TRANSPORTATION RESEARCH RECORD

500

Formerly issued as Highway Research Record

Corrosion and Corrosion Protection

6 reports prepared for the 53rd Annual Meeting of the Highway Research Board

subject areas

27 bridge design
32 cement and concrete
33 construction
34 general materials

TRB
TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

Washington, D. C., 1974

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Transportation Research Record 500
International Standard Book Number 0-309-02290-8
Library of Congress Catalog Card Number 74-17765
Price: $2.20

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Transportation Research Record 500 was edited for Transportation Research Board by Mildred Clark.

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FOREWORD

Corrosion—one of the most aggravating problems encountered by maintenance, design, and materials engineers—does not appear to be diminishing. Currently much interest and attention are focused on corrosion of reinforcing steel in bridge decks. This RECORD presents 5 papers on this topic and 1 paper on the related subject of corrosion from stray earth currents.

A novel way of protecting the bridge deck reinforcing steel cathodically is the subject of Stratfull's paper. Here the principles of cathodic protection, used so effectively by pipeline people, are applied to a bridge deck. The paper describes the construction of such a system on a bridge in California and includes data showing that the installation successfully imposes sufficient potential on the steel to counter corrosion.

Clear discusses tests to determine what construction techniques and portland cement concrete mix design factors are most beneficial in delaying reinforcing steel corrosion under exposure to de-icing salts. Although tests are not yet complete, preliminary results show that low water-cement ratios (0.40), densification of concrete (98 percent of rodded unit weight), and at least 2 in. (50.8 mm) of concrete cover on the reinforcing steel will significantly reduce corrosion possibilities.

Crumpton and Bukovatz present performance data on Kansas bridges, identify corrosion distress as to source, and recommend preventive measures. This very practical and documentary report also includes similar observations on corrosion of structural steel.

Weed describes what specifications have been found necessary to achieve the desired cover depth on construction. These data are based on measurements of actual versus specified cover depth on New Jersey structures.

Protection of the reinforcing steel by coatings is described by Pike et al. Their studies show that certain epoxy powder formulations, applied by electrostatic spray techniques, provide durable coatings that have desirable protective properties but do not adversely affect concrete bonding characteristics.

Todd discusses work done on the BART system to identify and correct sources of stray current arising from negative return grounding connections. Described is a method of negative return grounding through diodes; tests indicate that this method corrects the problem. Installation of this new ground procedure is being effected.

Anyone interested in bridge deck corrosion and its correction or prevention should find these papers of interest and value.

—R. V. LeClerc

EXPERIMENTAL CATHODIC PROTECTION OF A BRIDGE DECK

R. F. Stratfull, California Department of Transportation

An electrically conductive asphalt concrete was made by substituting coke breeze for the natural aggregate. Paving a bridge deck with the electrically conductive asphalt concrete indicated that cathodic protection could be applied to the reinforcing steel. The cathodic protection was measured to be effective when the corrosion of steel strips embedded in concrete containing 10 percent calcium chloride by weight of the cement was stopped. It is estimated that, for the approximately 3,300 ft^2 (307.6 m^2) of bridge deck under cathodic protection, the top mat of reinforcing steel has an applied current density of 0.7 mA/ft^2 (7.5 mA/m^2) of steel surface. The total current used is about 1.0 A with a driving voltage of 1.65 V for a total power requirement of 1.65 W. As an experimental method of repair, 2 polymers and an epoxy were injected to bond the undersurface fractures. The epoxy could be injected in all cases where the concrete emitted a hollow sound when struck with a hammer. However, the epoxy could not be injected when the concrete emitted a hollow sound from only the use of the chain drag but not from the use of the hammer. The cost of the cathodic protection installation is estimated to be about $3/ft² of deck ($33/m²).

• A NUMBER of reports in the literature have related to the problem of bridge deck deterioration from the use of de-icing salts (1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12). In general, the reports have been concerned with detection and determination of causes of reinforcing steel corrosion and techniques and methods for structural repair and prevention of corrosion by use of waterproof membranes. Even though the techniques may not be applicable to bridge decks, one report (12) describes cathodic protection used experimentally to control corrosion of reinforcing steel in beams of a bridge superstructure. However, use of cathodic protection to inhibit ongoing corrosion of concrete-embedded steel has been well established for concrete pipelines (14, 15, 16, 17). This report presents results of experimental cathodic protection installation on a deteriorated bridge deck scheduled for repair and also describes the use of experimental monomer-polymers and epoxy injection to bond delaminated concrete.

That corrosion of steel in concrete is electrochemical in nature has been well established (18, 19, 20, 21). The theory of cathodic protection (22, 23, 24) is to apply sufficient current in a proper direction so that corroding anodes on steel are prevented for discharging current (ions) into the electrolyte or, in this case, concrete. Thus, if the anodes on the steel receive current, they are no longer current-discharging anodes but are noncorroding current-receiving cathodes.

In the corrosion cell, the tendency is for the half-cell potential of the steel to come into equilibrium (23, 24). For example, if the open-circuit potential of a noncorroding cathode is, say, -0.10 V (relative to the saturated copper-copper sulfate half cell, CSE), and the corroding anode is, say, -0.50 V CSE, and if then they are electrically connected together, the combined or equilibrium potential of both electrodes may be -0.40 V CSE. The cathode always becomes more negative as it receives current from the more negative anode. As a result, even though the cathode will shift to a potential of -0.40 V, it still is a cathode and may not be corroding.

Publication of this paper sponsored by General Materials Section.

From the theory of cathodic protection, the existing anodes on the steel, to be protected, must be caused to receive electrical current. For this, the half-cell potential of all of the steel must be made more negative than the most negative potential of the anodes. For steel pipelines, the empirical criterion for cathodic protection is that the steel must be made more negative than -0.85 V CSE (25). Although this potential value of -0.85 V CSE has been successfully used on concrete pipelines (16), it has also been reported (26, 27) that cathodic protection should be effective for steel in calcium hydroxide solutions containing chloride at potentials of about -0.71 V CSE. In addition, other work has shown that the most anodic half-cell potential of corroded steel in corrosion-caused cracked concrete was -0.67 V CSE (28). Excessive voltage, however, must not be used because of possible impairment of the concrete to steel bond.

It would appear that the potential of concrete-embedded steel probably should be no less than -0.85 V CSE (where reported past experience on concrete pipelines has been successful) and no more than -1.10 V CSE (18, 26, 27) to avoid the possibility of loss of bond strength. However, the possibility for controlling the corrosion of concrete-embedded steel at a potential of -0.71 V CSE should receive further investigation.

CATHODIC PROTECTION SYSTEM

There are 2 basic means (22) for applying cathodic protection: (a) galvanic anodes and (b) impressed current.

In the galvanic anode system, a sacrificial metal higher in the electromotive or galvanic series is chosen because its electrical potential is more negative than the metal to be protected. Therefore, when the 2 different metals are electrically connected, a current will flow causing the metal under protection to receive current or become a cathode. For a bridge deck, the galvanic system appears to have 2 important drawbacks: (a) voltage between the metals is limited to the maximum electrical potential difference and (b) current output of the galvanic anode will vary with moisture content or electrical resistance of the electrolyte.

For bridge decks, the impressed current system has a number of distinct advantages: (a) voltage output of the anodes can be varied from less than 1 V to more than 100 V, (b) current output can be automatically controlled irrespective of moisture content or electrical resistance of the electrolyte, and (c) half-cell potential of the steel can be automatically maintained independently of the electrical resistance of the environment.

Although various methods for applying cathodic protection to other types of structures are well known, a different system for applying cathodic protection would be required for a bridge deck. From this, it is apparent that the basic electrical circuit of steel in the deck concrete and a theory for applying cathodic protection must be developed and evaluated.

Figure 1 shows some assumed electrical values for the circuit. With the anode in the concrete, an adjacent reinforcing steel bar theoretically can be cathodically protected. However, as shown in the lower part of the figure, bars beyond the most immediate one (rebar 2) are actually in a series parallel circuit with an essentially 0 resistance between bars and power source. Theoretically, to cathodically protect all the reinforcing steel equally, anodes would have to be placed in concrete longitudinally and transversely at the location of every second bar.

To get an effective current to rebar 2 requires that the anode be removed from the concrete and placed in an electrically conductive overlay on the concrete surface to provide essentially equal resistance from all bars to the power source as shown in Figure 2.

One material that can be used as a conductive overlay is carbon in the form of coke, which has had a long record of use as an anode backfill material (29). In a dry state, coke is reported to have a specific electrical resistance of 52 Ω·cm (29), which is about twice the electrical resistance of seawater (23). Coke is a highly conductive material as compared to the about 10 000-Ω·cm resistance of water-saturated concrete (30), and when wet coke has about one-half the specific electrical resistance of seawater.

The feasibility of using a highly conductive overlay is shown in Figure 2. For purposes of illustration, it is assumed that the electrical resistance of the coke for the distance between the reinforcing steel is 1 Ω and the electrical resistance of the concrete between the interface of the coke and concrete to the surface of the steel is 100 Ω. From these assumptions and laws of electrical current flow, horizontal travel of electricity through coke would only be reduced by the ratio of 1-Ω resistance in the coke to the 100-Ω concrete resistance to the reinforcing steel as the current spread out through the deck. Therefore, this general method was chosen as the most promising method of applying cathodic protection to a deck.

BRIDGE CONSTRUCTION

The bridge selected for the experimental cathodic protection scheme was built in 1964 and is located on US-50 at Sly Park, California, at an elevation of 4,000 ft (1219 m). The average annual precipitation is 42 in. (107 cm), which includes about 18 in. (46 cm) of snowfall. The annual air temperature range is from 15 to 105 F (-9.4 to 40.6 C).

The bridge is a continuous 3-span T-beam that is 48 ft (14.6 m) wide and 110 ft (33.5 m) long; it carries 2 lanes of westbound traffic.

The specifications for the concrete in the bridge deck called for 7 sacks/yd^3 (9.2 sacks/m^3) of type 2 low-alkali cement, 4 to 4½ percent entrained air. Mixing water, including that in the aggregate, was not to exceed 45 lb/sack (20.4 kg/sack) of cement, and the concrete curing was specified to have 7 days of curing by water. A pigmented curing compound was to be applied following the wet cure.

In a review of the construction records, only 1 of the many concrete cylinders could be positively identified as being from the deck concrete. The identification ticket with the sample indicated that the concrete contained 7 sacks/yd^3 (9.2 sacks/m^3) of cement (ASTM type 2 modified, low alkali) and had a 4½-in. (11.4-cm) slump. The mixing water content was 44 lb/sack (20 kg/sack) of cement. The 28-day compressive strength of this cylinder was 3,730 lb/in.^{2} (25.7 MPa). The other samples from this bridge showed entrained air contents of 4.4 percent and 28-day compressive strengths ranging from 3,860 to 4,460 lb/in.^{2} (26.6 to 30.8 MPa).

The specified concrete cover over the reinforcing steel was 1½ in. (3.8 cm).

BRIDGE CONDITION

Because of corrosion-caused concrete spalling, the bridge deck had been scheduled for initial repairs and overlay during the 1973 construction season. All evidence of deterioration on this bridge is the result of reinforcing steel corrosion. There is no visual evidence of distress as a result of reactive aggregate or freeze-thaw damage.

In April 1972, this bridge was surveyed for concrete delamination, electrical potentials, depth of cover over the steel, and chloride content in preparation for the repair contract. The surveying technique has been previously reported (12). In addition, as part of the cathodic protection installation, the deck was again surveyed for concrete delamination and electrical potentials in June 1973.

Results of chloride analysis from cores, shown below, indicate the high level of chloride-ion at the level of the steel.

Depth (in.)	Lb/Yd^3
0 to 1	7.24
1 to 2	3.52
2 to 3	0.96
3 to 4	0.44

From a total of 426 measurements with a pachometer, the average depth of concrete cover over the reinforcing steel was 1.68 in. (4.3 cm), the standard deviation was 0.22 in. (0.56 cm), and the range was between 1.10 and 2.70 in. (2.79 and 6.86 cm), which indicates reasonable compliance with specifications.

Comparison of results of 1972 and 1973 electrical potential and concrete delamination survey are given below and indicate a change in physical and electrical conditions of the bridge in 1 year (14 months) of service. The percentage of concrete delamination is the percentage of the total deck surface that is spalled. The percentage of corrosion potential is that percentage of all measured potentials that are corrosive (and are potentials more negative than -0.35 CSE).

Year	Concrete Delamination (percent)	Corrosive Potentials (percent)
1972	2	53
1973	12	71

DECK PREPARATION

Figure 3 shows the equipotential contours for the bridge deck survey made in June 1973. Also shown are the locations of the undersurface fractures. As indicated by the extent of the fractures, some deck repairs had to be made prior to the application of cathodic protection. Otherwise, there was a possibility that the existing concrete spalls eventually may be loosened by traffic and thus cause structural failure of the cathodic protection overlay.

In an attempt to keep costs as low as possible, it was decided not to repair the deterioration by the process of concrete removal and replacement. Previous repairs of this kind had cost up to \$16/ft^2 (\$172/m^2) of repaired area. Instead, it was reasoned that if the concrete could be bonded together by injection of a suitable "glue," then structural loosening by traffic would be minimized. Also, if the cathodic protection system were successful, then continued corrosion-caused spalling would be stopped. Two materials were selected to use to bond the loose concrete to the underlying deck: a monomer, used recently in experimental concrete impregnation studies and an epoxy resin (32). The locations where each material was used are shown in Figure 3. At each spall, at least one ½-in. (1.3-cm) diameter hole was drilled into a central area. The debris from the hole was then removed by the use of an industrial vacuum cleaner. All injections of monomer and epoxy resin were made via these holes as shown in Figure 4.

By use of a grease gun, methyl methacrylate and styrene monomers were injected into the spalls. Later, 4-in. (10.2-cm) diameter cores were obtained to determine whether the concrete was truly bonded. Except for 1 core sample from the styrene-injected area, none of the monomer-injected concrete spalls was bonded. It was surmised that the methyl methacrylate was too thin and was absorbed by the concrete instead of filling the crack void. From visual observations, the styrene monomer appeared to have great curing shrinkage, which may have adversely affected bonding. However, the experimental monomer injection was performed with hand equipment. Better equipment and the selection of other monomers could have produced different results.

As shown in Figure 3, when a maximum pumping pressure of 160 lb/in.^{2} (1.1 MPa) was used, the epoxy injection was both successful and unsuccessful. The successful cases, where the cores showed the concrete was well bonded together, were found where the concrete emitted a hollow sound when the surface was struck with a hammer. The unsuccessful locations were found where the chain drag indicated a hollow sound but the hammer did not. In these latter locations, the epoxy could not be injected into the spalls at the pressures normally used. Previous but unreported work (as evidenced by concrete cores) by the author has shown that the chain drag will indicate delaminated concrete in locations where the hammer method will not.

