Deep Impregnation of Concrete Bridge Decks

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The deep monomer impregnation (depth of impregnation 3 to 4 in.) and in situ polymerization of a bridge deck using the grooving technique is presented. The study shows that the process is commercially feasible and the work can be successfully completed to a set of specifications by contractors with no experience with the monomer impregnation and in situ polymerization process. Laboratory estimates of operation times are compared with field performances. Field operation times were significantly less for the impregnation time and the polymerization time but slightly greater for drying times. Safety procedures and cost estimates are also presented. The deep impregnation process is shown to be cost competitive with cathodic protection.

The nation's bridges continue to deteriorate at an alarming rate. In June of 1985, the Federal Highway Administration reported that about 75,000 bridges on the federal aid system and about 184,000 bridges off the federal aid system were deficient (*1*). Essentially, there has been no reduction in the backlog of deficient bridges despite significant increases in bridge rehabilitation and replacement efforts by the states. The 1986 rehabilitation or replacement upgrade cost for all the deficient bridges was about $48.3 billion, about $3 billion more than the 1984 estimate. Approximately one-half of the deterioration cost is related to concrete bridge decks with much of the deterioration related to chloride deicer salts penetrating the concrete and corroding the reinforcing steel (*2*).

The average bridge deck in the snow belt constructed with uncoated reinforcing steel with a 2-in. average cover depth will begin to spall about 7 yr after construction and will require rehabilitation at an age of 22 yr (*3*). This implies that one-half of the bridges constructed with uncoated reinforcing steel and 2 in. of cover will deteriorate at a more rapid rate.

In 1973, the first bridge deck to be constructed with epoxy-coated reinforcing steel was built in West Conshohocken, Pennsylvania. To date, it appears that epoxy-coated reinforcing steel will significantly increase bridge deck life (*4, 5*). However, even in Pennsylvania, the pioneer in the use of epoxy-coated reinforcing steel, acceptance was slow. Of 625 new bridge decks built in Pennsylvania from 1973 to 1978, 468 were built with uncoated reinforcing steel, 90 with galvanized reinforcing steel and only 67 with epoxy-coated reinforcing steel (*5*). More than half (36) of the new bridges built between 1973 and 1978 in Pennsylvania using epoxy-coated reinforcing steel in the deck were built in 1977 (22) and 1978 (14). Thus, presently there exists a significant number of bridges built with uncoated reinforcing steel that are still in sound condition, but these will begin to deteriorate in the near future.

From 1967 to 1975, extensive laboratory testing clearly demonstrated the capability of deep impregnation to combat the bridge deck problem (*6–12*). Deep impregnation consists of drying the concrete, using propane fired infrared heaters, to the desired depth of impregnation, soak-impregnating the concrete with a monomer, and thermally polymerizing the monomer in situ. The monomer system is a mixture of 100:10:0.5 parts of methyl methacrylate, trimethylolpropane trimethacrylate (promotor and cross-linking monomer) and 2, 2-azobisisobutyronitrile initiator (MMA-TMPTMA-AZO). Deep impregnation stops corrosion by encapsulating the chloride, replacing the corrosion cell electrolyte (concrete pore water solution) with a dielectric material (polymer), and restricting the ingress of moisture and oxygen needed in an active corrosion cell by partially filling the capillary void system.

In 1975, a small test section (3.5 ft by 11.5 ft) on an 8-yr-old heavily trafficked bridge deck near Bethlehem, Pennsylvania, was impregnated to a depth of 3 to 4 in. (*13*). At the time of impregnation no spalls or patches existed on the deck. However, the deck was critically contaminated with chlorides at the depth of the top reinforcing steel. In 1984, 9 yr after the impregnation, the deck had numerous spalls and delamination planes but there was no evidence of spalling or delamination in the test area (*14*). Spalling was adjacent to, and delamination planes extended to the borders of, the impregnated area, but was not within it. In addition, the surface wear of the impregnated area was 65 percent less than the surrounding nonimpregnated area and the chloride content within the impregnated area was significantly less at the 99 percent confidence level. A microscopic examination revealed the most significant finding, a preexisting corrosion cell that had been arrested by the impregnation process, and the deep impregnation

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significantly retarded the ingress rate of chloride at all depths even though the shrinkage or thermal cracking, or both, was not filled with polymer.

