was accomplished with hot water ponding.

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Field corrosion performance investigations were performed on both bridges in 1989. In March 1990 the Bethlehem structure was replaced. Before replacement, the impregnated section and an adjoining untreated section were removed for further study.

The corrosion activity of the Bethlehem deck sections has been periodically monitored since their removal. A second posttreatment corrosion assessment was carried out on the Boalsburg bridge in May 1992. This paper presents the results of the corrosion performance surveys, including visual inspection, delamination survey, cover depth survey, chloride contents as a function of depth, corrosion potentials, petrographic analysis of drilled concrete cores, and corrosion rate measurements.

BOALSBURG BRIDGE DECK

Background

The Boalsburg three-span multigirder bridge carries the State College Bypass over PA Route 45. The end spans are 12.8 and 11.6 m (42 and 38 ft) long, and the center span is 39.9 m (131 ft). Curb to curb, the deck is 13.4 m (44 ft) wide, with two 3.7-m (12-ft) traffic lanes and 3.5-m (10-ft) aprons. The concrete deck was placed in April 1972.

Visual Inspection and Delaminations Survey

In March 1983 a visual inspection of the deck showed the deck to be in excellent condition. The only deterioration observed was a series of shallow spalls about 13 mm (0.5 in.) deep immediately adjacent to the expansion dam cover plate at the east end of the center span. The deterioration appeared to be the result of poor construction practices. A cover depth survey showed the mean cover depth to be 73 mm (2.86 in.), with a range of 58 to 84 mm (2.3 to 3.3 in.) and a standard deviation of 6 mm (0.22 in.). The mean value of the Cu-$CuSO_4$ (CSE) half-cell measurements performed in March 1983 was -176 mV with a standard deviation of -28 mV. Therefore, the probability was less than 10 percent that corrosion cells exist in about 80 percent of the deck, and the remaining 20 percent of the area showed potentials in the questionable zone. Chloride sampling and analyses demonstrated that less than 0.005 percent of the reinforcing steel had a chance of being critically contaminated with chlorides in March 1983. No delaminations associated with the corrosion of the reinforcing steel were discovered in March 1983. It must be noted that all of these measurements were taken more than 2 years before impregnation (impregnated June 1985).

A section approximately half the size of the center span, 18.3 m (60 ft) long and 14.6 m (44 ft) wide, was impregnated using the grooving technique with a monomer (methyl methacrylate) and polymerized in situ in June 1985 (*1*). The grooves were backfilled with latex-modified mortar. The typical depth of impregnation was about 89 mm (3.5 in.).

Drying shrinkage cracking was observed in the impregnated and the control section cores. The observed cracks were fine and generally of shallow depth [less than 12 mm (0.5 in.)]. The frequency of cracking was not significantly different for the control and impregnated sections. However, the cracks in the impregnated section were generally deeper.

In May 1992 a visual inspection and delamination survey showed that the deck was in excellent condition approximately 8 years after impregnation and 9 years after the preimpregnation condition survey. The only visual evidence of concrete deterioration was a few delaminations along the east expansion dam in the control section. These delaminations were in the same area as the previously noted spalls, which had been repaired at an earlier date. They are probably the result of poor construction practices, as noted earlier. No additional delamination planes were detected in the impregnated or control sections.

Cover Depth Survey

In May 1992 a cover depth survey was conducted on the center span. The mean cover depth was 66 mm (2.60 in.) with a range of 54 to 79 mm (2.12 to 3.12 in.) and a standard deviation of 5 mm (0.19 in.) for 80 observations. These results validate 1983 cover depth measurements within the accuracy of the instrument [6.4 mm (0.25 in.)].

Chloride Contamination Levels

In March 1983, powdered samples for chloride analyses were taken at mean depths of 6, 19, 38, and 64 mm (0.25, 0.75, 1.50, and 2.5 in.) in the aprons, right wheelpath, and center of wheelpath in both the to-be-impregnated and control areas on the eastbound side of the bridge. In March 1989 five powdered samples were taken in the right wheelpath in both the impregnated and control sections. The mean depths for these samples were 13, 25, 38, 51, 70, and 92 mm (0.5, 1.0, 1.5, 2.0, 2.75, and 3.63 in.). Thirty powdered samples were taken in May 1992. The samples were taken in the aprons, right wheelpath, and center of wheelpath in both the impregnated and control areas of the east and westbound sides of the bridge. The mean sample depths were 13, 25, 38, 51, 64, and 76 mm (0.5, 1.0, 1.5, 2.0, 2.5, and 3.0 in.).