Prior to and after the injection of the monomers and the epoxy, half-cell potentials were made at the specific locations of the concrete spalls. The apparent maximum reduction in the half-cell potential of the steel after injection was in the order of 0.05 V. Therefore, it was concluded that the injection of bonding materials would not significantly affect the penetration of cathodic protection currents to the surface of the reinforcing steel. It is surmised that, for the tested areas of this bridge, the concrete

Figure 1. Schematic of anode in concrete.

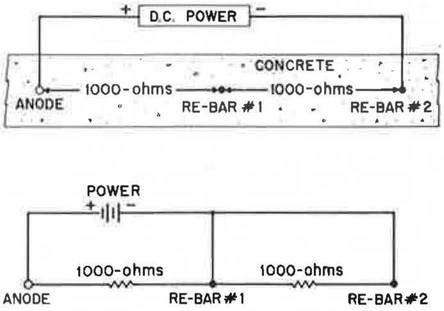


Figure 2. Schematic of anode in coke.

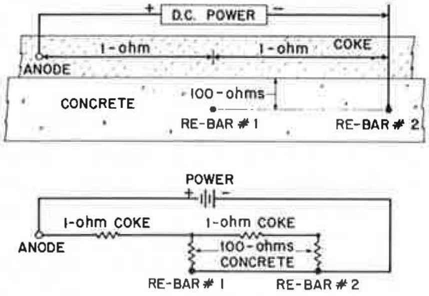


Figure 3. Original half-cell potentials and concrete spalls.

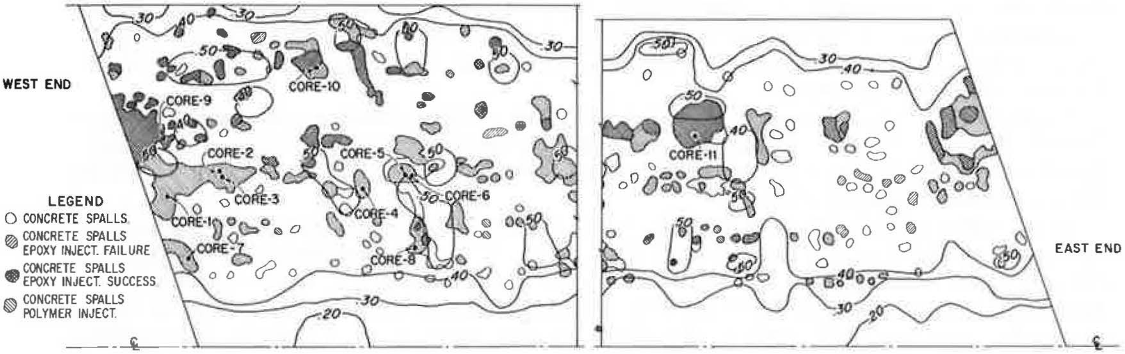
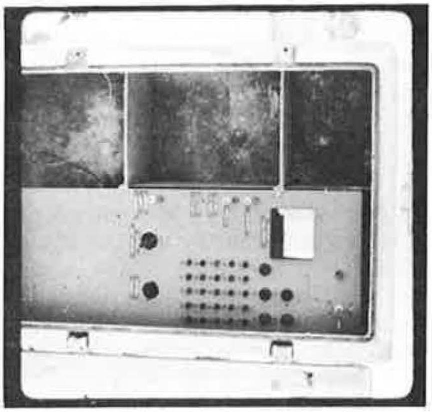


Figure 4. Injecting epoxy into undersurface structure.

Figure 5. Panel for controlling current to anodes.



within the undersurface fractures still has numerous points of contact. Therefore, filling of the void with dielectric epoxy or monomer does not create a continuous electrical shield between the reinforcing steel and the surface of the concrete. The cathodic protection currents to the steel should be effective in these areas of repair.

ELECTRICAL CONTINUITY

The use of cathodic protection depends on the electrical continuity of the structure being protected. If there are portions of the structure under cathodic protection that are not electrically connected to the system, they can be caused to corrode by stray currents at an accelerated rate (22, 29). Therefore, care must be exercised in determining whether the reinforcing steel in a bridge deck is electrically continuous.

Even with a detailed amount of testing, there is always the chance that one reinforcing bar out of the hundreds in a bridge deck may not be electrically continuous. In that case, damage will occur. If stray current damage occurs to 1 or 2 bars, the resultant concrete spalling and corrosion of the steel are expected to be no different from the condition that exists before cathodic protection is applied. However, when such a condition arises, repair can be made and the "loose" piece of steel welded to adjacent reinforcing steel. It then becomes a part of the protected grid. As a result, the corrosion can be stopped, which is not the case when conventional methods are used for repairing the damage.

In practice, it has been required that, at least at every third crossing of the reinforcing steel, a tie wire be used to mechanically interconnect the steel. Therefore, there is a strong likelihood that all reinforcing steel will be interconnected simply by normal construction procedures.

A previous report (28) showed that, if the half cell were left in the same location on a deck surface, the measured half-cell potentials would be different if an electrical contact were made to various electrically disconnected pieces of steel embedded in the concrete. Conversely, if the steel were interconnected, the half-cell potential relative to a stationary half cell would be the same, irrespective of the location of the connection to embedded steel. This assumes that the electrical resistance of the steel is minor compared to the electrical resistance of the concrete.

On the Sly Park Bridge, at 4 equidistant locations along the curb line and also on the concrete section of the railing, the electrical measurements showed that all reinforcing steel was interconnected. However, the bolted-on aluminum guardrail was not electrically connected to the reinforcing steel.

For the "ground" of the cathodic protection system, at all 4 deck locations at which the steel was used for continuity testing, No. 8 direct burial stranded copper wires were welded to the bars and brought out to the control panel.

CONTROL CABINET

As shown in Figure 5, a standard traffic controller cabinet was modified to house the electrical circuitry and the standard automotive type 6 and 12-V battery power sources.

Also installed on a panel inside the cabinet are 36 each of 3-Ω, 5-W wire-wound resistors. The purpose of these resistors is to control the amount of direct current to each of the anodes. This current control capability is necessary because of the expected variations in the electrical resistance of the portland cement concrete and coke-breeze asphalt concrete.

Included within the control panel are selector switches that allow the measurement of current flow by means of an 0.01-Ω shunt in series with each anode connection.

An ammeter is also installed on the panel to measure gross current flow. There are provisions for external equipment, such as a timer, that will automatically turn the current on and off so that polarization measurements can be obtained. The control panel without modification is to be used with an automatic potential control cathodic protection rectifier.

COKE-BREEZE ASPHALT CONCRETE

Insofar as coke breeze has been used as a backfill material for impressed current cathodic protection anodes, its feasibility as an asphalt concrete aggregate was evaluated. As received, coke breeze No. 90 was graded and found to meet January 1973 standard specifications of the California Division of Highways for aggregate grading of $^3/_8$-in. (0.95-cm) maximum aggregate for asphalt concrete.

Some of the physical properties determined were as follows: The specific gravity was 1.64 for the aggregate and 1.25 for the mix with 15 percent of 85 to 100 penetration asphalt (California test method 38); the K value was 3.0 for the coarse coke breeze and 1.7 for the fine aggregate (California test method 303); the surface area of the mix was 37.4 ft^2/lb (7.66 m^2/kg) (California test method 303); and the stabilometer value for a mixture of coke breeze and 15 percent of 85 to 100 penetration asphalt was approximately 28 (California test method 304).

Based on preliminary work, it was determined that, for optimum electrical properties, coke breeze should be 3 in. (7.6 cm) thick. However, the lower asphalt content mixtures tested were not sufficiently cohesive to be exposed directly to wheel loads. Therefore, it was decided to overlay the coke-asphalt concrete layer with about 2 in. (5 cm) of a dense-graded $^3/_8$-in. (0.95-cm) maximum natural aggregate asphalt concrete for a total overlay thickness of 5 in. (12.7 cm).

To determine the durability of composite pavement, a 50-ft-long by 12-ft-wide (15.2- by 3.66-m) test section was placed on a new but unused portland cement concrete pavement. After it was mixed at a plant, the coke-breeze asphalt concrete was spread by using a Layden box spreader. Initial rolling was done with a 4-ton (3628-kg) roller, and final rolling was done with a 10-ton (9071-kg) roller. The coke breeze was mixed with 15 percent of 85 to 100 penetration asphalt, and the $^3/_8$-in. (0.95-cm) maximum natural aggregate wearing course was mixed with 5.4 percent of the same asphalt.

At one end, a 12-ft (3.66-m) length of 5-in. (12.7-cm) thick all-natural-aggregate asphalt concrete was used as a control section. In the test section, the coke-breeze layer was covered with Petromat. However, after the natural aggregate concrete was placed, it was observed that the Petromat tended to wrinkle and its further use was questioned at that time. It was not used on the bridge overlay. A Dynaflect was used to measure deflections, and cross sections were taken at various stations.

To quickly test the load-carrying capacity of the composite pavement, a 10-wheel truck weighing about 44,000 lb (20 000 kg) complying with the legal load limit of 18,000 lb (8165 kg) per axle was used to apply loads to the test section. The results were that, at the end of 3,802 passes of the truck, no distress was observed or measured.

On the basis of this test series, it was decided that the composite asphalt concrete pavement would be reasonably durable when used on a bridge deck.

INSTALLATION OF ANODES AND OVERLAY

The iron-alloy anodes were disc shaped, 10 in. (25.4 cm) in diameter and 1$^1/_4$ in. (3.2 cm) thick, and had an average weight of approximately 29 lb (13.2 kg). Based on test data (31) for similar anodes, they had a consumption rate of about $^1/_4$ lb (0.1 kg) per ampere year of current flow. In other words, if 1 A was caused to be continuously discharged by the anode, it would be entirely consumed by corrosion in about 116 years.

Initially 3 rows of anodes were laid out on the bridge deck 12 ft (3.66 m) center-to-center. The anodes in effect were placed on 12-ft (3.66-m) centers directly beneath the 3 traffic strips that delineate the 2 lanes across the bridge. After locations of the anodes were marked out on the pavement, a fast setting epoxy adhesive (California standard specification 721-80-42) was placed on the concrete surface and the anode was then placed on the epoxy. Epoxy was used to hold the anodes in place during the paving operation and to prevent current discharge from the bottom surface of the anode. Limiting the current discharge from the bottom of the anode would inhibit the lifting of the anode, which could cause pavement distress due to the formation of a layer of rust between the anode and the concrete pavement. Also, the epoxy layer would reduce the current discharge directly beneath the anode, which could cause a high current density flow to the reinforcing steel directly under it and, thus, result in a "hot spot."

At the conclusion of the paving operation, 1 out of 36 anode connections was damaged. The damage was likely caused by the roller passing close to the point where the lead wire leaves the anode, thereby pulling it loose.

Before the bridge deck was paved, an SS-1 asphalt emulsion tack coat was applied at a rate of 0.05 gal/yd^2 (0.23 liter/m^2). Previous electrical testing on the pavement test section showed that use of the tack coat at this rate of coverage would not adversely affect the electrical performance of the cathodic protection system.

The coke breeze was initially dried at the batch plant to a temperature of about 230 F (110 C) to which the 85 to 100 penetration grade asphalt at 310 F (154 C) was added. Final temperature of 21 tons (19 050 kg) of coke-breeze asphalt concrete at the batch plant ranged between 240 and 270 F (116 and 132 C). The haul distance from the batch plant to the bridge was approximately 55 miles (89 km). A Blaw-Knox rubber-tired paving machine was used to lay all coke-breeze asphalt concrete in about 10-ft (3-m) widths. An area approximately 1,500 ft^2 (139.4 m^2) was paved with an all-natural-aggregate asphalt concrete. The 5 anodes in this area cannot operate as intended and are, therefore, not included in the protection system. As will be discussed later, only 7 of the remaining anodes were needed to provide the desired protection.

The coke-breeze asphalt concrete layer was consolidated initially with a 4-ton (3628-kg) roller and finally with a 12-ton (10 886-kg) roller. Initially, there was some "shoving" of this mixture because of its lack of cohesion. Further studies are being made to improve the cohesion of the coke-breeze asphalt concrete by using a heavier grade of asphalt or a higher asphalt content or both.

The natural-aggregate asphalt mix for the surface or wearing course arrived at the job site at a temperature of 270 F (132 C), and its placement in 1-in. (2.54-cm) lifts and rolling to final grade were performed without incident.

Thus far (2 months of service) there is no evidence of distress on the pavement due to traffic, which includes up to maximum legal load limits of commercial and logging truck traffic. However, the pavement has not yet been subjected to inclement weather, such as rain or snow, or to chain traffic.

Figure 6 shows the anodes in place, and the paving operation in progress. Figures 7 and 8 show the actual depths of the loose or uncompacted coke-breeze asphalt concrete as well as the area that contains full-depth (5-in., 12-cm) natural-aggregate asphalt concrete. As shown in Figure 8, the thickness of various areas of the uncompacted depth of coke breeze is 3½, 3, and 2½ in. (8.9, 7.6, and 6.4 cm). The varying depth of coke-breeze asphalt concrete was used to explore the feasibility of reducing the total depth of the composite asphalt concrete.

The Sly Park Bridge does not have expansion joints; therefore, no consideration was given to the use of expansion dams.

CIRCUIT RESISTANCE

After the installation was completed, but before any current was applied, electrical measurements were made on the deck at various intervals of time. It was observed that for about 1 week after construction the half-cell potentials of the steel would not reasonably duplicate those values that were originally measured on the concrete surface.

It was speculated that, when the hot (270 F, 132 C) asphalt concrete was placed on the deck surface, free water was driven out of the portland cement concrete. In the dry and hot climate typical at the time of construction (air temperatures about 95 F, 35 C), a few more than 7 days were required for the moisture level to increase enough to make the upper surface of the concrete electrically conductive.

By use of a commutated direct current ohm meter, the average electrical resistance was measured between the anodes and the reinforcing steel. The average values of electrical resistance for the various uncompacted thicknesses of coke-breeze asphalt concrete are given below. Compacted thickness is probably about ½ in. (1.3 cm) less than that given. Also given are the average electrical resistance values when cathodic protection was being applied at a current of 1.01 A and a driving voltage of 1.65.

Uncompacted Depth (in.)	Commutated dc Resistance (Ω)	After Polarization (Ω)
2½	1.43	14.7
3	1.16	12.1
3½	1.09	9.3

There is a significant difference in electrical resistance for the different uncompacted depths of the coke-breeze asphalt concrete. The effect of polarization during the flow of cathodic protection currents is also shown as an electrical resistance for the different depths of coke asphalt concrete. The measurements were made on the compacted composite pavement, and the term "uncompacted depth" applies to the depth of the asphalt concrete before compaction or consolidation.