Although the deep impregnation by soaking was shown to be capable of stopping reinforcement corrosion, two problems remained. First, the time required for the 3- to 4-in. deep impregnation was too long, about 4 days. Second, large, nonhorizontal deck cracks required large excesses of monomer in order to pond their surface. Also, problems of containment of the monomer and potential hazards of having a large area of monomer, a highly volatile and flammable material, exposed during the impregnation process had to be addressed. The grooving technique (*15, 16*) alleviated these problems. Grooves cut along lines of equal elevation act as vessels for the monomer and minimize the amount of monomer while reducing the exposed monomer surface area. Because the impregnation takes place through the sides and bottoms of the grooves, 1- and ½-in. deep grooves reduced the 4-in. depth impregnation time from about 4 days to about 16 hr. The grooves are cut to a depth of ½ in. above the top reinforcing steel and the width and spacing are sized to accommodate the total volume of monomer required to impregnate the concrete to the desired depth.

However, small-scale laboratory tests and field trials of deep impregnation were not sufficient to resolve a number of significant questions that had to be addressed before the technique could become a commercially feasible field procedure. The questions included the effects of heating large areas of the deck to the temperature required for rapid and adequate drying, potential problems of bridge geometry on groove cutting, ability of the grooves to provide adequate containment after drying, means of providing effective weather protection during drying and impregnation, and potential problems in providing uniform groove-filling in the field. Also, there is a question of whether a typical bridge contractor, unfamiliar with the process, would be capable of impregnating a bridge deck to a given set of specifications.

The following presents the results of a full-scale deep impregnation of a bridge deck using the grooving technique to determine the commercial feasibility of deep impregnation and to compare laboratory results with field results.

TEST BRIDGE

The test bridge is a three-span multigirder bridge with simply supported steel plate girders, permanent steel deck forms, and composite design. The end spans are 42 ft and 38 ft and the center span is 131 ft. The deck width, curb to curb, is 44 ft (two 12-ft traffic lanes and two 10-ft aprons). The deck concrete was placed in April 1972 and the first live load (construction equipment) application occurred on May 12, 1972. The bridge is located on the Mt. Nittany Expressway (US-322) over Pennsylvania Route 45 near Boalsburg, Pennsylvania. The bridge is on a skew, 7 degrees, 40 minutes, 03 seconds, essentially on a tangent, and is on a slight upgrade of about 1.4 percent. According to the deck plans, the traffic lanes are cross sloped ⅛ in./ft and the aprons are cross sloped ⅜ in./ft. The design deck thickness is 8 in., with a 2-in. minimum cover depth. The main reinforcement (transverse direction) is made up of No. 5 bars on 6-in. centers, top and bottom. The top longitudinal bars are No. 4 bars 12 in. on center, and the bottom bars are No. 5 bars 9 in. on center. The concrete mixture was Pennsylvania Class AA concrete using No. 57 crushed limestone and a natural bank sand with a 28-day compressive strength of 3,750 psi. The design slump and air content values were 2.5 in. and 6.5 percent. Measured slump values averaged 2.25 in. and the air content varied from 5.4 to 8.0 percent. Averages of two concrete compressive strength cylinders were 3,440 psi at 6 days and 3,643 psi at 10 days.

Sixty ft, or approximately one-half of the center span, was selected for the deep impregnation trial installation. The remainder of the span is to serve as a control for future performance reference purposes. The bridge had been open to traffic for 13 yr before the trial deep impregnation.

PRELIMINARY TEST WORK

Precise leveling survey was performed on the test area to establish the equal elevation groove cut lines. The leveling survey elevations and mean directions are presented in Figure 1. The determined groove orientations were subsequently verified using a 6-ft spirit level.

A hand-held pachometer was used to take rebar depth of cover measurements at a sufficient number of points to determine the distribution of the rebar depth at a statistical significance level comparable to the reported accuracy of the instrument (*17*). The average cover is 2.86 in., with a range of 2.3 to 3.3 and a standard deviation of 0.22 in. Thus, there is a probability of about 1 in 20,000 of having any steel in the deck with a cover depth of less than 2 in. A rolling *R*-meter (pachometer) set at a cover depth of 2 in. verified the hand-held results by showing no reinforcement with less than a 2-in. cover depth.

The groove width, depth, and spacing are interrelated functions of reinforcement depth and impregnation rate and time. Three 4-in. diam by approximately 6-in. deep cores were taken to determine the rate of impregnation and percent by weight of polymer loading. The cores were dried in an oven at 230°F ± 5°F for 72 hr, allowed to cool and be soak impregnated from the top surface only for 16 hr using the MMA-TMPTMA-AZO monomer system, and polymerized in a hot water bath. The results of the depth of impregnation for the three cores are presented in Table 1. The average 16-hr impregnation was 2.9 in., unit weight of the unimpregnated concrete was 141 lb/ft^3 and the monomer loading was 3.5 percent by weight.