The difference between the right wheelpath chloride contents for the sample years 1983–1989 and 1983–1992 for the impregnated and control sections would be a measure of the effectiveness of the impregnation process to exclude chloride ions (Table 1). The difference between the chloride contents for the eastbound right wheelpath is higher for the control section for all depths except 64 mm (2.5 in.) for both 1983–1989 and 1983–1992 differences. The mean chloride content at a depth of 64 mm (2.5 in.) is less than 0.71 kg chloride/m^3 (1.2 lb/yd^3) of concrete in all cases. It must also be noted that the March 1983 chloride samples were taken 2¼ years before the section was impregnated in June 1985.

Tables 2 and 3 show a comparison between the impregnated and control sections for the shoulder and center of wheelpath, respectively. Without preimpregnation chloride contents for these areas, it is not possible to determine the effectiveness of the impregnation process, but the results indicate that the impregnation process reduces the rate of chloride diffusion.

Besides the effectiveness of the impregnation process to exclude chlorides, the percentage of reinforcing steel in critically contaminated concrete is also of interest. The May 1992 chloride contamination levels for the impregnated and control sections are approximately equal. Assuming 0.71 kg chloride/m^3 (1.2 lb/yd^3) of concrete as the corrosion threshold level,

TABLE 1 Difference in Average Chloride Content, Right Wheelpath in Eastbound Lane, Boalsburg Bridge

Depth mm.	Chloride Content, kg/m^3		Difference Con.-Imp.	Percent Excluded by Impregnation
	Control	Impregnated		
1989-83				
13	3.0	2.1	0.9	29
25	2.7	1.0	1.7	63
38	1.3	1.1	0.1	9
64	0.4	0.4	0.0	0
1992-83				
13	4.5	3.9	0.6	13
25	2.8	1.8	1.0	35
38	1.2	0.6	0.5	45
64	0.2	0.2	0.0	0

Note: 1 lb/yd^3 = 0.5933 kg/m^3
1 in = 25.4 mm

TABLE 2 Average Chloride Content, Shoulders of East- and Westbound Lanes of Boalsburg Bridge, 1992

Depth mm.	Chloride Content, kg/m^3		Difference Con.-Imp.	Percent Difference
	Control	Impregnated		
13	4.9	3.0	1.8	38
25	5.0	2.7	2.2	46
38	3.5	1.5	2.0	56
64	0.8	0.3	0.5	64
76	0.4	0.2	0.2	43

Note: 1 lb/yd^3 = 0.5933 kg/m^3
1 in = 25.4 mm

0.03 percent of the reinforcing steel in the wheelpaths has a chance of being in critically contaminated concrete. This is based on the May 1992 cover depth survey.

Corrosion Potentials

Corrosion half-cell potentials were measured for the eastbound lane with a CSE in March of 1983 and 1989 and May of 1992. Table 4 presents the mean, standard deviation, and number of observations for all three surveys.

As given in Table 4, the eastbound mean corrosion potentials for both the control and impregnated sections are approximately equal in 1983, 1989, and 1992. The mean CSE potential has increased slightly in both the impregnated and control sections over the 9 years. For the control section in 1992, approximately 84 percent of the area surveyed is in the range of uncertain corrosion activity (−200 to −350 mV). Similarly, 85 percent of the impregnated area is in the uncertain corrosion activity range. There were 11 observations with a greater than 90 percent probability of active corrosion (greater than −350 mV): 2 in the impregnated area and 9 in the control area.