During the process of cathodic protection, the polarization of the anode and cathode results in a back electromotive force, EMF (22, 29). In the case of the Sly Park Bridge, when the anodes were disconnected, the polarization (or back EMF) that was measured between the anodes and reinforcing steel was an average of 1.44 V.

The locations of the 7 operating anodes outside the traveled lanes are shown on Figures 7 and 8. The only reason that not all of the anodes are being used is that during the preliminary testing it was possible to sustain the cathodic protection system without using all of the installed anodes. The locations of the anodes being used have a mechanical and economical advantage over the use of anodes that were installed in the traveled lanes. These anodes would be subject to greater traffic loading as compared to those in the shoulder and median areas of the bridge.

DISTRIBUTION OF CATHODIC PROTECTION CURRENTS

It was initially planned that for equal current distribution throughout the bridge deck surface all of the originally installed anodes might be used. However, initially, 4 adjacent anodes that were located at the shoulder side of the west end of the bridge were turned on with a total current flow of 3.6 A. The cathodic protection currents could polarize the reinforcing steel to a protective potential for a longitudinal distance of 65 ft (19.8 m) from the nearest anode. Also, the steel began to polarize quite rapidly so that after 2 hours the output current from the 4 anodes was reduced to 2.0 A. Three days later, the current flow to the 4 anodes was further reduced to 1.6 A. Seven days later, the 4 anodes at one end of the bridge were deactivated and the 7 anodes (as shown in Figs. 7 and 8) were activated with a total current output of 1.08 A.

The performance of the 7 anodes after 19 days of operation is given below. The anodes that are numbered 1-1 through 1-9 are near the shoulder area; 1-1 is at the most westerly end of the bridge. The anodes numbered 3-1 and 3-4 are nearest the centerline or median area of the twin-bridge installation.

Anode	Ampere	Driving Voltage	Back EMF
1-1	0.12	1.65	1.48
1-3	0.12	1.65	1.52
1-5	0.21	1.65	1.42
1-7	0.20	1.60	1.38
1-9	0.15	1.60	1.38
3-1	0.10	1.72	1.42
3-4	0.15	1.82	1.48

Figure 7 shows the voltage gradients relative to the CSE when the cathodic protection currents are on. Even though the current is on, the measurements of the half-cell potential of the reinforcing steel beneath the 5-in. (12.7-cm) thick nonconductive natural-aggregate asphalt concrete are unaffected and near the same values as originally measured and shown in Figure 3. This shows that the steel in this area is not

Figure 6. Anodes on concrete surface.

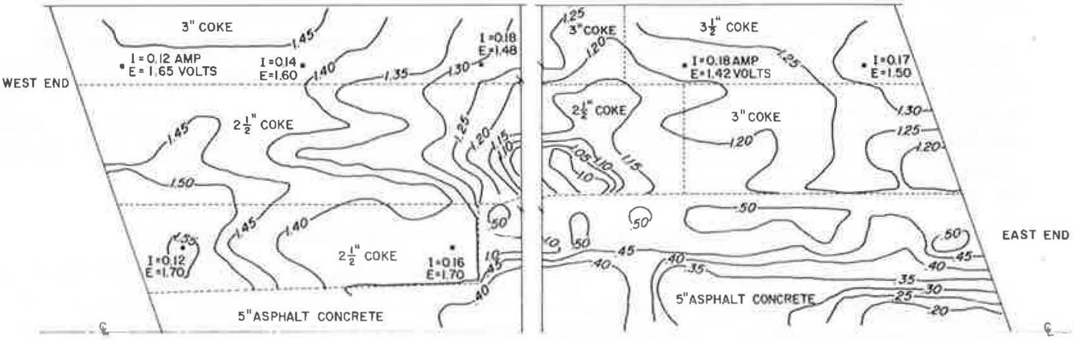


Figure 7. Cathode protection current on.

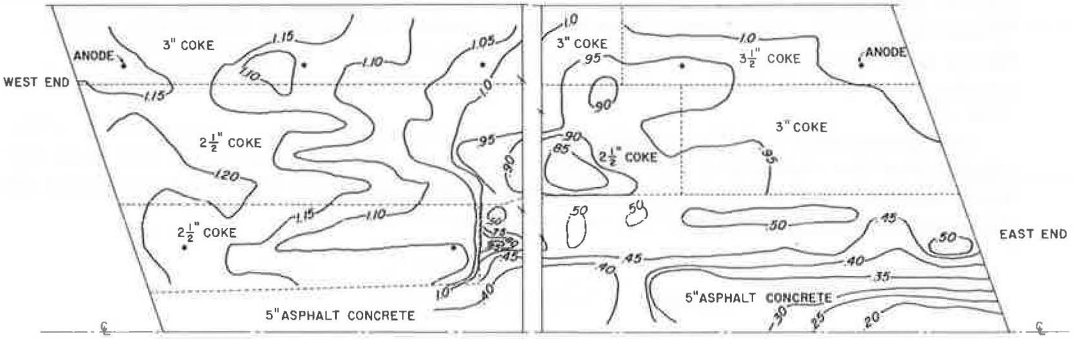


Figure 8. Current off polarized potentials.

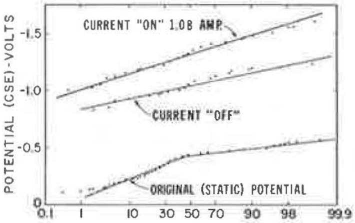


Figure 9. Distribution of half-cell potential.

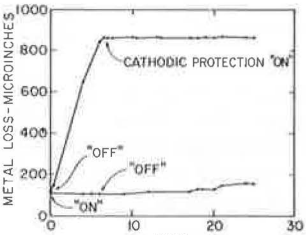


Figure 10. Effect of cathodic protection.

affected by the other parts of the system, and the basic theory of deck protection is confirmed (i.e., no current will flow through the nonconductive layer).

Figure 8 shows the current in the off condition, or the distribution of the polarized half-cell potentials of the steel. The maximum range of the difference of polarized potentials of the steel is 0.35 V. Also, as indicated by the potentials being more negative than -0.85 V, the cathodic protection currents should be effectively controlling the corrosion of the steel.

Even though the activated anodes shown in Figures 7 and 8 are at a 24-ft (7.3-m) center-to-center spacing, the inactive anodes could be energized and result in a more even distribution of potentials. However, as previously pointed out, it is desirable to have the cathodic protection system operate with anodes not placed in the traveled lanes of the pavement. The inactive anodes will be placed under cathodic protection so that, when or if the operating ones are consumed or become inoperative, the inactive ones will be available and no new anodes will have to be installed.

The "storage" of in-place inactive anodes by cathodic protection that are installed at the same time as the active anodes may be of considerable value on structures of high vehicular density where maintenance operations are of critical concern.

Measurements were made to determine whether the cathodic protection currents were affecting the bottom mat of reinforcing steel. The half-cell potential of this steel changes only a few millivolts when the cathodic protection current is turned on and off. The bottom mat is not significantly affected by the cathodic protection currents, so any corrosion of this steel will not be controlled by the system.

Because the 2 mats of steel are interconnected by the "crank" or truss bars, a calculation of the cathodic protection current density to the surface of the steel can only be an estimate. Therefore, for this particular structure, the existing current density used to obtain cathodic protection is estimated to be about 0.7 mA/ft^2 (7.5 mA/m^2) for the top mat of steel.

As an indication of the distribution of the half-cell potentials of the reinforcing steel, Figure 9 shows the original current on and the current off polarized potentials.

Figure 9 shows that 98 percent of all of the polarized potentials are greater than -0.85 V; therefore, about 2 percent of the area of the steel within the conductive asphalt concrete may not have adequate cathodic protection. The potentials at those locations of less than -0.85 V CSE can be easily changed so that the current output of the anodes is increased.

Although the conditions reported were observed during battery operation, a rectifier has been installed that will automatically control the cathodic protection current according to the half-cell potential of the steel. As a result, a permanent half-cell is being placed on the deck that will provide the means for the rectifier to "sense" the half-cell potential of the steel and make automatic adjustments of the current. Automatic operation will supply the proper level of cathodic protection regardless of other varying conditions.

EFFECTIVENESS OF CATHODIC PROTECTION

Literature references are cited wherein effectiveness of a cathodic protection system can be related to the polarized half-cell potential of the steel. However, since this might be the first demonstration of cathodic protection on a bridge deck and involves some unique features as compared to, say, pipelines or tanks, effectiveness of the system should be demonstrated, preferably by a short-time test. A test of sorts was devised to measure the effect of the system on corrosion of steel strips embedded in 3- by 3- by 12-in. concrete blocks containing 10 percent calcium chloride by weight of cement. By external means, the steel strips were measured to determine their electrical resistance. Any corrosion of the steel strips will result in a change in their cross section, and thus there will be an associated change in electrical resistance of the steel. This technique has been widely reported (33) and used.

The concrete blocks and embedded steel strips described above were placed in the conductive coke asphalt concrete as shown in Figure 10. Steel strip 1 was allowed to corrode for 6½ days before it was connected to the cathodic protection system. Corrosion was essentially stopped after the application of cathodic protection current. The polarized potential of the strip was measured and found to be -1.31 V CSE. Steel strip 2, which also was embedded in the same kind of concrete, was placed in the electrically conductive coke asphalt concrete and immediately connected to the cathodic protection system. Essentially no corrosion occurred. After 6 days, the strip was disconnected from the cathodic protection system and corrosion began. However, as shown by the performance of steel strip 2, the loss of cathodic protection does not result in an immediate and catastrophic corrosion rate because of the apparently long-term ''decay'' of polarization. The half-cell potential of steel strip 2 was -0.67 V CSE on the twentieth day of test.

As indicated by the corrosion measurements of the steel strips with and without cathodic protection applied, the system is feasible and does control corrosion of embedded steel. However, the long-time durability and performance of the paving system and the anodes themselves have not yet been confirmed for this type of application.

There is still the possibility that there will be pavement failure as the result of untouched and loose concrete spalls in the deck. However, such failures would be considered not a failure of the cathodic protection system but an indicator of necessary deck preparation prior to the placement of the overlay.

COST OF CATHODIC PROTECTION

Although an experimental installation provides a poor criterion of costs, it is at this time the only available indicator. Therefore, the cost of the cathodic protection installation is to be regarded as an estimate and could vary considerably from that given below:

Item	Estimate
Paving (including cost of coke breeze)	$ 8,867.57
Epoxy injection (repairs of deck)	1,507.50
Anodes	1,500.00
Installation of anodes	75.00
Wiring (ac power)	1,600.00
Rectifier	900.00
Control panel	1,500.00
	$15,950.07

Based on the total square feet of deck area, the cost for the cathodic protection system was about $\$3/\mathrm{ft}^2$ ($\$33/\mathrm{m}^2$) of deck area. This figure does not include the cost of the original bridge survey or the testing that was performed subsequent to the installation of the cathodic protection system.

SUMMARY AND CONCLUSIONS

Cathodic protection has had a long history of successful use in protecting concrete-embedded steel in pipelines. The results of these experimental installations demonstrate that, if certain conditions are met, cathodic protection can be applied to a bridge deck. One condition necessary for successful uniform application of electrical current to embedded steel is a conductive layer of relatively low electrical resistance that can be spread over the area to be protected. Although there are other materials that can be used as an electrically conductive overlay, coke breeze was found to provide the necessary properties. When mixed with a relatively low amount of asphalt binder, the coke-breeze asphalt mixture was stable enough to function also as a base for a regular asphaltic concrete wearing course.

Prior to the installation of the cathodic protection system, the bridge deck was surveyed for half-cell potentials and concrete delaminations by means of the chain drag. For the 1-year period between 1972 and 1973 prior to this work, the undersurface concrete fractures increased from 2 to 12 percent of the total deck area and the percentage of corrosive potentials increased from 53 to 71 percent of the total measurements. The average chloride-ion content of the concrete 1 year before the installation of the cathodic protection system was 3.52 lb/yd^3 (2.08 kg/m^3) at the level of the reinforcing steel.

Because past experience has indicated that concrete removal and replacement at delaminations have cost as much as $16/$ft^2$ (172/m^2) of repaired area, an experiment for bonding rather than removing the concrete was performed. A methyl methacrylate, a styrene monomer, and also an epoxy were injected into the undersurface fractures. From cores, the results indicated the epoxy injection was by far the best bonding agent. However, since this was the first test with monomer injection of this kind, the results of the use of these materials are not considered conclusive.

The effectiveness of the cathodic protection was demonstrated by arresting corrosion of steel strips that were embedded in concrete bars containing 10 percent calcium chloride by weight of the cement. The bars were placed within the coke-breeze asphalt layer on the bridge deck. Corrosion losses of the steel were measurable as increases in electrical resistance.

From the test results, a current density of about 0.7 mA/ft^2 (6.5 mA/m^2) of reinforcing steel surface (upper bar mat) may control corrosion in a salt-laden concrete bridge deck. Measurements recorded on the experimental deck cathodic protection system show that the corrosion is apparently controlled on about 3,311 ft^2 (307.6 m^2) with a driving voltage of about 1.65 and about 1 A of current for a total power consumption of about 1.65 W. As a result, power, per se, is not considered to be a limiting factor in the cathodic protection system.

Although the optimum spacing of the impressed current anodes was not clearly determined by this experiment, their effectiveness can exceed a 12-ft (3.7-m) radius. However, the maximum polarization potential of the steel should be limited to a maximum of about -1.10 V CSE to prevent any possible loss of bond of the steel to the concrete.

ACKNOWLEDGMENT

This project was performed in cooperation with the Federal Highway Administration. The contents of this report reflect the views of the author, who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the state of California or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation. The author wishes to acknowledge the contributions of G. H. C. Chang, P. J. Jurach, E. Maggenti, and T. Scrimsher, Transportation Laboratory, California Division of Highways; R. E. Hay, Federal Highway Administration; J. Fontana, Brookhaven National Laboratory; and D. R. Higgins and G. A. Hood, Office of Structures, California Department of Transportation. The contribution of some materials by the Farwest Corrosion Control Company is also gratefully acknowledged.

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TIME TO CORROSION OF REINFORCING STEEL IN CONCRETE SLABS

K. C. Clear, Federal Highway Administration

The purpose of this investigation is to provide administrators and designers with factual data on which to base decisions as to the type of protection to provide for bridge decks constructed in corrosive environments. The specific study objective is to determine the relative time to corrosion of reinforcing steel embedded in concrete slabs that are fabricated from various mix designs and construction procedures and subject to periodic wetting with a 3 percent sodium chloride solution. Constructed and tested were 124 reinforced concrete slabs 4 by 5 by 0.5 ft. Data obtained from tests of the effect of portland cement concrete mix design, concrete cover over the reinforcing steel, and consolidation of the fresh plastic concrete are discussed in this paper. A brief description of slab fabrication and testing procedures is also presented.