Using previously developed procedures (*18*), various combinations of groove dimensions and spacing and impregnation times were evaluated. However, the primary consideration for this deep impregnation test trial was to evaluate a combination of factors that are representative

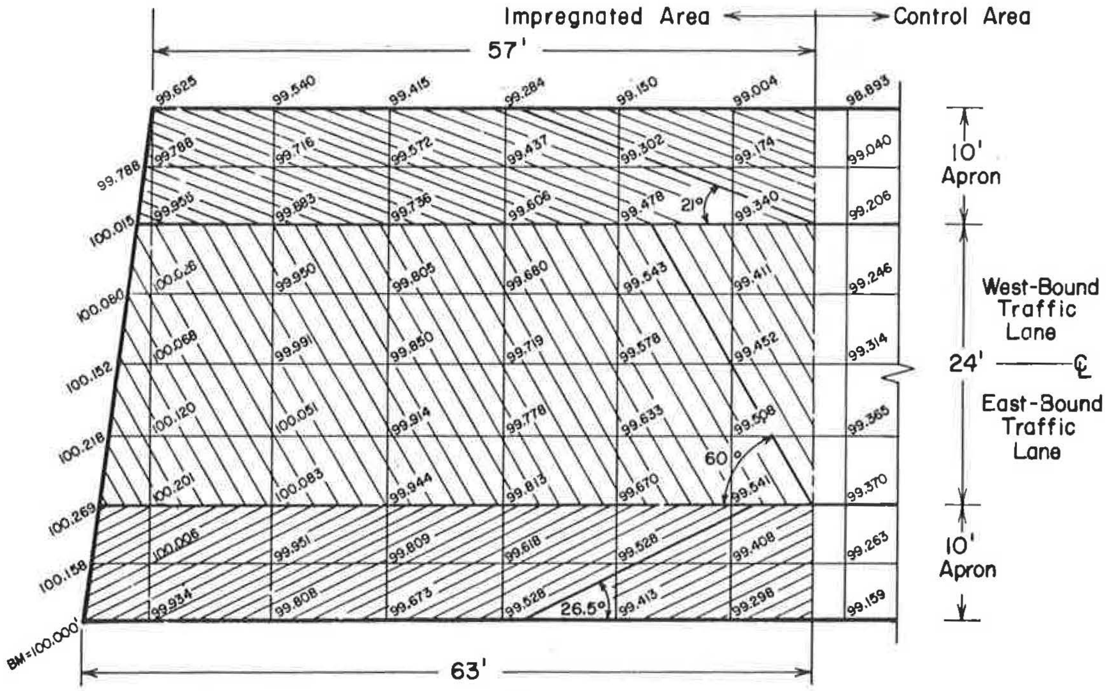

FIGURE 1 Results of precise leveling survey on deck surface and resulting groove orientations (indicated by direction of cross-hatching).

of typical bridge decks. Therefore, the groove width, depth, and spacing were determined for a typical cover depth of 2 in., depth of impregnation of 3.63 in., and the impregnation time based on the rate of impregnation of the test cores. Given these conditions, the following groove-impregnation characteristics were selected:

Groove width = 0.75 in.,
Groove depth = 1.50 in.,
Groove spacing = 3.00 in. center to center, and
Impregnation time = 16 hr.

The estimated quantity of monomer was 3,950 lb, based on an average depth of penetration of 4 in. and 3.5 percent by weight monomer loading determined from the cores.

GROOVE-CUTTING OPERATION

Approximately 11,000 lineal ft of grooves had to be cut to cover the 2,640-ft^2 test area. The specifications required that the grooves extend to within 1 ft of the curb lines, and meet the following tolerances:

1. Groove spacings:
   (a) ± 0.25 in. between any two adjacent grooves,
   (b) number of whole groove widths (including the equivalent of partial width at ends) over any 10-ft length measured perpendicular to grooves = 40 ± 1,
2. Groove width: ± 0.0625 in.,
3. Groove depth: from a straight edge resting on the pavement surface to all points vertically below on the groove root: ± 0.125 in.

The contractor used a standard water-cooled concrete saw with two diamond set blades sandwiching a smaller diameter abrasive cut-off wheel to cut the groove width in one pass. The grooves were cut one at a time with snap lines set about every 5 ft for controlling the groove orientations. A wheel and guide on the front of the machine assisted in maintaining proper groove spacing between adjacent grooves. The groove-cutting operation is illustrated in Figure 2.

Some early problems were experienced by the contractor's forces in maintaining the direction and spacing of the grooves. However, after cutting about five grooves (about 60 lineal ft), they became accustomed to the operation and were producing acceptable work at a rate of 120 ft/hr. The groove spacing and depth were within specifications for the entire job. However, the groove width was generally 0.125 in. narrower than the 0.75 specified width, or about 0.6 in. narrower than the lower specification limit. This

TABLE 1 IMPREGNATION OF PRELIMINARY TEST CORES

Core No.	Length (in.)	16-hr Impregnation Depth (in.)
1	4.7	3.0
2	4.5	2.8
3	2.8	2.8[a]
Average		2.9[b]

[a]Complete penetration.
[b]Omitting Core 3.