In May 1992 the westbound potential measurements were taken. The mean potential readings were 206 and 208 mV based on 319 and 405 observations for the impregnated and control sections, respectively. Again the mean corrosion potentials for both the impregnated and control sections are approximately equal. Both mean potentials are slightly lower than on the eastbound side, possibly because the cross slope of the bridge causes runoff to flow from the westbound to the eastbound side of the deck. Approximately 57 and 56 percent of the impregnated and control sections, respectively, are in the uncertain range of corrosion activity. Six observations indicated a greater than 90 percent probability of corrosion activity: one in the impregnated section and five in the control sections.

Petrographic Analysis

Four cores 10 cm (4 in.) in diameter were drilled with a water-cooled diamond bit for petrographic analysis: two in the impregnated section and two in the control section. The depth of impregnation was determined to be 76 to 102 mm (3 to 4 in.), which agrees with previous findings (*1*).

The concrete volume composition values are in the range of typical construction grade concretes. The coarse aggregate is a crushed limestone. The fine aggregate is a highly siliceous natural gravel sand. Both the fine and coarse aggregate are good-quality aggregates. However, the quality of the cement paste is poor, showing considerable evidence of excessive mixing water. Besides drying shrinkage cracks, there is excessive near-surface porosity and bleeding channels and high porosity and large irregularly shaped voids adjacent to coarse aggregate particles. The drying shrinkage/thermal and nonspecific cracks, though numerous, are minor and are equally distributed between the impregnated and the control areas.

Two of the four cores contained rebar, one in the impregnated area and one in the control area. The core from the impregnated area showed no corrosion. The core in the nonimpregnated area showed heavy corrosion deposits from a supporting chair. The core is just next to a sliding plate expansion joint at the east end of the span. This is the area in which spalling has occurred for some time due to poor construction practices. The corrosion of the chair was most likely

TABLE 3 Average Chloride Content, Center Wheelpath in East- and Westbound Lanes of Boalsburg Bridge, 1992

Depth mm.	Chloride Content, kg/m^3		Difference Con.-Imp.	Percent Difference
	Control	Impregnated		
13	10.0	9.0	1.0	10
25	6.4	4.3	2.1	33
38	3.6	1.8	1.7	48
64	0.5	0.5	0.1	11
76	0.3	0.2	0.1	20

Note: 1 lb/yd^3 = 0.5933 kg/m^3
1 in = 25.4 mm

TABLE 4 Half-Cell Potential Readings for Eastbound Boalsburg Bridge Deck

Statistical Parameter	Potential -mV					
	Impregnated Section			Control Section		
	1992	1989	1983	1992	1989	1983
Mean	235	192	173	234	200	176
Standard Deviation	34	19	5	39	26	28
Number observations	331	24	9	388	28	72

preexisting and is to be considered a special case. The inordinately deep cover of the reinforcing steel appears to have prevented the corrosion of the reinforcing steel as of May 1992, despite the poor-quality concrete.

Corrosion Rate

In May 1992 and March 1990, corrosion rate measurements were taken in both the control and impregnated areas. The March 1990 measurements were made with a three-electrode linear polarization (3LP) device, which is based on the linear polarization resistance technique with changes in cathodic polarization currents measured at changes in potentials of 0, 4, 8, and 12 mV. In May 1992 the Geocisa Gecor device was used in addition to the 3LP. The Gecor device is also based on the linear polarization resistance technique with the addition of a guard ring electrode to confine the area of polarization. Table 5 presents the mean, standard deviation, and number of observations for the corrosion rate measurements taken on the eastbound side of the deck. Table 6 presents the results of the May 1992 corrosion rate measurements and the cover depth at the test locations.

In 1990 the control section was corroding at a rate of about 2.5 times the impregnated section as measured by the 3LP. In 1992 the control section was corroding at a rate of about 2.2 times the impregnated section as measured by the 3LP and 3.5 times as measured by the Gecor device. Also note that there is no correlation between the rate of corrosion and the corrosion potentials taken during the corrosion rate measurements.

Concrete Resistivity

Concrete resistivity measurements were taken with the Nippon Steel device in the March 1989 survey, and concrete resistance measurements were obtained from the Geocisa Gecor device in the May 1992 survey (Table 7). Unfortunately, the concrete resistance measurements taken with the Gecor device cannot be directly converted to concrete resistivity at this time.