• ONE OF THE most severe problems facing the highway industry is reinforcing steel corrosion induced by chloride de-icer and the subsequent deterioration of concrete bridge decks. The Offices of Research and Development in the Federal Highway Administration consider elimination of bridge deck deterioration one of their highest priority efforts. The problem and approach to the solution of the problem are defined in Task 4B1 of the Federally Coordinated Program of Transportation Research. One work unit within that task is the FHWA staff research study, "Time-to-Corrosion of Reinforcing Steel in Concrete Slabs Versus Various Parameters of Design and Construction." The purpose of that work unit is to provide administrators and designers with factual data on which to base decisions as to the type of protection to provide for bridge decks constructed in corrosive environments. The specific study objective is to determine the relative time to corrosion of reinforcing steel embedded in concrete slabs fabricated from various mix designs and construction procedures and subject to periodic wetting with a 3 percent sodium chloride solution.

Interim findings have been published in 2 interim reports (1, 2). The first report documents the effect of mix designs, construction techniques, and special treatments on chloride migration and reinforcing steel corrosion in 124 reinforced concrete slabs 4 by 5 by 0.5 ft (1.22 by 1.52 by 0.15 m). Although the data obtained indicate the relative resistivity of the various concretes to de-icer penetration, the relative number of salt applications in this test is not considered to be suitable for estimating bridge deck service life or the time to corrosion of reinforcing steel in a concrete bridge deck subjected to a specific de-icer exposure. An evaluation of the corrosion detection device is also presented. The second report presents data on the half-cell potential versus time (with increased de-icer applications) in the form of plots for each of the 124 reinforced concrete slabs.

Data obtained from the portion of the test slabs used to document the effect of portland cement concrete mix design, concrete cover over the reinforcing steel, and consolidation of the fresh plastic concrete are discussed in this paper. A brief description of slab fabrication and testing procedures is also presented.

Publication of this paper sponsored by Committee on Corrosion.

FABRICATION AND TESTING

The test slabs in the outdoor exposure yard are shown in Figure 1. The 20-ft^2 (1.86-m^2) slabs were molded in watertight molds, and a grid of No. 4 reinforcing bars was positioned at the required depth. The concrete was vibrated to 99 ± 1 percent of the rodded unit weight; a direct transmission nuclear density apparatus was used to control consolidation. The nuclear apparatus was a commercially available moisture-density gauge of the type commonly used to monitor soil compaction. Screeding was accomplished by a reproducible manual finishing process. The appropriate curing procedure was applied immediately after loss of surface sheen and remained on the concrete for 7 days.

After slabs were removed from the molds, the sides were coated with epoxy resin to prevent water loss during testing. A small dike was placed around each slab to permit ponding of the sodium chloride solution. After 7 days of on-ground curing, the slabs were placed and leveled in the outdoor exposure yard. The surfaces were wire-brushed to remove the membrane curing compound before the initial sodium chloride application (at 6 weeks of age).

The top surface of each slab was subjected to ponding to a depth of 1/16 in. (1.6 mm) in a 3 percent sodium chloride solution each afternoon. Three evaluation techniques are used to determine the ability of each variable in preventing chloride migration to the level of the reinforcing steel and its subsequent corrosion.

1. The electrical half-cell potential of the reinforcing steel is monitored by weekly measurement of its potentials at 6 predetermined positions on the surface of each slab. Analysis yielded the following meaning of the potentials (referenced to the copper-copper sulfate half-cell, CSE): (a) potentials consistently greater than 0.35 V CSE—high probability of corrosion; (b) potentials consistently less than 0.20 V CSE—high probability of no corrosion; and (c) potentials in the range of 0.20 to 0.35 V CSE—uncertain area with regard to the condition of the reinforcing steel, i.e., may be active or passive, and use of another detection technique is indicated.
2. The chloride content at the level of the reinforcing steel is determined at select times by dry-coring and analysis according to the procedure described by Berman (3). Work by Lewis and studies in the FHWA laboratories have shown the chloride content corrosion threshold (i.e., the minimum quantity of chloride required to initiate reinforcing steel corrosion in a bridge deck of portland cement concrete when sufficient moisture, oxygen, and other necessary factors are present) to be approximately 0.20 percent of Cl^- per gram of cement or 330 parts per million of chloride on a concrete basis, i.e., 1.3 lb of chloride ion per 1 yd^3 of concrete (3, 4). All chloride contents presented in this paper are in parts per million, ppm, chloride ion (by weight) in the concrete. For conversion to pounds of Cl^- per cubic yard of concrete, multiply Cl^- ppm by 0.003915.
3. Visual and delamination surveys are made to locate cracking, rust stains, and hollow planes. To date no cracking or delamination is present although surface rust stains are present on many slabs.

MIX DESIGN PARAMETERS

Water-Cement Ratio—The effect of water-cement ratio on water permeability of concrete is widely documented. Since a similar effect has been suggested with respect to chloride migration, water-cement ratios spanning the feasible range were included. Specifically, water-cement ratios of 0.4, 0.5, and 0.6 by weight (i.e., 4.5, 5.6, and 6.8 gal/bag; 17, 21, and 26 litres/bag) were studied by using mix designs with a constant cement content of 658 lb/yd^3 (390 kg/m^3).

Cement Content—Cement contents of 564, 658, and 752 lb/yd^3 (335, 390, and 446 kg/m^3) were included to ascertain the effect of changes in quantity of portland cement. Water-cement ratio was constant at 0.5.

Aggregate Proportions—The ratio of coarse aggregate to fine aggregate may have an effect on the movement of chloride within the concrete. The standard mix design for concrete with a water-cement ratio of 0.5 and cement content of 658 lb/yd^3 (390 kg/m^3) used a sand-stone ratio of 0.822 by volume (i.e., 55 percent coarse aggregate and 45 percent fine aggregate). In selected instances, the sand-stone ratio was changed to 0.429 (i.e., 70 percent coarse aggregate and 30 percent fine aggregate) with the other major variables remaining constant, i.e., water-cement ratio = 0.5, and cement content = 658 lb/yd^3 (390 kg/m^3).

The mix designs are given in Table 1, and the consistency of each mix is illustrated by the slump tests shown in Figure 2.

Cover Over Reinforcing Bars

Numerous bridge deck surveys have linked the incidence of spalling with the depth of concrete cover over the reinforcing steel. Therefore, the depth of the reinforcing mat was included as a variable. Concrete covers used were 1, 2, and 3 in. (25, 51, and 76 mm).

Consolidation

The in-place density was believed to be a prime determinant of concrete permeability. Therefore, the normal placement procedure included vibration to 99 ± 1 percent of the rodded unit weight. However, several slabs were not properly consolidated during fabrication; therefore, we were able to determine the effect of improper consolidation on the chloride permeability of the concrete.

The majority of the 20-ft^2 (1.85-m^2) reinforced concrete slabs have received more than 300 daily salt applications. Approximately 35,000 electrical potential measurements indicative of the reinforcing steel condition in the 124 slabs have been obtained. In addition, 203 concrete samples at the reinforcing steel level have been subjected to chloride content determination. Severe surface rust stains are present on a portion of the slabs, and severe corrosion of the reinforcing steel is apparent in some instances. Although no delamination has yet occurred, the above data have been used to formulate the interim conclusions discussed below.

EFFECT OF CONCRETE MIX DESIGN ON CHLORIDE-INDUCED CORROSION

As documented earlier, a portion of the 20-ft^2 (1.85-m^2) slabs was used to determine the effect of water-cement ratio, cement content, and aggregate proportions on chloride-induced corrosion of the reinforcing steel. Figure 3 shows the effect of concrete mix design on chloride content at the level of the reinforcing steel (1 in., 25.4 mm) after approximately 330 daily salt applications. The data are given in Table 2.

The water-cement ratio of the concrete appears to be the primary determinant of the ability of portland cement concrete to resist chloride intrusion. The maximum chloride content at a 1-in. (25.4-mm) depth within the concrete with a water-cement ratio of 0.40 after 330 salt applications with a 3 percent NaCl solution was 437 ppm chloride on a concrete basis. Conversely, for water-cement ratios of 0.5 and 0.6, the maximum chloride contents were 1,555 and 1,701 ppm chloride respectively.

The resistance of the air-entrained portland cement concrete with water-cement ratios of 0.4, 0.5, and 0.6 to 100 cycles of freezing and thawing was studied by using 14- by 10- by 3-in. (356- by 254- by 76-mm) specimens tested in accordance with ASTM C 672-72T. All 3 concretes were resistant to scaling, exhibiting a maximum rating of 3 on a 0 to 10 visual rating scale (5). That rating signifies light scale over one-half of the surface. No differences in scaling with variation in water-cement ratio were evident for the air-entrained concretes.

Variation in the cement content of the concrete (at a constant water-cement ratio of 0.5) had little effect on chloride migration. As shown in Figure 3, for cement contents of 564, 658, and 752 lb/yd^3 (335, 390, and 446 kg/m^3), average chloride contents of 904, 912, and 1,147 ppm at a 1-in. (25.4-mm) depth were found after approximately 330 salt applications. Data overlap was large and statistical analysis (assuming normality) showed that, at the 95 percent confidence level, cement contents in the range of 564 to 752 lb/yd^3 (335 to 446 kg/m^3) did not affect chloride intrusion when the water-cement ratio was constant at 0.5.

Figure 1. Test slabs.

Table 1. Mix designs.

Mix	Water-Cement Ratio	Cement Content (lb/yd^3)	Sand-Stone Ratio	Slump (in.)
1	0.5	564		2.1
2	0.5	658	0.822	3.3
3	0.5	752		6.9
4	0.4	658		1.0
5	0.6	658		8.1
6	0.5	658	0.429	4.4

Note: 1 lb = 0.45 kg; 1 yd^3 = 0.76 m^3; and 1 in. = 25.4 mm.

Figure 2. Slump tests.

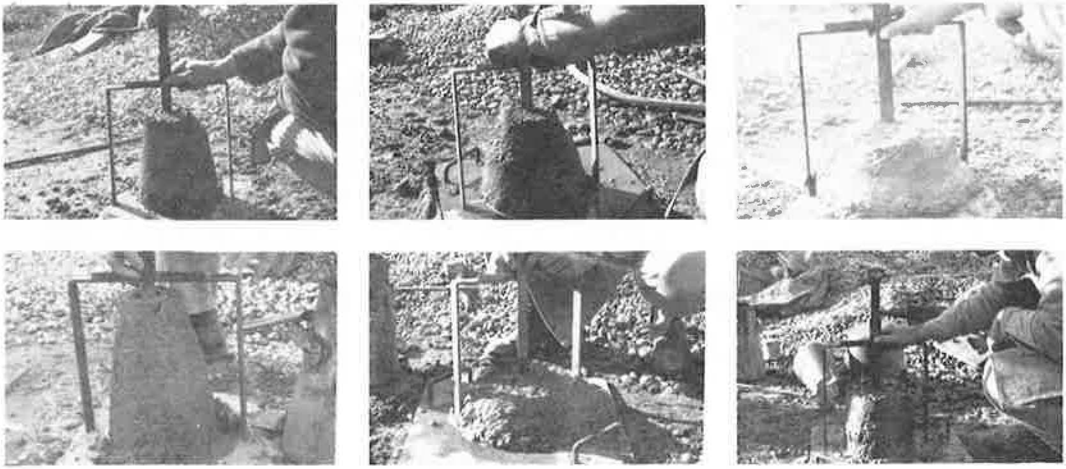


Figure 3. Effect of mix design parameters.

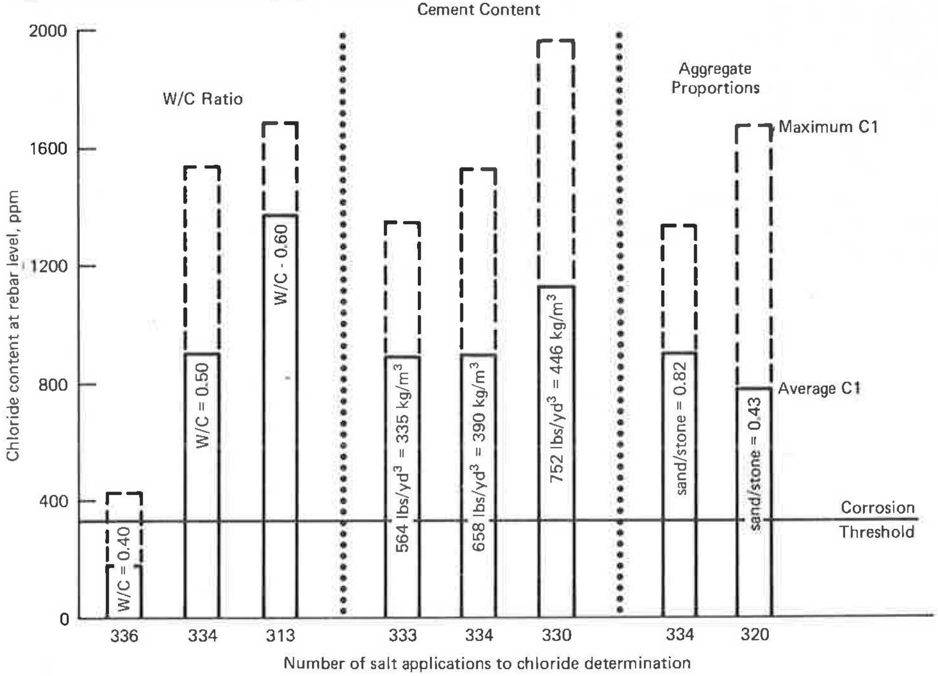


Table 2. Effect of mix design and cover depth on chloride migration.

Description	Number of Slabs	Number of Cores	Cover Depth[a] (in.)	Chloride Content at Level of Reinforcing Steel (ppm of Cl^- on concrete basis)		Number of Salt Applications
				Average	Range	
Water-cement ratio[b]						
0.4	2	10	1.0	179	15 to 437	336
	2	10	2.0	18	13 to 28	337
	2	10	3.0	18	13 to 23	337
	1	3	0.25	2,355	2,289 to 2,450	359
0.5	2	10	1.0	912	386 to 1,555	334
	2	10	2.0	250	23 to 562	348
	2	10	3.0	20	10 to 77	344
	1	3	0.25	3,249	2,437 to 4,332	341
0.6	1	3	1.0	1,407	960 to 1,701	313
	1	3	2.0	1,093	679 to 1,581	354
	2	6	3.0	189	105 to 255	316
	1	3	0.25	3,757	2,965 to 4,666	313
Cement content[c], lb/yd^3						
563	2	6	1.0	904	235 to 1,369	333
658	2	10	1.0	912	386 to 1,555	334
752	2	6	1.0	1,147	444 to 2,010	330
Sand-stone ratio[d]						
0.822	2	10	1.0	912	386 to 1,555	334
0.429	2	6	1.0	792	309 to 1,693	320

Note: 1 lb = 0.45 kg; 1 yd^3 = 0.76 m^3; and 1 in. = 25.4 mm.