FIGURE 2 Groove-cutting operation.

deviation was considered acceptable because the depth of the grooves was 0.1 in. greater than 1.50 in. (the depth and width deviations offset each other).

There were no significant problems with the groove-cutting operation. Only minor learning difficulties were experienced. This was also true for the small-scale laboratory trial impregnations. The task was time consuming but improvements can be made by using larger equipment with gang saws. Also, the removal of the sediments is a problem if they are allowed to dry out in the grooves. Any equipment development should include a tailings vacuum system.

WEATHER PROTECTION

For the drying and impregnation phase of the deep impregnation process, decks need to be protected from precipitation and surface runoff. A tent arrangement was developed consisting of heavy plastic tarpaulin supported on half-arch pipe sections attached to the parapets and railings and supported by cables. The tent was subjected to several periods of moderately heavy rainfall up to ½ in. and 20 mph winds. Water collected in sags of the tent and threatened to collapse it. The problem was eliminated by using lollipop support props in the tent.

Surface runoff was collected by two diversion dams constructed with asphalt cold mix and sealed with asphalt emulsion. Four-in. diam holes through the deck in front of the second dam on each side of the deck drained the water from the deck.

The performance of the weather protection devices was exceptional. The deck remained dry during the drying and impregnation phases.

DRYING

The drying equipment was specially designed and built for the contractor. The drying train consisted of six units, 36 in. deep, 60 in. wide, and 86 in. long. The train formed by bolting the six units together was able to dry a 43-ft long section (the full width of the 44-ft wide deck) covering 5 ft of the bridge at a time. Each unit housed three 120,000 BTU/hr, propane-fired, infrared radiant heaters operating at 3 psi pressure. Pressure regulators were installed in the fuel line of each heating element and permitted individual heating adjustments for the 18 heater elements.

TABLE 2 SURFACE HEATING RATE SPECIFICATIONS

Time (min)	Surface Temperature °F
Start	Ambient
15	375 ± 25
30	475 ± 25
45	575 ± 25
Until dry	575 ± 25

To minimize thermal gradients and thus thermal stresses, the heating rate was controlled by surface temperatures in accordance with the specifications presented in Table 2. In addition, the dried areas were covered with R-19 glass wool insulation immediately after the heaters were removed to reduce thermal gradients during cool down. A 24-in.-wide strip of R-19 glass wool insulation was placed on the deck in front of the heater to reduce heating losses and to reduce thermal gradients in front of the heating train. The front side of the heating train is shown in the photograph in Figure 3.

Small scale laboratory drying trials with a 600°F surface temperature showed that drying to a depth of 4 in. below the surface took about 3.5 hr at an ambient temperature of 75°F (*18*). The drying times on the trial deck impregnation took somewhat longer and ranged from 3.9 hr to 6.0 hr, with an average of 4.6 hr for the 14 drying operations (4.5-ft advance with 0.5 ft overlap per setup). The mean ambient temperature was somewhat lower than 75°F and ranged from 57°F to 82°F, with a mean ambient temperature of 60°F.

The increased drying times were most likely related to lower temperature experienced in the field and wind veloc-

FIGURE 3 Front side of heating train during drying.

ities not experienced in the laboratory. To determine when the concrete is dry at the desired depth of impregnation, it appears necessary to measure the temperature of the concrete at the desired depth of impregnation. The concrete is to be considered dry at a temperature of 220°F. For thermocouples set from the top of the deck to measure the temperature at the desired depth of impregnation, a correction factor must be applied to account for the false high temperatures caused by the conduction of heat to the junction of the thermocouple. Laboratory experiments indicated the correction factor to be about 50°F. However, field measures indicate the correction factor to be about 25°F.

Laboratory and small-scale field trials showed that the high surface temperatures caused shrinkage or thermal cracking, or both. Generally, these cracks were minor and extended to a depth of about 1 in. Fine drying shrinkage or thermal cracks were also observed in the field trial. These cracks are generally oriented perpendicular to the groove directions. A typical shrinkage or thermal crack is shown in Figure 4.