It is generally believed that corrosion cannot occur if the concrete resistivity is greater than 12 K·ohms·cm (*2*). Though the mean resistivity value for the impregnated concrete as measured by the Nippon Steel device is 17 percent below this value, it is 48 percent greater than the value for the control section. The mean concrete resistance measurement from the Gecor device is 118 percent greater in the impregnated area. Both of these comparisons indicate that the impregnation process increases the concrete resistivity, which should decrease the likelihood of corrosion.

BETHLEHEM BRIDGE DECK

Background

The Bethlehem dual-lane bridge carries Pennsylvania Route 378, the spur route linking Bethlehem to US-22 to I-78, over Union Boulevard. In March 1975 a test section at the south end of the bridge, 1.1 × 3.5 m (3.5 × 11.5 ft), was deep polymer impregnated from the surface using the pressure method. The bridge was 8 years old at the time of the impregnation. The wheelpath areas were deeply rutted. The chloride content at the depth of the reinforcing steel in the impregnated area exceeded the corrosion threshold level but the deck was sound, with no spalled or patched areas. Details of the deep impregnation with methyl methacrylate and in situ polymerization of the field test installation are presented elsewhere (*3*,p.77).

A visual examination of the bridge in December 1983 revealed some obvious differences in the performance between the deck in general and the deep impregnated test area that

TABLE 5 Corrosion Rate Measurements for Eastbound Boalsburg Bridge

Statistical Parameter	i_{corr}, $\mu A/cm^2$					
	Impregnated Section			Control Section		
	1990 3LP	1992 3LP	1992 Gecor	1990 3LP	1992 3LP	1992 Gecor
Mean	1.54	2.20	0.121	3.94	4.89	0.424
Standard deviation	0.38	1.13	0.113	0.98	2.37	0.411
Number observations	10	8	8	10	7	7

Note: 1 $\mu A/cm^2$ = 0.929 mA/ft^2

TABLE 6 Corrosion Rate Measurements for Boalsburg Bridge Deck

Location	Cover Depth mm	Corrosion Potentials mV, CSE	Corrosion Rate $\mu A/cm^2$ 3LP	Gecor
Impregnated Section				
Eastbound				
1	69	134	1.28	0.064
2	63	178	4.48	0.389
3	66	199	3.10	0.141
4	64	157	0.93	0.054
5	60	186	2.21	0.099
6	61	222	2.11	0.056
7	71	128	1.50	0.054
8	65	184	1.99	0.103
Westbound				
1	66	117	1.24	0.091
2	67	194	1.77	0.024
3	66	160	1.39	0.029
4	61	128	0.32	0.014
5	69	242	NA	0.070
6	61	284	NA	0.034
Control Section				
Eastbound				
1	68	359	9.61	1.348
2	69	152	4.40	0.210
3	66	167	5.65	0.265
4	71	127	2.43	0.251
5	66	160	3.54	0.200
6	67	197	5.25	0.354
7	78	153	3.32	0.338
Westbound				
1	67	178	6.70	1.214
2	65	159	3.81	0.237
3	71	188	6.84	0.717
4	70	193	5.78	0.228
5	57	142	2.42	0.227
6	61	147	2.71	0.249

initiated further investigation. The investigation, performed in February 1984, consisted of a delamination survey, corrosion potential measurements, chloride content analysis, and a microscopic analysis of drilled concrete cores. The results of the February 1984 study have been reported elsewhere (*4*). A second investigation, performed in March 1989, consisted of a visual inspection and delamination survey, corrosion potentials, and microscopic analysis of drilled concrete cores.

In March 1990 the structure was replaced. Both the deep-impregnated test section and an adjoining section, 1.4 × 3.1 m (4 ft 7 in. × 10 ft 4 in.), were removed and subsequently placed in the outdoor exposure area of the Structures and Materials Research Laboratory of the Virginia Polytechnic Institute and State University.