[a] Nominal, for example, for a 1.0-in. (25.4-mm) core depth; the actual concrete analyzed was from a ¾- to 1¼-in. (19- to 32-mm) depth.

[b] Cement content constant.

[c] Water-cement ratio = 0.5.

[d] Water-cement ratio = 0.5, and cement content = 658 lb/yd^3.

Table 3. Time to corrosion of slabs exhibiting potentials consistently greater than 0.35 V CSE.

Slab Number[a]	Special Treatment	Time to Corrosion (week)[b]
72-7		1
126-126		1
127-127		1
116-118	2-in. (50.8-mm) reinforcing steel spacing	2
117-119	2-in. (50.8-mm) reinforcing steel spacing	1
78-19	Burlap cure	2
62-22	Poly-sheet cure for 3 days	2
51-24	Poly-sheet cure for 7 days	3
50-69	Permanent metal forms	2
63-70	Permanent metal forms	2
32-73	No rain (cover over slab)	2
106-74	No rain (cover over slab)	1
34-77	Chromate inhibitor	4
71-78	Chromate inhibitor	2
55-89	Monthly NaOH wash	4

[a] Concrete for all slabs listed had a water-cement ratio of 0.5, cement content of 658 lb/yd^3 (390 kg/m^3), and a cover over the reinforcing steel of 1 in. (25.4 mm).

[b] One week is equivalent to 7 daily applications of a 3 percent sodium chloride solution ponded to a $^1/_{16}$-in. (1.6-mm) depth.

Similarly, the sand-stone proportions (water-cement ratio = 0.5) had little effect on the chloride contents at the 1-in. (25.4-mm) level within the concrete. Both the average chloride content and the range were virtually identical for concrete with sand-stone ratios of 0.82 and 0.43.

TIME TO CORROSION OF TYPICAL BRIDGE DECK CONCRETE

The portland cement concrete with a water-cement ratio of 0.5, cement content of 658 lb/yd^3 (390 kg/m^3), and 45 percent sand in the total aggregate is typical of much bridge deck concrete. The concrete exhibited a slump of 3 ± 0.5 in. (76 ± 13 mm) and air content of 5 to 7 percent and was placed and cured by using strict quality control. In-place consolidation was controlled by vibrating to 99 ± 1 percent of the rodded unit weight. Standard field construction procedures were used, and the reinforcing steel was positioned to obtain 1 in. (25.4 mm) of clear concrete cover. Electrical potentials were used to determine the number of daily applications of a 3 percent sodium chloride solution required to induce reinforcing steel corrosion.

The time-to-corrosion tests on this bridge deck concrete yielded the most striking (and disturbing) finding of the study. The summary given in Table 3 shows that the time to corrosion for the typical uncracked bridge deck concrete with 1-in. (25.4-mm) reinforcing steel cover was only 1 week (7 applications of a 3 percent NaCl solution) in many instances. The test procedure corresponds to a daily NaCl application rate of only 0.2 lb (0.09 kg) NaCl per 20 ft^2 (1.85 m^2) of concrete surface. Therefore, the total salt applied prior to initiation of corrosion was 1.4 lb (0.64 kg) over the entire 20-ft^2 (1.85-m^2) slab. The maximum time to corrosion for this concrete (in the instances where electrical potentials permitted determination of a valid time to corrosion) was 4 weeks (28 de-icer applications). The total quantity of NaCl applied in this instance was less than 0.3 lb/ft^2 (1.5 kg/m^2) of concrete surface.

This finding shows that uncracked quality concretes, similar to those used in field construction, provide little protection against reinforcing steel corrosion when the reinforcing steel is placed at a 1-in. (25.4-mm) depth. It substantiates the necessity of modifying present bridge deck concrete mix designs and the need to use alternate means of preventing de-icer intrusion. This finding should also lay to rest the misconception that cracks are necessary for de-icer-born chlorides to get into the concrete.

EFFECT OF CLEAR CONCRETE COVER OVER THE REINFORCEMENT

The tremendous adverse effect of failure to provide sufficient concrete cover over the reinforcing steel is suggested in the discussion presented above. Figures 4 and 5 show the effect of the reinforcing steel cover on chloride migration within quality uncracked portland cement concrete with water-cement ratios of 0.4, 0.5, and 0.6.

After 330 daily NaCl applications, extremely large chloride contents were encountered near the concrete surface. For example, for concrete with a water-cement ratio of 0.5, the maximum chloride content found at a ¼-in. (6.4-mm) depth was 13 times that required to induce corrosion of the reinforcing steel. With depth, the chloride contents decreased rapidly to negligible values for the concrete with water-cement ratios of 0.4 and 0.5. The importance of water-cement ratio is also demonstrated in this instance. Figure 5 shows that for the uncracked portland cement concrete with a water-cement ratio of 0.4, 2 in. (50.8 mm) of clear concrete cover over the reinforcing steel are required to ensure against chloride-induced corrosion for at least the field equivalency of 330 salt applications in this test. For the typical bridge deck concrete with a water-cement ratio of 0.5, the cover requirement is increased to a minimum of 3 in. (76.2 mm); and for the concrete with a water-cement ratio of 0.6, a clear concrete cover significantly greater than 3 in. (76.2 mm) is probably necessary to prevent chloride intrusion to the reinforcing steel. The importance of concrete cover is further demonstrated by the finding that, for the typical uncracked bridge deck concrete, corrosion of reinforcing steel was initiated after 7 salt applications when the cover was 1 in. (25.4 mm). With 3 in. (76.2 mm) of clear concrete cover, the reinforcing steel in the uncracked concrete does not corrode after 330 de-icer applications.

Figure 4. Average chloride content versus depth.

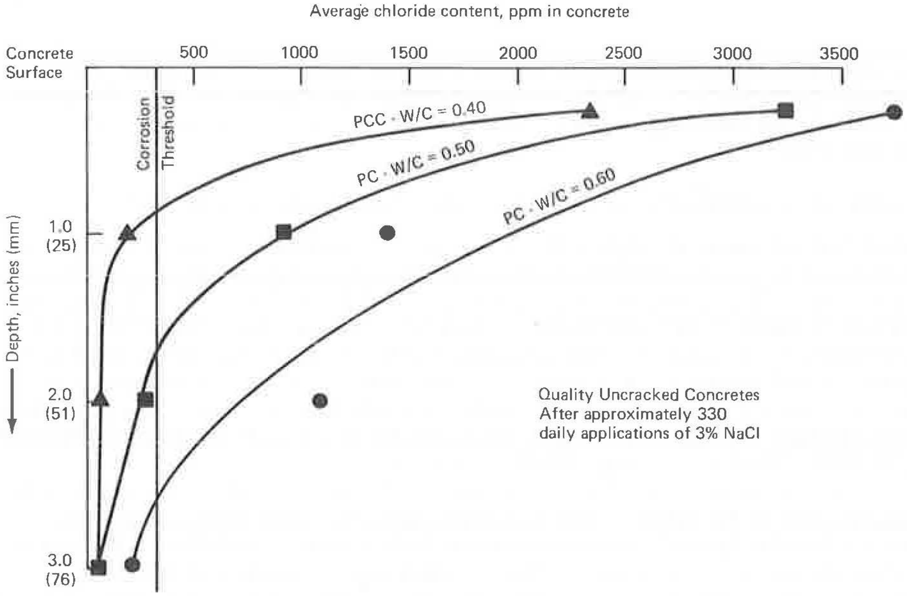


Figure 5. Maximum chloride content versus depth.

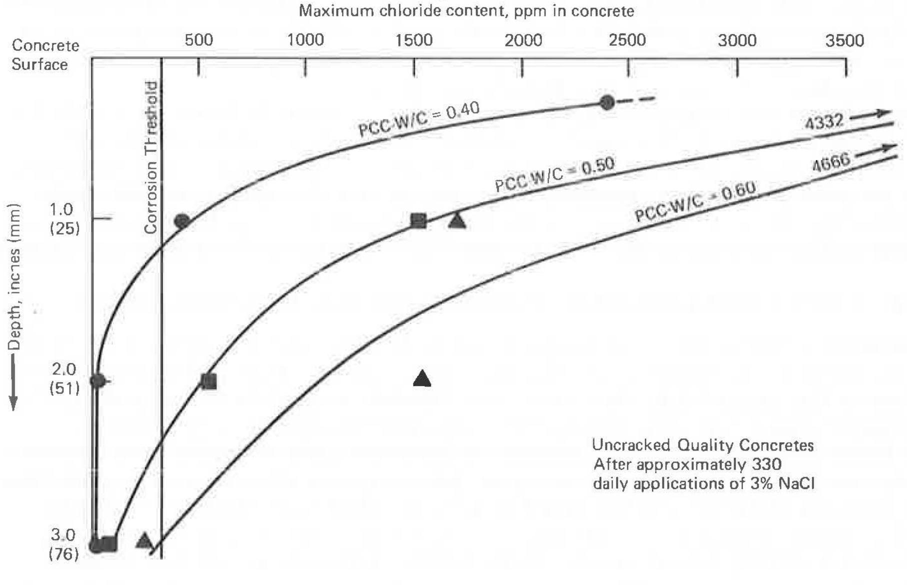
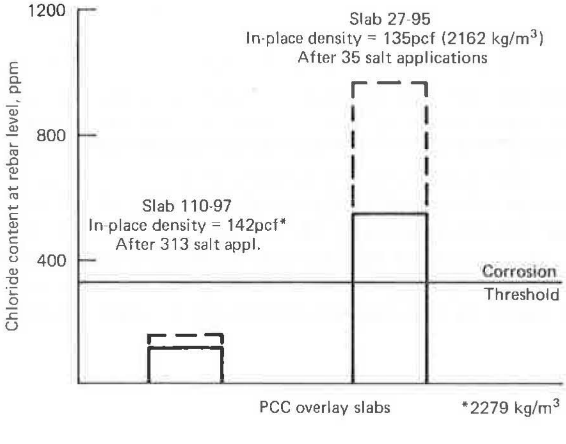


Figure 6. Effect of insufficient consolidation.



EFFECT OF INSUFFICIENT CONSOLIDATION

Failure to achieve at least 98 percent of the rodded unit weight of the concrete during placement resulted in extremely poor resistance to de-icer penetration. The adverse effect of failure to consolidate the concrete is shown in Figure 6. Slab 27-95, which is a portland cement concrete overlay with a water-cement ratio of 0.32, a cement content of 823 lb/yd^3 (486 kg/m^3), and a 1-in. (25.4-mm) reinforcing steel cover, was not properly consolidated during fabrication even though the slab received extensive internal vibration and appeared to have adequate density. The direct transmission nuclear density apparatus indicated the final in-place density was 135.0 lb/ft^3 (2162 kg/m^3), i.e., 92.5 percent of the rodded unit weight, whereas the unit weight of the concrete was 146.1 lb/ft^3 (2340 kg/m^3). Sufficient chloride to induce corrosion had migrated to the level of the reinforcing steel within this concrete after only 28 salt applications as determined by electrical potential measurements. Actual chloride determinations made after 35 salt applications confirmed this finding (965 ppm chloride). Slab 110-97, on the other hand, which was fabricated by using the same concrete mix design, cover, and construction techniques but was properly consolidated, had little chloride at the level of the reinforcing steel after 313 salt applications. The in-place density was 142.3 lb/ft^3 (2279 kg/m^3), i.e., 97.5 percent of the rodded unit weight. Maximum Cl^- was 161 ppm. No reinforcing steel corrosion is occurring in this concrete.

These data indicate that proper consolidation of the concrete is a prerequisite to achieving a chloride-resistant bridge deck concrete. An in-place density of approximately 98 percent of the rodded unit weight of the fresh concrete appears to be a necessity.

CONCLUSIONS

The study findings presented above show that conventional bridge deck concrete, placed with strictest quality control, is not impermeable to chlorides. Redefining "quality" concrete will yield a more chloride-resistant (although not impermeable) portland cement concrete. Other considerations may dictate alternate means of protecting a deck from de-icer penetration and subsequent damage. But if the decision has been made to use a bare concrete bridge deck in an area where significant de-icer exposure occurs, interim findings of this study indicate that

1. The water-cement ratio of the bridge deck concrete should be as close to 0.4 by weight (4.5 gal/bag, 1.5 litre/bag) as is feasible;
2. Some method such as a direct transmission nuclear density apparatus should be used to control consolidation of bridge deck concrete to ensure a minimum in-place density of 98 percent of the rodded unit weight of the concrete; and
3. Minimum clear concrete cover over the reinforcing steel should be 2 in. (50.8 mm) for concrete with a water-cement ratio of 0.4 and 3 in. (76.2 mm) for concrete with a water-cement ratio of 0.5.

These cover depths are recommended on the basis of interim findings and should be considered minimum. Data to be obtained after additional de-icer treatments may or may not indicate that greater cover depths over the reinforcing steel are required. Also, these recommendations are based solely on chloride-penetration data and do not consider the design or construction problems that may be inherent in their use.

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CORROSION AND KANSAS BRIDGES

Carl F. Crumpton and John E. Bukovatz, State Highway Commission of Kansas

Kansas, like other snow-belt areas, has experienced increased corrosion of bridge deck reinforcing and structural steel as the use of de-icing salts has increased. Spalls and hollow planes in the concrete deck form in areas of shallow steel as a result of the corrosion. Some localized small hollow planes, however, are associated with reactive aggregates and also speed up the corrosion process. The copper-copper sulfate half-cell potential detection method has been found useful to predict future hollow plane or spall development related to corroding reinforcing steel. Even new bridges may show a small amount of active corrosion as measured with the half-cell equipment. Much structural steel corrosion seems to be related to the quality of workmanship in preparation and painting and to de-icing salts rather than to the quality of the paint. Steel girders have been observed to act as sacrificial anodes, forming magnetite as they corroded but protecting the reinforcing steel from corrosion. The concrete deteriorated anyway and had to be removed and recast. Concrete deterioration associated with the corrosion of reinforcing steel can be reduced or delayed by increasing the depth of cover over the top reinforcement. Structural steel corrosion can be reduced by proper preparation and quality inspection during painting operations. Carrying salty meltwater away from the steel will also help.

•BRIDGE DECK deterioration has been studied in Kansas for many years. During this time several facets of corrosion problems associated with bridge steel have been observed. This is a report of some of those observations.