One week after the field impregnation trial was completed (i.e., backfilling of the grooves), 12 4-in. diam cores by approximately 6 in. in depth were taken. Three were taken from the control section and 9 from the impregnated section. The shrinkage or thermal cracks were observed in the impregnated cores and generally ranged in depth from 0.10 in. to 1.35 in. and were not filled with polymer. Only in one case did a shrinkage or thermal crack exceed the depth of the groove (1.5 in.). That crack depth was 2.98 in. However, a core taken from the control area also contained shrinkage cracks to a depth of about 0.60 in. and there was no significant difference in the cracking between the impregnated area and the control (unimpregnated) area. A microscopic examination of the other two cores taken from the control area was not performed because these two cores were taken for compressive strength tests.

The shrinkage or thermal cracks were visible to the unaided eye during the heating phase of the drying cycle for both laboratory and field trials. These cracks were not visible on cooling and presented no problems during impregnation for either the laboratory or field trial impregnations.

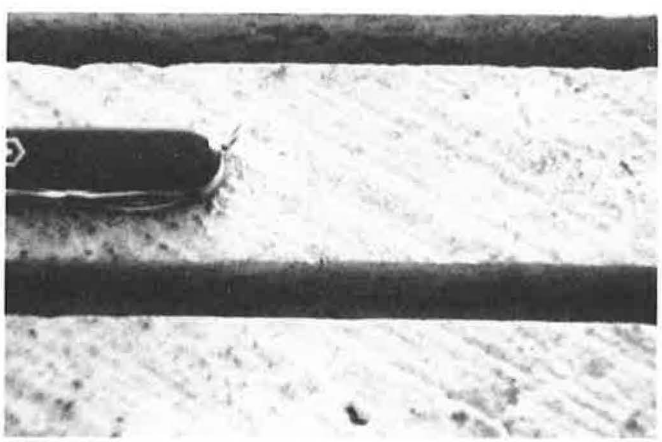

FIGURE 4 Typical minor drying shrinkage/thermal crack.

FIGURE 5 Two-man crew filling individual grooves.

IMPREGNATION

Laboratory experiments indicated that 0.75-in.-wide by 1.5-in.-deep grooves cut to impregnate to a depth of about 4 in. would empty in about 16 hr (*18*). The filling of the grooves with monomer was carried out by three 2-man crews working simultaneously. Groove filling was done at the ends of the grooves; polyethylene sheets covering the deck were folded back just enough to expose the groove ends, thus minimizing direct exposure of the monomer to the atmosphere. All the grooves were filled in about 4 hr. The grooves were refilled as the concrete absorbed the monomer. Refilling continued until all 14 drums (5,600 lb) of monomer were used (4,000 lb or 10 drums was the estimated amount required to impregnate to a depth of 4 in.). The entire process, from mixing of the first drum until the last drum was emptied, took about 6.5 hr. A two-man crew filling the grooves is shown in Figure 5. The monomer was allowed to soak for an additional 15 hr. However, it appeared that all of the monomer that was going to soak in did so within the first 4 hr.

The reduction in the field impregnation time from the estimate of 16 hr based on laboratory results to about 4 hr is most likely related to the higher field drying temperatures (600°F field surface temperature, 450°F at 1 in., 380°F at 2 in., 300°F at 3 in. and 220°F at 4 in. at the end of the heating cycle compared with a 230°F oven-drying temperature).

As previously stated, the estimate of monomer needed to impregnate the deck test area to an average depth of 4 in. was 3,950 lb based on the laboratory loading of cores of 3.5 percent by weight. A total of 5,600 lb of monomer was placed in the grooves. Approximately 1,000 lb of excess monomer was vacuumed from the grooves after 21.5 hr of soak impregnation time. It is difficult to estimate vaporization losses, but it appears that about 4,000 lb of monomer soaked into the deck. Therefore, grooves should be only filled once and the polymerization process begun immediately once the grooves are empty.

POLYMERIZATION

Hot water ponding polymerization in the laboratory on full-depth simulated 6 ft^2 deck slabs indicated whether the water was maintained at about 205°F; the concrete temperature at a depth of 4 in. reached a steady state temperature at 122°F in about 16 hr. The polymerization time for impregnated concrete at 122°F is about 4.5 hr. Therefore, the estimated total polymerization time is about 21 hr.

Precast concrete barriers placed across the ends of the test section and the parapets acted as the lateral supports for the bridge hot water polymerization pond. A vinyl tarpaulin was used to cover the deck and act as the hot water pond containment vessel. The weatherproofing tent was spread over the deck surface to protect the vinyl tarpaulin. Two distribution heaters, one on each of the two 200-hp portable boilers, injected live steam into the 30,000 gallon polymerization hot water pond. The minimum depth of 10 in. was maintained at the highest elevation point within the test area. The surface of the hot water polymerization pond was open to the atmosphere during the polymerization process.

Except for leakage and evaporation losses, the heating system was a closed loop. The boiler feed was drawn continuously from the water bath. Boiler No. 1 was fired and boiler No. 2 came on about 2 hr later. The temperature of the pond was slightly less than 200°F 6 hr after boiler No. 2 came on.