Visual Inspection and Delaminations

A visual inspection and delaminations survey of an area about 4.3 × 11.3 m (14 × 37 ft) encompassing the impregnated test area was performed at the south end of the northbound traffic lane in March 1989. Approximately 20 percent of the nonimpregnated area was delaminated or spalled from corrosion of the reinforcing steel, whereas the impregnated area remained sound with no patched spalls, spalls, or delaminations. One corner of the impregnated area was patched as part of a repair to an adjoining spall in the untreated area.

Two deck sections, one containing the impregnated area and the other an untreated area to be used as a control, were transported to Virginia Tech's Structures and Materials Lab

TABLE 7 Concrete Resistivity and Resistance Measurements for Boalsburg Bridge

Statistical Parameter	Impregnated Section 1989 Nippon Steel K ohm·cm	Impregnated Section 1992 Gecor K ohm	Control Section 1989 Nippon Steel K ohm·cm	Control Section 1992 Gecor K ohm
Mean	9.95	1.57	6.72	0.72
Standard deviation	1.66	0.44	1.91	0.17
Number observations	5	8	5	7

in May 1990. Upon arrival, a visual inspection and delamination survey was conducted on each of the deck sections.

The impregnated slab was 3.5 m^2 (38 ft^2) in area. One of the corners of the slab, 0.3 m^2 (3.6 ft^2), was damaged during removal. An additional 0.2 m^2 (1.9 ft^2) of one corner had been patched during the repair of an adjacent spall in an untreated area of the original bridge deck. No delaminations were located. The damaged corner was repaired with LMC.

The control slab was 4.4 m^2 (47 ft^2) in area. A delamination survey revealed that 1.0 m^2 (10.4 ft^2), roughly 22 percent, of the control slab was delaminated. In addition, 0.3 m^2 (3.4 ft^2) of the slab had been previously patched. The concrete in the damaged area was removed to the bar level with a 23-kg (50-lb) jackhammer. The exposed rebar was sandblasted to near-white metal. The damaged areas were patched with LMC, and a 51-mm (2-in.) LMC overlay was placed over the entire slab in July 1990.

Cracking between the overlay and the substrate concrete was noticed on the edges of the control slab in April 1991. A subsequent delamination survey found that 71 percent of the overlay had delaminated from the substrate concrete. These delaminations were the result of poor construction practice.

After the delaminated concrete was removed, a second delamination survey on the substrate concrete located an additional 0.1 m^2 (1.3 ft^2) of corrosion-induced delamination. The delaminated concrete was removed, the exposed rebar was cleaned of corrosion products, and a new LMC overlay was placed at night.

The most recent visual inspection and delamination survey was conducted in June 1992. Neither the impregnated slab nor the control slab showed any sign of deterioration at this time.

Concrete Cover Depth and Chloride Content

Before the deck replacement, a pachometer was used to determine the depth of concrete cover over the reinforcing steel. Forty measurements were taken, 20 within and 20 outside of the impregnated area. The mean cover depth is 37 mm (1.45 in.) with a standard deviation of 10 mm (0.40 in.) and a range of 25 to 50 mm (1.00 in to 2.00 in.) Thus, the depth of cover is relatively small, which would account for the premature deterioration of the deck concrete.

During chloride sampling the top 6-mm (0.25-in.) sample was discarded and four samples were taken at depth increments of 13 mm (0.50 in.) with the fifth sample increment being 25 mm (1.00 in.). Chloride content samples were taken in the left wheelpath at 10 locations, 5 within and 5 outside the impregnated area. The mean depth of the samples were 13, 25, 38, 51, and 70 mm (0.50, 1.00, 1.50, 2.00 and 2.75 in.). Table 8 presents the average chloride content for the wheel path areas as a function of the depth for the 1984 and 1989 samplings. The chloride content at the depth of the reinforcing steel was above the corrosion threshold value in 1975 (*3*), and there was not a significant difference between the chloride contents of the impregnated area and the rest of the deck as a whole at the 95 percent confidence level (*4*). Thus, any difference in the chloride contents between the impregnated section and the nonimpregnated section should be a measure of the efficacy of impregnation to reduce the rate of chloride diffusion into concrete.