The increased use of chloride de-icing salts since the mid-50s has aggravated the problem of bridge steel corrosion and brought about an earlier onset of deterioration in some structures. Bridge corrosion problems are primarily of 2 major types. One is associated with corrosion of the concrete reinforcing steel, and the other is associated with corrosion of the structural steel. Each of these problems has several variations that increase the difficulty of control.

CORROSION OF REINFORCING STEEL

Corrosion of the reinforcing steel has been cited by many investigators (3, 6, 9, 11, 12) as the single most important probable cause of the development of hollow planes or delaminations and spalls in bridge decks. In Kansas, we have investigated the effect of depth of cover on preventing or delaying the onset of corrosion (1, 5).

Depth Studies

Figures 1 and 2 show the depth of the reinforcing steel as measured by a pachometer on a 14-year-old Kansas bridge (5, 8). The 20- by 26-ft (6.10- by 7.92-m) section of the deck shows extensive hollow plane and spall deterioration areas related to steel that is generally less than $1^1/_2$ in. (38.1 mm) deep, and much of it less than 1 in. (25.4 mm) deep (Fig. 1). Figure 2 shows a similar-sized area of the deck where there is little deterioration and the steel is generally more than $1^1/_2$ in. (38.1 mm) deep, and

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Publication of this paper sponsored by Committee on Corrosion.

Figure 1. Reinforcing steel pattern, depths, and types of damage on a 20-ft section of deck.

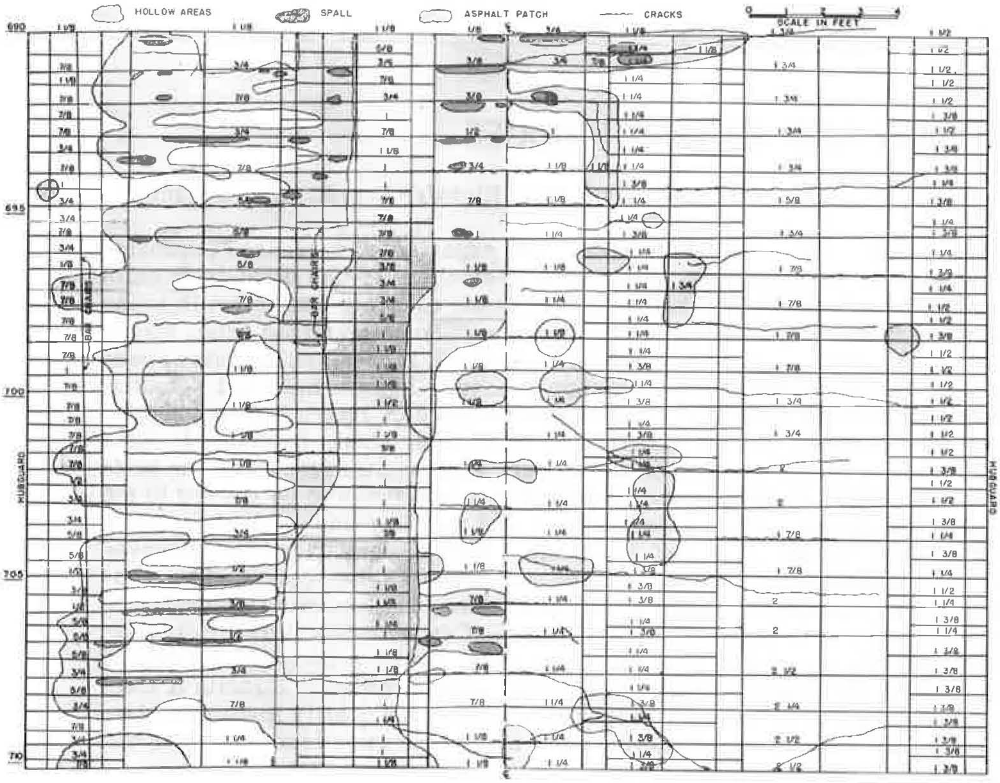
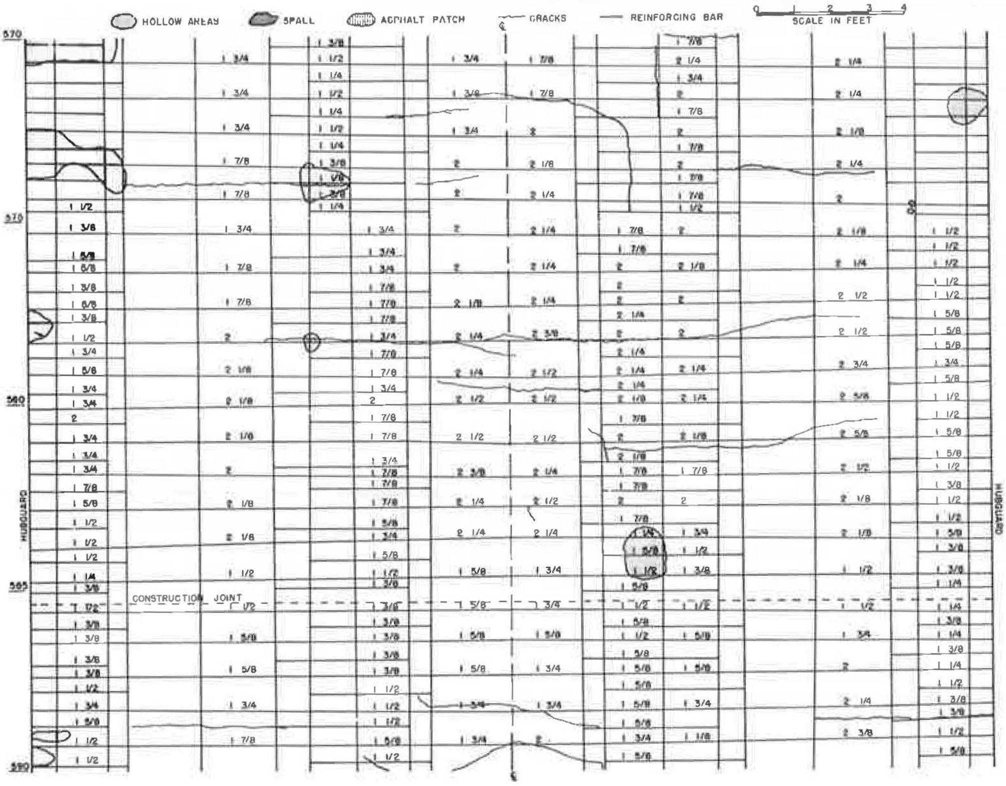


Figure 2. Isolated hollow planes in an area where steel is primarily deeper than 1½ in. (38.1 mm).



much of it more than 2 in. (50.8 mm) deep. Most of the hollow areas shown in Figure 2 are related to expansive chert aggregate particles, and the plane of cleavage is above the reinforcing steel (5).

Figure 3 shows more clearly the relation between the depth of the reinforcing steel and the amount of deterioration on the bridge deck. With passing time more deterioration occurred, and the percentage associated with deeper steel increased but not nearly so rapidly as with shallower steel. The 16-year curve shown in Figure 3 represents the deck surface area actually removed for repair purposes when a 2-in. (50.8-mm) bonded portland cement concrete overlay was placed on the deck (5). This figure shows the importance of placing the steel as deep as possible. Where the average steel depth was 2 in. (50.8 mm) on this deck, only 8 percent of the area was associated with deterioration after 16 years. Nearly all of that was related to expansive chert aggregate or to 1½-in. (38.1-mm) deep steel. Even though the average steel depth in this area of the bridge is 2 in. (50.8 mm), the normal distribution of variation in depth provided about 8 percent of the steel with a cover of 1½ in. (38.1 mm) or less. A cover of 3 in. (76.2 mm) rather than 2 in. (50.8 mm) over the steel on a properly designed deck with concrete of higher cement content and lower water-cement ratio than commonly used should provide many more years of protection (2).

Corrosion Activity Measurements

Field measurement of corrosion activity was made by the Cu-$CuSO_4$ half-cell method (11) on a number of bridges and predicted coming problems. One bridge, described in our final bridge deck report (2), showed no visible damage. A chain sweep found no hollow planes. This bridge was being evaluated in a special research study so that we had yearly data on the deck. The Region 15 Corrosion Device Demonstration Team of the Federal Highway Administration checked the bridge, and the half-cell measurements showed active corrosion (0.35 V or greater) in one area. New inspections and chain sweeps still revealed no damage. About 1 year later, when the bridge was 11 years old, spalls and hollow planes developed in the area of active corrosion. New half-cell measurements showed potentials even higher than those of the previous year. This deck also had a few small hollow areas that developed where there was no active corrosion of the steel. These were determined to be due to large reactive aggregate particles; the plane was above the reinforcing steel.

Another bridge that was overlaid with 2 in. (50.8 mm) of concrete was checked for corrosion potential. The first survey, which was made about 1 year after overlay construction, showed 6 percent of the deck to have a reading of 0.35 V or greater. Two years later, the reading was 9 percent. This indicates that even with the new concrete cover corrosion continues. The added concrete cover, however, should be an aid in keeping the progress of corrosion damage to a minimum.

To follow the corrosion history of some of our bridges, we have begun a study of new bridges. Eight newly constructed bridges were tested for corrosion potential before they were opened to traffic or de-icing salts were used on them. The results are given in Table 1. The tests show that even new decks may have corrosion potentials in the active region. Four of these 8 showed active potentials. Most of the higher values were at the bridge ends, which are places of high readings in older decks. The new bridges are a part of the primary road system and are subject to moderate to heavy applications of de-icing salts. We plan to resurvey these bridges each year and are not sure that covering those that show active corrosion potentials with a membrane would be beneficial.

Corrosion potential surveys were made on a statistical sample of bridges as old as 15 years and now in service; data are given in Table 2. In general, the older decks have somewhat higher potential values than the younger ones, but all decks of the same age do not have similar percentages of high corrosion potentials. Data given in Table 1 indicate that some new bridges also have high readings. The 15 in-service bridges have had de-icing salts used on them and carry primary road system traffic. Bridges of the same age have had about the same amount of de-icing salts applied. Detailed steel depth measurements have not been made on any of the in-service bridges.

Figure 3. Reinforcing bar depth related to the percentage of deterioration.

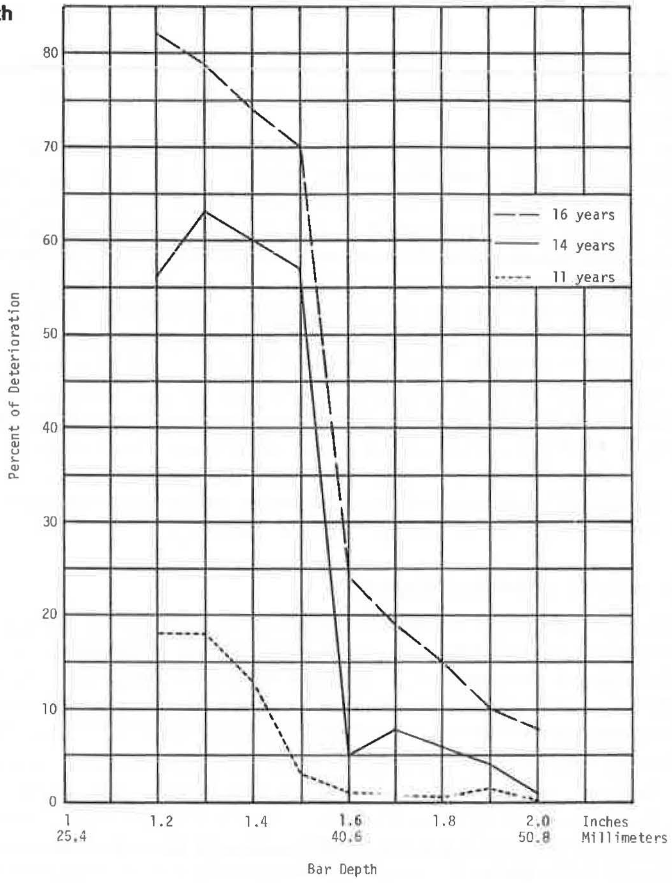


Table 1. Corrosion potentials of new bridges.

Deck Area (ft^2)	Observations	Observations Having Midcell Values for Cell Range of 0.1 V (percent)				
		0.05	0.15	0.25	0.35	0.45
5,200	216	29	64	6	1	0
5,320	243	28	63	6	3	0
5,320	252	43	46	6	5	0
7,504	385	96	4	0	0	0
7,504	388	21	71	4	3	1
14,480	656	87	13	0	0	0
14,480	640	80	20	0	0	0
14,680	650	12	88	0	0	0
Total	3,430	49.5	46.1	2.8	1.5	0.1

Note: 1 ft^2 = 0.09 m^2.

Table 2. Corrosion potentials of in-service bridges in 1973.

ID	Deck Area (ft^2)	Construction Year	Observations	Observations Having Midcell Values for Cell Range of 0.1 V (percent)				
				0.05	0.15	0.25	0.35	0.45
1	4,095	1958	153	24	53	13	4	6
2	11,950	1959	644	7	58	8	2	25
3	6,910	1959	256	0	46	39	10	5
4	5,940	1960	280	1	21	63	14	2
5	4,920	1963	217	57	19	11	12	1
6	6,980	1963	310	90	10	0	0	0
7	8,935	1963	417	49	33	6	6	6
8	9,610	1964	423	30	56	10	3	1
9	5,000	1964	232	36	60	4	0	0
10	4,500	1965	201	22	48	18	10	2
11	6,912	1965	314	46	47	7	0	0
12	14,100	1968	398	36	46	15	2	1
13	7,820	1968	332	24	66	7	3	0
14	9,135	1969	375	99	1	0	0	0
15	6,370	1970	292	50	45	5	0	0
Total			4,844	38.1	40.6	13.7	4.4	3.2

Note: 1 ft^2 = 0.09 m^2.

Changes in Environment

The change of an anode area to a cathode area of a corrosion cell by repairs or by changing environment has been suggested by Stewart (10). We believe this explains some unusual pH relation that we observed on some reinforcing steel.

On a bridge that had been heavily salted, many delaminated concrete sections formed at the anode portion of the corrosion cells. The delaminated concrete was jackhammered out at the anodes leaving many potholes. A heavy rain filled the potholes with water, which rapidly became highly basic from dissolved calcium hydroxide. At several locations we observed small "blisters" on the corroded reinforcing steel. The blisters formed rapidly enough for their growth to be observed. In studying this phenomenon, we discovered that the pH inside the blister was usually below 3 and often below 1. The pH of the water outside the blisters but only 1 or 2 mm away usually was 12 or more. Chemical investigations revealed that the low pH liquid inside the blister was hydrochloric acid, apparently formed because of the changed environment.