The temperature at a depth of 4 in. in the concrete reached a steady level of 135°F (123°F actual temperature, corrected for thermal conductivity). The temperature of the pond was difficult to maintain at 200°F because of water leakages and evaporation losses, which had to be replaced. Pond and concrete temperatures throughout the polymerization process are presented in Figure 6.

The polymerization process took about 17 hr or about 4 hr less than the estimated time of 21 hr. This occurred in spite of the adverse weather (temperatures of 45°F to 60°F, sporadic light rain, and a steady northwest wind at about 20 mph) and equipment malfunctions and water loss that kept the temperature 10°F below the desired 205°F. Thus, it appears that hot water polymerization of large areas is more efficient than small laboratory test slabs.

GROOVE FILLING

The grooves were backfilled with a latex-modified mortar with a 10-in. slump using rubber-edged squeegees to distribute and compact the mortar. The grooves were easily filled in 1 working day. The groove-filling operation is shown in Figure 7 and a close-up of the surface after 1 day is shown in Figure 8.

Sections of cores 1, 2, 3, and 6 were subjected to 300 cycles of rapid freezing and thawing in water (ASTM C 666, Procedure A). The primary purpose of freeze-thaw testing was to evaluate the performance of the latex-mod-

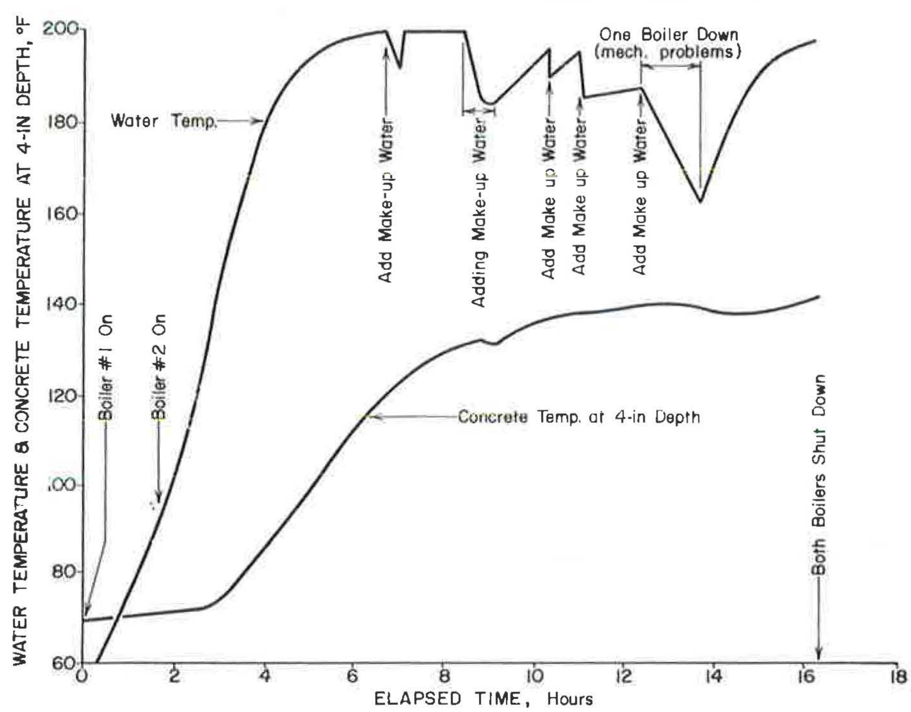

FIGURE 6 Deck polymerization temperatures.

FIGURE 7 Groove-filling operation.

ified mortar groove filler. The range of results is presented in Figure 9. As shown, the latex-modified mortar groove filling fared well. In most instances, nil to very light scaling of the groove filling occurred and the groove filling remained intact. The photographs presented in Figure 9 also illustrate the expected superior performance of polymer-impregnated concrete. The dashed lines indicate the approximate depths of impregnation and the arrows the groove filling.

SAFETY PROCEDURES

Potential safety hazards inherent in the process of deep monomer impregnation of bridge decks are related to the nature of the chemicals used. The monomer is volatile and flammable, and its vapor is explosive (explosion limits of 2.12 to 12.5 percent). Therefore, the prevention of sources of ignition, the minimization of monomer exposure to the atmosphere, and the provision of emergency facilities must be thoughtfully provided for.