As presented in Table 8, chloride contents in both the impregnated and control area appeared to have continued to increase between 1989 and 1984. The difference in chloride contents between the measurements indicate that more chloride penetrated the impregnated concrete than the control concrete for the top 1 in. of concrete. Dutta showed that at higher drying temperatures, such as those found near the deck surface during the impregnation process, not all of the water-accessible voids are filled by monomer (*5*). This is probably related to the large size of the organic monomer molecules. As a result, the impregnation process may make the top inch of concrete more susceptible to chloride intrusion.

However, for the depths of 38 to 70 mm (1.50 to 2.75 in.) the impregnated concrete excluded an average 0.9 kg chloride/m^3 (1.5 lbs/yd^3) of concrete. The average percent exclusion, which would be a measure of the effectiveness to prevent further chloride penetration, is about 60 percent for the depths of 38 mm (1.50 in.) and below. It is interesting to note that the chloride content at the depth of the reinforcing steel in the impregnated area remained above the threshold level of 0.7 kg/m^3 (1.2 lbs/yd^3) for 14 years without spalling or delaminating.

TABLE 8 Difference in Average Chloride Content, Right Wheelpath of Bethlehem Bridge

Depth mm.	Chloride Content, kg/m^3 Difference Between 89-84		Difference Con.-Imp.	Percent Excluded by Impregnation
	Control	Impregnated		
13	0.1	1.7	-1.6	--
25	-0.1	0.9	-0.8	--
38	1.5	0.7	0.9	58
51	1.1	0.4	0.7	63
69	1.8	0.6	1.2	67

Note: 1 lb/yd^3 = 0.5933 kg/m^3
1 in = 25.4 mm

Corrosion Potentials

Corrosion half-cell potentials were measured with a CSE. Table 9 presents the mean, standard deviation, number of observations, and the percentage more negative than −350 mV. Before 1990 the potentials for the nonimpregnated area increased slightly in the percent more negative than −350 mV. The corresponding percent area with a greater than 90 percent probability that reinforcing steel corrosion is occurring has increased 6 percent. However, there is no change in the mean or standard deviation. Thus, for all practical purposes there was little change in corrosion potentials of the nonimpregnated areas before deck replacement. In the same period the corrosion potentials in the impregnated area became more negative by 90 mV, from −260 to −350 mV, and the standard deviation increased by 30 mV. Also, the percentage more negative than −350 mV, indicating a greater than 90 percent probability that corrosion is occurring, has increased from 0 to 50 percent. Thus, it appears that the corrosion activity increased in the impregnated section during the period from 1984 to 1989 before replacement.

Once the salvaged impregnated slab and nonimpregnated control slab arrived at Virginia Tech, a 15-cm (6-in.) control grid was established on both slabs for corrosion potential measurements. CSE corrosion potentials were taken on the slabs in July 1990 before the initial repair, in July 1991 before the replacement of the control slab's LMC overlay, and in June 1992. The results for the control slab are divided into repaired and nonrepaired areas.

The results presented in Table 9 indicate that upon arrival at Virginia Tech the mean potential for the impregnated slab and the weighted mean of the repaired and nonrepaired sections of the nonimpregnated slab were approximately the same. After repair the mean potential in the repaired areas of the nonimpregnated slab decreased 61 percent. Though the mean potential of the nonrepaired area increased somewhat after placement of the first overlay, it has decreased after the placement of the second overlay. The reduction probably resulted from the second overlay reducing the amount of moisture at the bar level.

The mean potential of the impregnated slab decreased by 39 percent in the 2 years since it arrived at Virginia Tech. The reasons for this decrease are unclear at this time.

Microscopic Analysis

Seven cores 10 cm (4 in.) in diameter were drilled with a water-cooled diamond drill bit. In general, the cores were about 127 mm (5 in.) long. Vertical sections were cut, polished, and examined with the aid of a petrographic microscope.

The coarse aggregate is a blast furnace slag, and the fine aggregate, a natural sand. The cement paste appeared to be of excellent quality. The depth of impregnation is approximately 76 mm (3 in.), which agrees with previous results (*3*).