High pH conditions were created by the rainwater reacting with the cement-paste phase of the concrete around the pothole. This changed the original anode area to a cathode area compared to another nearby spot in the concrete. Hydrogen ions released at the new cathode apparently reacted with chloride ions in the water to form the hydrochloric acid.

The process was repeated with deteriorated concrete and corroded reinforcing steel in laboratory experiments approximating field conditions, and blisters formed. We found that cathodic areas sometimes formed and HCl was produced on the surface of reinforcing steel even when no blister formed. These areas could be located by placing a solution of potassium ferricyanide [$\frac{1}{2}$ gram of $K_3Fe(CN)_6$ dissolved in 100 ml of distilled water] on the reinforcing steel. If hydrochloric acid was present at the surface of the steel, the acid and potassium ferricyanide reacted with the ferrous iron to form Turnbull's blue. If no acid was present, no blue was formed.

Occasionally the environment of a bridge deck is such that different corrosion aspects are observed. On a pair of 14-year-old steel girder bridges, the outermost eighth of the decks were badly deteriorating full depth. The remainder of each deck was in reasonably good condition. Both decks had been covered with a 2-in. (50.8-mm) hot-mixed overlay before traffic was allowed on the bridge because the decks would receive large amounts of de-icing salts. Water seeped completely through the decks where there was full-depth deterioration. A black deposit accumulated on the bottom of each deck alongside the outermost steel girder. The dark material appeared to form above the girder and to be squeezed out to the side of it by live loads on the bridge. X-ray diffraction studies showed that the black substance was magnetic iron oxide identical to the natural mineral magnetite (Fe_3O_4). We thought that it was being derived from the reinforcing steel in the deteriorating concrete.

The following year the deteriorated concrete was all removed, and a new section cast in its place. We were surprised to find the reinforcing steel with little rust. The magnetic iron oxide formed from corrosion of the top of the steel girders where there was considerable corrosion pitting. In this instance, it appeared that the steel girder acted as a sacrifical anode protecting the reinforcing steel from corrosion even in the presence of seeping saltwater. In spite of the fact that the reinforcing steel had very little corrosion, the concrete was deteriorating full depth, but it was still in place.

Aggregates and Corrosion

Alkali reactive aggregates have been responsible for the formation of small hollow planes above the reinforcing steel (7). When these are near enough to the surface of the deck, they create pop-outs that act as an entranceway for salty meltwater to flow into the hollow plane and carry chlorides close to the reinforcing steel (5). This process speeds the time to corrosion. One means of reducing aggregate reactivity is to use low alkali cement; this has been used in Kansas as in many other states with alkali reactive aggregates. However, alkalies are a corrosion inhibitor (4), and we may be contributing to our corrosion problem in another way by reducing the alkalies in cement to solve our reactive aggregate problem. Whether high alkali or low, the problems seem to persist either way.

Suggestions have been made that some aggregates may affect corrosion. Studies were conducted in our laboratory to determine the corrosion activity of reinforcing steel under conditions of high, medium, and low pH in the presence of carbonate aggregates from different geologic units. The corrosion activity was determined by observing rust formation and by measuring the differential voltage between 2 reinforcing bars of each cell.

Each cell consisted of 2 No. 4 bars separated by $2^3/_8$ in. (60.3 mm) in a 600-ml beaker. The bar ends that were in the cell were insulated so that corrosion would take place only on the bar perimeter. The different aggregates were placed to a depth of 3 in. (76.2 mm) and then covered with the preselected pH solutions.

The pH of the cells originally were 12.2, 5.7, and 4.4. A gel-like rust formed in the higher pH (12.2) cells first. All cells eventually formed the gel-like rust, which eventually permeated the cell liquid and the aggregate. Electrical potential measurements of the cells were not significantly different. At the end of the test (166 hours), the pH of all the cells were in the range of 6.9 to 8 no matter what the beginning pH. No apparent difference in corrosion due to the aggregate was noted.

CORROSION OF STRUCTURAL STEEL

The major problem and expense involved with structural steel has thus far been repainting to prevent extensive corrosion. This is costly and a nuisance, yet we must keep corrosion under control. We have tried various kinds of paints, primers, and preparation methods. The end result always seems to point to the same general conclusion: The success and longevity of the paint were usually more dependent on the quality of the preparation, inspection, and application techniques than on the materials used. The quality of the paint has been controlled well enough, but shortcuts in preparation or lackadaisical inspection have sometimes contributed to a poor final performance. Several years ago we changed our preparation requirements from wire-brushing to sandblasting techniques, and that has been beneficial.

Corrosion begins on some steel girders quite early. It starts in the area below the scuppers where de-icing salts flow through the deck drains and fall onto the bottom flanges of the outer girder. A plastic extension on the cast-iron deck drains to carry the saltwater below the girders before it drops should do much for the longevity of the paint and help prevent rusting of the structural steel.

SUMMARY OF OBSERVATIONS

Bridge deck corrosion has increased in Kansas as the use of de-icing salts has increased. Spalls and hollow planes develop in concrete bridge decks as a result of corrosion. Reactive aggregate particles also create hollow planes and pop-outs that allow de-icing saltwater to come in close proximity to the reinforcing steel.

The copper-copper sulfate half-cell method, if properly used and intelligently interpreted, will reveal areas of corroding reinforcing steel. This method was used to locate corroding steel in a bridge that had no detectable hollow planes or spalls. About a year later both types of damage were found in the area of active corrosion. Some new bridges that have shown a small percentage of readings in the active corrosion range will be monitored through the coming years.

Steel girders seem to serve as sacrificial anodes in unusual situations. Magnetite formed as a product of the girder corrosion, but the reinforcing steel was protected from corrosion even though the concrete deteriorated to the extent that it had to be removed and replaced.

Very low pH spots found on reinforcing steel during repair operations were determined to be hydrochloric acid that was formed as a result of a change in environment, which caused an anode area of a corrosion cell to change to a cathode. Hydrogen ions released at the cathode apparently reacted with chloride ions from the salt-contaminated concrete to form the hydrochloric acid.

Concrete deterioration associated with the corrosion of reinforcing steel can be reduced or delayed by increasing the depth of cover over the top reinforcement and by improving the quality of the concrete. Structural steel corrosion can be reduced by proper preparation and quality inspection during painting operations. Keeping salty meltwater away from the painted steel also helps.

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RECOMMENDED DEPTH OF COVER FOR BRIDGE DECK STEEL

Richard M. Weed, New Jersey Department of Transportation

Seventeen New Jersey bridge decks were surveyed by means of a pachometer (nondestructive electronic testing device). The overall standard deviation for depth of cover was found to be approximately $^3/_8$ in. (1.0 cm). Researchers have generally recommended a minimum depth of 2 in. (5.1 cm) to protect the steel from moisture and de-icing salts and thereby reduce the potential for spalling. If all the steel is to be kept below 2 in. (5.1 cm), a specified depth of cover of $3^1/_8$ in. (7.9 cm) is indicated. An operating characteristic curve is presented from which it is possible to determine the necessary specification to protect any selected percentage of steel. The curve indicates, for example, that 90 percent of the steel can be protected with a specification of $2^1/_2$ in. (6.4 cm). Also included are the statistical parameters for individual bridge decks and several basic suggestions to aid in the reduction of spalling distress.

•SPALLING is attributed to corrosion of the top mat of reinforcing steel due to the penetration of water and de-icing salts. The closer the steel is to the surface, the more vulnerable it is to attack. If the current studies to evaluate bridge deck protective systems should prove waterproofing membranes to be the most practical and effective means of protection, depth of cover over the steel will no longer be an important durability factor. However, the results of these long-range studies will not be known for some time and, in the meantime, many bridges will be built without such protection. The purpose of this paper is to determine the appropriate depth of cover to adequately protect the steel in those bridges that are not provided with membranes.

Although the nature of chloride penetration into concrete is such that it is impossible to define a specific depth of cover below which corrosion of the reinforcing steel will not occur, researchers (1, 2, 3) generally agree that 2 in. (5.1 cm) is an effective minimum depth. Given this and knowing the standard deviation for depth of cover, a researcher can determine the appropriate target value to ensure that all (or some specified percentage) of the steel will have the 2 in. (5.1 cm) minimum amount of cover.

The standard deviation for depth of cover was determined from a recent survey of 17 bridge decks in New Jersey. Although the scope of this study was such that it was not feasible to make a true random selection from all bridge decks in the state, an attempt was made to include various shapes, sizes, and methods of construction (hand and machine finished). Because the data are to be used to derive a specification for use with current construction practices, the decks selected were all comparatively new, ranging in age from a few months to about 5 years.

Depth of cover was measured at approximately 40 random locations on each deck by means of a pachometer, an electronic device that nondestructively measures the distance to the top of the steel by means of magnetic flux. The actual bar size must be known in order to interpret the readings accurately. Depth of cover read directly from the dial is accurate enough for many applications [well within ±$^1/_4$ in. (0.6 cm) when the steel is 2 in. (5.1 cm) below the surface]. Another way is to take numerical readings and determine the depth from calibration curves provided with the instrument. We found a variation of this latter technique to be the most practical for research work.

Publication of this paper sponsored by Committee on Corrosion.

We plotted a calibration curve each day the instrument was used because the curve tended to shift slightly as the batteries became weaker.

With our procedure, the standard deviation for repeat readings was determined to be 0.035 in. (0.089 cm). That is, the instrument will repeat within approximately $\pm^1/_{16}$ in. (0.2 cm) 95 percent of the time. Originally, we intended to account for this component of variance in order to separate it from the variability of the steel in the bridges. After the first calculation was made, it became apparent that the instrument error was a negligible component of the overall variance and, thereafter, it was ignored.

Of the 17 decks surveyed, 9 were built with the previous depth of cover specification of $1^1/_2$ in. (3.8 cm) and 8 were built with the current specification of 2 in. (5.1 cm). Figures 1 and 2 show the data from these 2 groups of bridges. The distributions are approximately normal with means close to the specified depth of cover. The mean for the older bridges is greater than the specified value (i.e., greater depth of cover) by 0.16 in. (0.41 cm); the mean for the newer bridges is less than the specified value by the same amount. Both of these differences are statistically significant, suggesting the existence of an assignable cause.

Basic differences between the 2 groups of bridges may account for this opposite shift of the means, but it is not known which, if any, of these differences was a causative factor. In addition to the difference in the required amount of cover, another major difference is the method of finishing. The older bridges were mostly hand finished, and the newer bridges were predominantly machine finished. As far as we could determine, the steel was secured similarly for all the bridges. A minor change, which occurred at the time the depth of cover specification was changed, was a slight increase in the overall thickness of the deck slab, but it is not known whether this increase in dead load would influence the deflection of the span during the pour in a way that would reduce the cover over the steel. No tolerances on the depth of cover specifications were given, but most of our construction personnel were of the opinion that both specifications were treated as minimums. The data from the earlier bridges (Fig. 1) appear consistent with this belief, and the data from the recent bridges (Fig. 2) do not. The increase in specified depth may have created an attitude on the part of inspectors to the effect that, "if $1^1/_2$ in. (3.8 cm) was satisfactory before, an occasional value less than 2 in. (5.1 cm) will not matter." Finally, the opposite departures of the means from the specified values may simply reflect the inability of the contractors to control the final location of the steel.

For practical purposes, these differences between the actual values and the expected values of the means are not large enough to prevent the use of the data for the derivation of a useful depth of cover specification. There is no known reason why a contractor would benefit by intentionally setting the steel either slightly low or slightly high. Therefore, the belief is that the departure of the mean from a specified target value will be essentially zero on the average, especially if the specification has equal plus and minus tolerances that are enforced. Therefore, the expected distribution for a new specification will be assumed to be normal with a mean equal to the specified value. The standard deviation for this distribution may be determined either (a) by ignoring the departure of the mean and using the data from the more recent bridges (Fig. 2) as the best estimator of current construction practices or (b) by pooling all the data (coded by subtracting the specified value), which will essentially eliminate the departure of the mean. Both methods produce exactly the same result, a standard deviation of 0.38 in. or approximately $^3/_8$ in. (1.0 cm).

This value is then used to construct the operating characteristic curve shown in Figure 3. This curve indicates the depth of cover required to keep any selected amount of steel below the 2-in. (5.1-cm) desired depth. For example, if a designer decided to protect 90 percent of the steel, a depth of cover of $2^1/_2$ in. (6.4 cm) would suffice. If all the steel is to be kept below 2 in. (5.1 cm), a specification of $3^1/_8$ in. (7.9 cm) is necessary.

Once an appropriate target value has been decided on, a suitable plus or minus tolerance must be added to complete the specification. Ideally, a statistical survey of the level of top steel for several bridge decks just prior to placement of the concrete would determine what degree of accuracy could reasonably be expected. In the absence of such a survey, field experience and engineering judgment must be relied on. Realiz-

Figure 1. Bridge decks built with 1.5-in. (3.8-cm) cover specification.

OVERALL DISTRIBUTION

$N = 398$
$\bar{X} = 1.66''$ (4.22 cm.)
$\sigma = 0.33''$ (0.84 cm.)
$\sigma_{\bar{X}} = 0.02''$ (0.05 cm.)

INDIVIDUAL BRIDGE DATA

MEAN (IN.)	MEAN (CM.)	STD. DEV. (IN)	STD. DEV. (CM.)
1.75	(4.45)	0.19	(0.48)
1.27	(3.22)	0.23	(0.58)
1.31	(3.32)	0.22	(0.56)
1.88	(4.78)	0.24	(0.61)
1.63	(4.14)	0.27	(0.69)
1.79	(4.55)	0.20	(0.51)
1.72	(4.37)	0.25	(0.64)
1.74	(4.42)	0.27	(0.69)
1.87	(4.75)	0.28	(0.71)

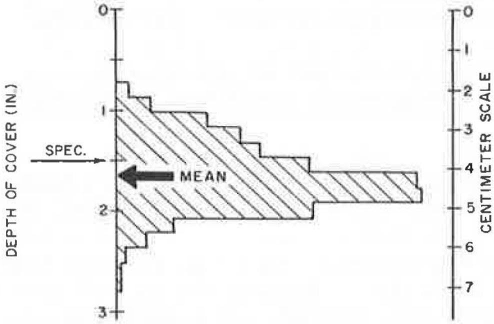


Figure 2. Bridge decks built with 2-in. (5.1-cm) cover specification.

OVERALL DISTRIBUTION

$N = 314$
$\bar{X} = 1.84''$ (4.67 cm.)
$\sigma = 0.38''$ (0.96 cm.)
$\sigma_{\bar{X}} = 0.02''$ (0.05 cm.)