Fire protection was provided during the period beginning with the mixing of the monomer until the completion of the polymerization. The fire-fighting facilities were staged

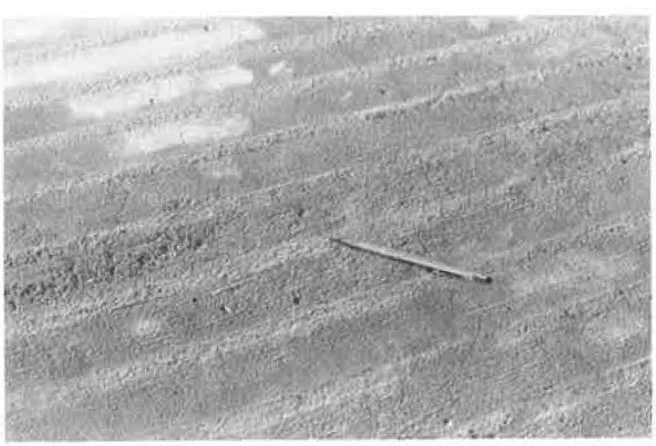

FIGURE 8 Deck surface 1 day after groove filling.

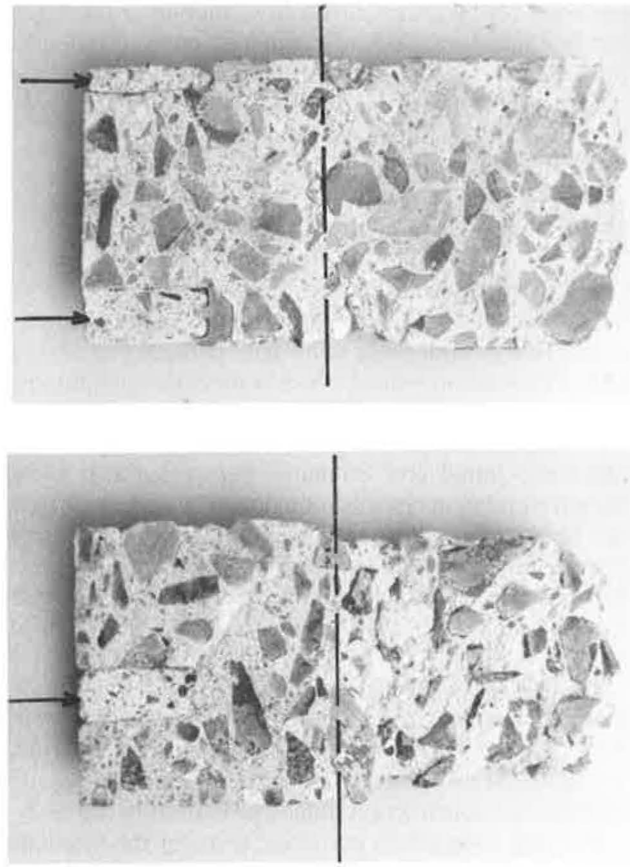

FIGURE 9 Condition of core remnants after 300 freeze-thaw cycles in water.

upwind, beyond the monomer mixing and distribution area. Water and foam facilities were provided. The catalyst was added to the monomer and mixed in electrically grounded 55-gallon drums with air-driven, propeller-type stirrers. Polyethylene sheets covered the deck during the monomer groove-filling and impregnation operation and minimized monomer evaporation. The air in the weather protection tent and below the bridge was checked at frequent intervals for monomer vapor concentrations. Concentrations remained well below the lower explosive limit (2.12 percent) throughout the groove-filling and impregnation operations. At the end of the soaking period, the polyethylene sheeting was removed and the excess monomer remaining in the grooves was vacuumed up using an air-motor-driven, explosion-proof industrial vacuum unit. This step proved to be the most potentially dangerous activity of the entire operation. Monomer vapors in the atmosphere within a radius of about 1½ ft from the vacuum exhaust showed concentrations typically in the range of 1.5 to 1.75 percent, but at times exceeded the lower explosive limit of 2.12 percent.

In addition to fire and explosion hazards, the chemicals used are toxic to varying degrees. The monomer components are considered to be moderately toxic (primarily irritants). Therefore, personnel protection against skin contact and breathing high vapor concentrations must be provided for. Workers involved in monomer mixing and distribution wore one-piece hooded coveralls, goggles, long rubber gloves, and dust masks. Those distributing the monomer to the grooves inside the tent wore canister masks as protection against organic vapors.

COST

Pilot projects, such as the one being reported here, always have associated with them extraordinary high costs related to the lack of contractor familiarity (risk factor) and suitable, efficient equipment. With respect to deep monomer impregnation and in situ polymerization, based on the experience of this project, process inefficiencies were identified and initial cost estimates were calculated using a capital equipment amortization rate of 10 percent. Because the amortization costs of large capital equipment are an inverse function of the square footage of bridge deck to be treated by a contractor per year, costs were determined for 1, 4, and 10 bridges per year using a typical bridge deck size of 44 ft wide, curb to curb, by 120 ft long. Obviously, larger bridges at a given location will result in lower unit costs. The total initial cost per ft^2 in 1985 dollars for 1, 4, and 10 bridges treated in a year by a contractor is \$16.98, \$13.05 and \$11.96, respectively. Unit cost per process and construction item is presented in Table 3.