The primary interest in the examination was the corrosion of the reinforcing steel since the purpose of deep impregnation is to prevent or arrest the corrosion process. Table 10 presents the summary of the corrosion performance examinations. The microscopic examination of polished and fractured sections revealed details of the corrosion state of the deck. With the exception of the Conn. Core, corrosion products were observed around the reinforcing steel. Cores A and D from the nonimpregnated area showed evidence of slight corrosion. All four cores from the impregnated area showed evidence of corroding reinforcing steel. However, the corrosion products were impregnated with polymer. Thus, the deep impregnation process successfully arrested active corrosion cells in all four of the cores. Figure 1 presents a microphotograph of impregnated Core J, showing a vertical crack caused by corrosion of the rebar. The microscopic investigation presents di-

TABLE 9 Half-Cell Potentials for Impregnated and Control Slabs, Bethlehem Bridge

Date	Mean -mV	σ_x mV	No.	More Negative Than -350 mV %
Impregnated Area				
1984	260	60	5	0
1989	350	90	18	50
1990	342	40	126	47
1991	278	39	138	3
1992	207	41	138	0
Non-Impregnated, Non-Repaired				
1984	340	110	34	41
1989	340	110	134	47
1990	313	55	119	37
1991[1]	381	68	119	66
1992	285	55	119	14
Non-Impregnated, Repaired				
1990[2]	454	70	29	97
1991	178	71	41	0
1992	193	65	41	0

[1] Readings taken after the delaminated overlay was removed.
[2] Readings taken prior to repairs.

TABLE 10 Microscopial Examination of Polished Vertical Sections from Cores from Bethlehem Bridge Deck

Core No.	Impreg. Area	Cracking Observed: Drying Shrink	Cracking Observed: Micro Shrink	Cracking Observed: Subsidence	Rebar Corr.	Cover mm.	Corr. of Rebar
A	No	No	No	No	No	56	Slight
D	No	No	No	No	No	33	Slight
Conn	No	Few	No	No	No	36	No
F	Yes	Two	No	Yes	No	38	Y/Arrested
H	Yes	Few	No	No	No	38	Sl/Arrested
I	Yes	(a)	No	(a)	(a)	30	Y/Arrested
J	Yes	No	Minor	No	Yes	36	Y/Arrested

(a) Two cracks originating from rebar -- cannot discern if shrinkage, subsidence, or rebar corrosion related, but definitely pre-exist impregnation.
Note: 1 in. = 25.4 mm

rect evidence that the deep polymer impregnation process will arrest the corrosion of steel in concrete. These results verify the previous results of deep polymer impregnation arresting the corrosion of steel in concrete (*4*).

Corrosion Rate

In March 1990, 1 year after the previously described investigation, eight corrosion rate measurements were taken in both the control and impregnated areas. Table 11 presents the results of the 3LP corrosion rate measurements. The rate of corrosion in the impregnated section for the March 1990 measurements is significantly less, 15 times slower, than the rate of corrosion in the control section before replacement.