For cost comparison purposes, costs for the installation of a cathodic protection system, the only other process capable of arresting the corrosion of black steel in concrete, were obtained for four 10-yr-old bridge decks. The installation of the cathodic protection systems was performed under one contract. The cathodic protection system used was a platinized wire primary anode with secondary carbon-strands anodes placed 1 ft on centers with transverse locations for redundancy and covered with a 1¼ in.-thick latex-modified concrete overlay. The initial 1985 cost per ft^2 for the four bridge decks, not including deck repairs carried out preliminary to the installation of the cathodic protection system, is \$13.34 (monomer impregnation and in situ impregnation work did not require preliminary deck repairs). For a valid comparison between deep impregnation and cathodic protection, it is necessary to compare life-cycle cost rather than initial cost because cathodic protection has additional future costs of electrical power, system maintenance, and periodic monitoring. These costs total, in 1985 dollars, \$0.13/ft^2. Using an average true (inflation-adjusted) interest factor of 5 percent *(19)*, the break-even point for cathodic protection and deep impregnation based on life-cycle costing is \$15.57/ft^2. This would occur at about two bridges/yr/contractor. However, it needs to be pointed out that the cost of cathodic protection was based on four bridges under a single contractor with both systems, cathodic protection and deep impregnation, having a 40-yr service life. Thus, on an equivalent comparison life-cost basis, deep impregnation of concrete bridge decks would be a least cost solution to corrosion protection over cathodic protection or no less than a cost-competitive solution.

TABLE 3 ESTIMATED INITIAL COSTS BASED ON VOLUME APPLICATION (\$/ft^2)

Item	No. of Bridges/Yr/Contractor 1	4	10
Grooving	1.82	1.75	1.57
Drying	2.15	1.29	1.08
Weather protection	2.76	1.49	1.23
Impregnation	2.67	2.35	2.29
Polymerization	1.16	0.56	0.40
Groove filling	0.71	0.71	0.71
Monitoring (process control)	0.33	0.25	0.23
Fire protection	0.45	0.45	0.45
Lightning and electric power	0.54	0.54	0.54
Construction superintendent	1.24	1.24	1.24
Mobilization	0.93	0.72	0.66
Traffic maintenance and protection	0.39	0.30	0.28
Surety bonds	0.08	0.07	0.06
Profit	1.05	0.81	0.74
General overhead	0.70	0.52	0.48
Total	16.98	13.05	11.96

RESULTS AND DISCUSSIONS

Nine 4-in. diam cores were taken from the impregnated area as stated previously. Four were along the center line at the joint of two heaters, three within the interiors of heating units, one in a heater overlap area and one next to the parapet. The field and laboratory impregnation depth measures for the nine cores are presented in Table 4. Cores 5, 7, and 12 were taken from areas under heating units and thus should represent typical condition. The depth of impregnation of about 3.5 in., which agrees with laboratory estimates, is indicated in Table 4.

The project clearly demonstrated the technical feasibility of deep impregnation of bridge decks and that it can be done on a commercial basis. A contractor who had no experience with deep impregnation was able to successfully impregnate a deck area of about 2,600 ft^2 to the depth of 3 to 4 in. Although the drying times were greater than laboratory estimates, impregnation times appear to be significantly less and field polymerization times also appear to be less than laboratory estimates.

ACKNOWLEDGMENTS

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TABLE 4 DEPTH OF IMPREGNATION DETERMINATIONS

Impregnation			Comments
	Depth (in.)		
Core No.	Field[a]	Lab[b]	
1	3.25	3.0	Cores 1 through 4 taken along center line, coincided with jointure of heating units; expect lowest impregnation depths. Also, looking for gradient because of time lapse before impregnation
2	3	3.2	
3	3	3.5	
4	2.75	3.5	
5	3.25	[c]	Interior of a heating unit (typical conditions)
6	4.5	4.6	Overlap area of heater set-ups (expect deepest impregnation)
7	3.25	[c]	Interior of a heating unit (typical conditions)
11	2 to 2.75	2.3	Core taken 6 in. from parapet (expect shallow impregnation)
12	3.75	3.4	Interior of heating unit (typical conditions)

[a]Acid etched along one narrow vertical line immediately after coring.
[b]Average of at least four measurements on etched face of vertical slab cut from core.
[c]Compressive strength specimen—not sectioned.

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