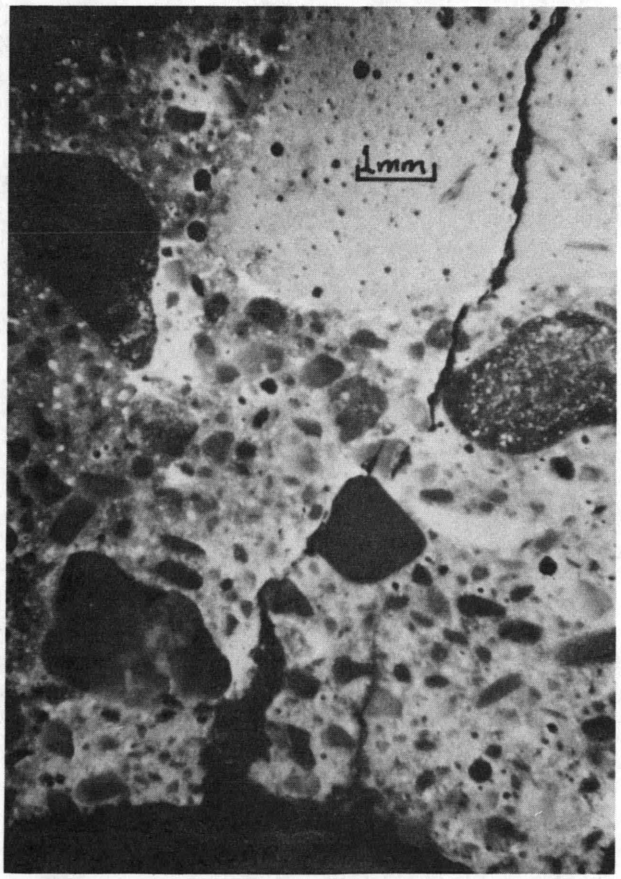

FIGURE 1 Core J, Bethlehem bridge rebar corrosion and associated vertical crack.

Additional 3LP corrosion rate measurements (i_{corr}) were taken on both the impregnated and the control slabs salvaged from the Bethlehem Bridge deck. It can be seen that the corrosion rate as measured by both the 3LP and Gecor devices is approximately a factor of 10 less in the impregnated area as compared to the control areas. From the 3LP measurements it may be estimated that damage may be expected in the impregnated slab in 10 to 15 years, whereas damage would be expected in the control slab in 2 to 10 years (*6*). It must also be noticed that the mean 3LP i_{corr} for the impregnated slab borders on the threshold of no corrosion activity, given at 0.22 $\mu A/cm^2$ (0.20 mA/ft^2) (*6*).

CONCLUSION

The major findings of this paper are that deep polymer impregnation will reduce the diffusion of chloride ions at the bar level, increase the resistivity of the concrete, and tend to significantly reduce the corrosion rate expected in a similar untreated deck.

The Bethlehem test section has shown no sign of deterioration in the 18 years after impregnation, even though the chloride content was greater than the 0.71 kg/m^3 (1.2 lb/yd^3) at the time of impregnation. This indicates that the in situ polymerization was sufficient to encapsulate the upper rebar mat. The mean 3LP i_{corr} for the impregnated section has remained slightly above 0.22 $\mu A/cm^2$ (0.20 mA/ft^2) during the past 3 years of testing, which suggests that damage may be expected in 10 to 15 years. However, it must be noted that for 3LP i_{corr} rates less than 0.22 $\mu A/cm^2$ (0.20 mA/ft^2), no corrosion damage is expected. Therefore, it may be assumed that damage might be expected in 15 years even though the i_{corr} has shown no sign of increase over the past 3 years. Based on these observations of the Bethlehem trial section, it can be estimated that deep polymer impregnation will increase the service life of a bridge deck by a minimum of 30 years, 18 years current service since impregnation and 15 years ex-

TABLE 11 Corrosion Rate Measurements for Bethlehem Bridge Slabs

Date	3LP Corrosion Rates Mean $\mu A/cm^2$	σ_{x-1}	Number	Gecor Corrosion Rates Mean $\mu A/cm^2$	σ_{x-1}	Number
Impregnated Slab						
March 1990	0.28	0.18	8	NA	NA	NA
July 1990	0.23	0.19	21	NA	NA	NA
June 1992	0.25	0.30	24	0.02	0.01	13
Control Slab, Repaired						
July 1990[1]	5.04	3.55	12	NA	NA	NA
July 1991	3.84	0.53	12	0.19	0.32	6
June 1992	1.71	0.74	12	0.11	0.05	7
Control Slab, Non-Repaired						
March 1990	4.28	1.43	8	NA	NA	NA
July 1990	1.60	0.71	18	NA	NA	NA
July 1991[2]	3.63	1.36	18	0.50	0.31	13
June 1992	2.26	0.45	18	0.10	0.04	14

[1] Readings taken prior to repairs.
[2] Readings taken prior to removal of delaminated overlay.
Note: 1 mA/ft_2 = 0.929 $\mu A/cm^2$

pected service until damage occurs (based on 3LP i_{corr} measurements).

This prediction is further supported by a comparison of the i_{corr} rates of the impregnated and control section. The control section has undergone a typical rehabilitation with an LMC overlay which has an expected service life of 20 years. One year after rehabilitation, the control section is corroding at rate approximately 10 times greater than the impregnated section, 18 years after impregnation.

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