INDIVIDUAL BRIDGE DATA

MEAN (IN.)	MEAN (CM.)	STD. DEV. (IN.)	STD. DEV. (CM.)
1.88	(4.78)	0.48	(1.22)
2.04	(5.18)	0.37	(0.94)
2.24	(5.69)	0.21	(0.53)
1.60	(4.06)	0.37	(0.94)
1.98	(5.03)	0.24	(0.61)
1.63	(4.14)	0.30	(0.76)
1.65	(4.19)	0.20	(0.51)
1.72	(4.37)	0.18	(0.46)

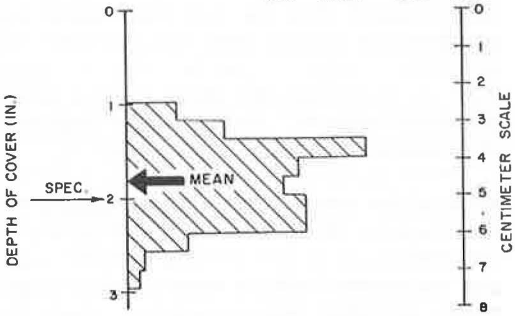
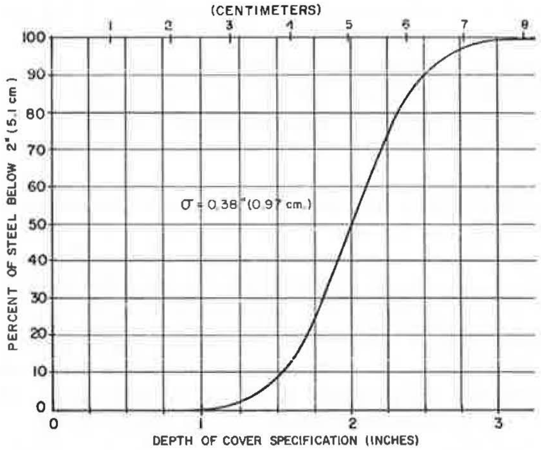


Figure 3. Steel protected at various specified cover depths.



ing that the variability of the steel before the pour would be expected to be less than after the pour and considering the relative ease of correcting steel that is improperly placed, we suggest that $\pm ^1/_4$ in. (0.6 cm) is a suitable tolerance for the setting of the steel.

SUMMARY AND CONCLUSIONS

1. It is believed that the information obtained from 17 New Jersey bridge decks may be generalized to apply to all bridges within the state and should be a useful guide in other areas where construction conditions and techniques are similar.
2. On an overall basis, depth of cover over bridge deck steel was found to be approximately normally distributed with a mean close to the specified value and a standard deviation of approximately $^3/_8$ in. (1.0 cm).
3. On an individual bridge basis, the standard deviations ranged from approximately $^3/_{16}$ to nearly $^1/_2$ in. (0.5 to 1.3 cm), and the means ranged, both plus and minus, up to $^3/_8$ in. (1.0 cm) from the specified value.
4. A statistically significant difference was found between the older bridges built with the $1^1/_2$-in. (3.8-cm) specification and the newer bridges built with the 2-in. (5.1-cm) specification. The mean depth for the older bridges was greater than the specified value, and the mean for the newer bridges was less. Although the reason for this is uncertain, these differences were small in comparison to the overall variability and were ignored in order to develop an operating characteristic curve.
5. To adequately protect the top steel [based on the opinion of other researchers that 2 in. (5.1 cm) is an effective minimum depth of cover], a specification of 3 in. (7.6 cm) or more appears to be necessary. If structural designers are reluctant to go that far, other means to improve the situation should be considered, such as (a) provide better (or more frequent) support for the steel to reduce the variability [for example, if the standard deviation could be reduced from $^3/_8$ to $^1/_4$ in. (1.0 to 0.6 cm), the necessary target value to keep all the steel below 2 in. (5.1 cm) in depth would be reduced from $3^1/_8$ to $2^3/_4$ in. (7.9 to 7.0 cm)]; (b) require more thorough inspection procedures; and (c) specify a less permeable concrete mix (higher cement factor, lower water-cement ratio) for bridge decks to make it more difficult for water and de-icing salts to penetrate to the steel.
6. Although this study focused primarily on the spalling problem resulting from steel that is too close to the surface, structural designers should take note that it is equally possible for the steel to be deeper than the target value and thereby to reduce the load-carrying capability of the deck. The mean depth of steel for individual bridges (Figs. 1 and 2) was observed to deviate from the specified value by as much as $^3/_8$ in. (1.0 cm). In isolated sections of the deck, it may deviate by amounts greater than this.
7. The pachometer proved to be a handy and accurate instrument for use in collecting a large amount of data in a relatively short period of time. It is also a very useful device for checking a contractor's performance. Readings can be taken as soon as the concrete is hard enough to walk on. It is possible that the steel may be properly set prior to placement of the concrete but that subsequent displacement occurs during the construction operation. If this were found to be the case, appropriate remedial action could be taken for the remaining decks on the job.

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NONMETALLIC PROTECTIVE COATINGS FOR CONCRETE REINFORCING STEEL

R. G. Pike and R. E. Hay, Federal Highway Administration; and
J. R. Clifton, H. F. Beeghly, and R. G. Mathey, National Bureau of Standards

The study reported here was conducted to determine the feasibility of using organic coatings to protect reinforcing steel embedded in concrete from corrosion accelerated by chloride ions. Coatings were evaluated on the basis of their chemical and physical durabilities, their protective qualities, their bond to steel, and the bond of the coated bars to concrete. These tests, including pullout and creep tests, indicate that 4 epoxies applied by electrostatic spray techniques are suitable for coating reinforcing steel.

•THE PREMATURE deterioration of concrete bridge decks has become a major problem during the past decade (1). Chloride ions from de-icing materials, usually calcium or sodium chlorides, accelerate corrosion of the steel reinforcing bars. As the products of corrosion increase the volume of the bars, the concrete cracks and spalls and expensive repairs are necessary.

Several possible methods of combating this problem are being studied: cathodic protection of the steel, neutralization of the chlorides, waterproof and salt-proof membranes, internal sealing of the concrete by adding low melting point polymer to the plastic concretes, and use of polymer-impregnated or of polymer concrete. Another possible method for protecting the reinforcing is to coat the steel with some protective material. Zinc (2, 3), cadmium (4), nickel (5), and organic coatings (6, 7) have been used or suggested for such protective coatings.

This study was conducted to ascertain the feasibility of using organic materials for protective coatings. Specific objectives of the study were to (a) select the most promising materials based on physicochemical testing and the economics involved in coating, fabricating, and handling the reinforcement; (b) determine the most practical method of testing such coatings; and (c) prepare sample specifications that might be used to obtain such coated bars.

Forty-seven commercially available materials were furnished by various manufacturers (Table 1). Not all were subjected to the same amount of testing. If a material was determined to be unacceptable by some method, further testing was discontinued. For example, those materials that were extremely brittle, those with a loss greater than 3 grams in the immersion test, those with gel times longer than 8 hours, those with poor film integrity and excessive entrapped air in the cured state, those with excessive softening at 60 C, those with more than 500 percent elongation, those that gave off H_2 gas in $Ca(OH)_2$, those with excessive film thickness (high creep), and those that softened and decomposed at 37.8 C were eliminated from further consideration. In some cases, not enough material was furnished for the complete testing program.

TESTING PROCEDURES AND RESULTS

Tests of Cured Epoxy Disks in Aqueous Solutions

Tests were made of the resistance of the coating materials to various aqueous solutions. Cast disk-shaped, cured epoxy specimens were immersed in water, in an aque-

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Publication of this paper sponsored by Committee on Performance of Concrete—Chemical Aspects.

Table 1. Description of coating materials.

Number	Type	Color	Uncured State	Comments
1	Epoxy	Red	Liquid	
2	Epoxy	Amber transparent	Liquid	
3	Epoxy	Light green	Liquid	
4	Epoxy	Orange	Liquid	Undercoat
5	Epoxy	Brown	Liquid	Polysulfide
6	Epoxy	Gray	Liquid	Topcoat
7	Epoxy	Iron oxide	Liquid	
8	Epoxy	Black	Liquid	
9	Epoxy	Red	Liquid	Primer
10	Epoxy	White	Liquid	
11	Epoxy	Orange	Liquid	
12	Epoxy	Yellow buff	Liquid	Ketamine
13	Epoxy	Light green	Liquid	Ketamine
14	Epoxy	Red	Liquid	
15	Epoxy	Light buff	Liquid	
16	Epoxy	White	Liquid	
17	Epoxy	Gray	Liquid	
18	Epoxy	Black	Liquid	Coal tar
19	Epoxy	Brownish-red	Liquid	
20	Epoxy	Green	Powder	
21	Epoxy	Light green	Powder	
22	Epoxy	Light green	Powder	
23	Polyvinyl chloride	Dark olive green	Powder	
24	Polyvinyl chloride	Dark purple	Powder	
25	Epoxy	Blue	Powder	
26	Polyvinyl chloride	Transparent	Primer	
27	Epoxy	Black	Powder	
28	Epoxy	Black	Powder	
29	Epoxy	Yellow	Powder	
30	Polyvinyl chloride	Pale green	Powder	
31	Epoxy	Light green	Powder	
32	Epoxy	White	Powder	
33	Urethane	Dull orange	Liquid	Unsolicited
34	Phenolic nitrile	Red	Liquid	
35	Urethane	Black	Liquid	100 percent solids
36	Urethane	Black	Liquid	100 percent solids
37	Epoxy	Black	Liquid	Adhesive
38	Epoxy	Gray	Powder	
39	Epoxy	Brown	Powder	
40	Epoxy	Red	Powder	
41	Epoxy	Red	Powder	
42	Epoxy	Red		
43	Epoxy	Red	Powder	
44	Zinc-zinc silicate	Gray	Liquid	
45	Coal tar epoxy	Black	Liquid	Coal tar
46	Epoxy	Red	Liquid	Polysulfide
47	Polypropylene	Clear	Powder	

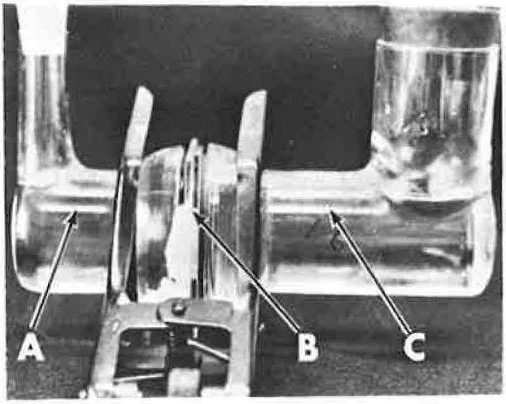


Figure 1. Permeability cell (A—compartment containing distilled water, B—epoxy film sandwiched between 2 glass plates, each having centered 1-in. diameter holes, and C—compartment containing 3M NaCl).

ous solution of 3M NaOH, and in a solution saturated with $Ca(OH)_2$, $CaSO_4 \cdot 2H_2O$, and 0.5M $CaCl_2$. However, because it was impossible to fabricate this type of specimen from epoxy powders, this method of test is not recommended for prequalification of organic coatings for steel reinforcing bars.

As shown in a previous report, which gives a detailed description of the test method and results (8), some materials are highly unsatisfactory. One material, after 31 weeks of exposure, showed an increase in weight of 17 percent in water, 20 percent in 3M $CaCl_2$, 15 percent in 3M NaOH, and 18 percent in $Ca(OH)_2$, $CaSO_4 \cdot 2H_2O$, and 0.5M $CaCl_2$. Another coating, after 31 weeks of exposure, showed a decrease in weight of 6.8 percent in water, 10 percent in 3M $CaCl_2$, and 14 percent in a saturated solution of $Ca(OH)_2$, $CaSO_4 \cdot 2H_2O$, and 0.5M $CaCl_2$ and an increase in weight of 9.3 percent in 3M NaOH.

Chloride Permeability

Cured films 3 to 7 mils (0.08 to 0.2 mm) thick were clamped in a permeability cell (Fig. 1), and the rate of passage of chloride ions was determined. This method and the results obtained have also been described in detail previously (8). All but 2 films (13 and 16) tested appeared to be sufficiently impervious to chloride penetration.

Immersion Tests of Coated Reinforcing Bars

Coated No. 6 reinforcing steel bars were immersed in an aqueous solution of 3M NaOH and in a solution saturated with $Ca(OH)_2$ and examined periodically for evidences of softening, color changes, disbonding, and changes in film integrity. Table 2 gives the appearance of some of the bars after 270 days of exposure. The effects after 45 days of exposure have been described (8). Rusting occurred on some of the coated bars in $Ca(OH)_2$ after 15 days, but the uncoated bar showed no corrosion after 45 days. Bars 39 and 40 showed no change after 270 days when they were sandblasted before coating, but showed rusting during the first 15 days of exposure to $Ca(OH)_2$ when they were sandblasted and phosphatized. The corrosion of the uncoated bar in $Ca(OH)_2$ at some period after 45 days is unexplained.

Effect of Impressed Voltage

Stresses that can destory the bond of coatings to steel can be induced by cathodic protection devices, stray currents, or corrosion. The effects of such stresses were evaluated by a modification of the disbonding tests (ASTM G 8-69T). Duplicate 6-in. (152.4-mm) coated bars were used as both the cathode and anode. They were immersed in a 7 percent solution of NaCl, and a potential of 2 V was applied. The electrodes were observed periodically for evidence of hydrogen gas evolving at the cathode and for corrosion products of iron forming at the anode. Results of these tests were previously reported (8). Coatings (applied in the indicated thicknesses) that permit the evolution of hydrogen gas within 15 minutes are of doubtful value.

Electrical Potential and Resistance in Solutions

The electrical potential and electrical resistance of the coated bars were compared with those of uncoated bars, and the results are also recorded in the interim report (8). As discussed more fully below, these results could not be satisfactorily rationalized.

Electrical Potential and Resistance of Bars Embedded in Concrete

The corrosion potential of bars embedded in concrete was determined by fabricating "lollypops," similar to those described by Stratfull (9, 10). The coated bars were embedded in concrete prisms 2⅞ by 4⅞ by 15 in. (7.5 by 12.5 by 38 cm) coincident with the longitudinal axis of the prism. A terminal was inserted into one end of each bar for making connections to a voltmeter or conductivity bridge. The other exposed end of the bar was covered with a thick coating of silicone sealant. The lollypops were then placed vertically in a tank containing enough 3.5 percent sodium chloride solution to

Table 2. Results of immersion tests of coatings on reinforcing bars.

Number	3N NaOH	Saturated CA(OH)
22	No change	No change
25	No change	No change[a]
29	No change	Slightly rusted
30	No change	No change
31	No change	No change
32	No change	No change
38		
Sandblasted by ore coating	No change	Rusted[b]
Sandblasted and phosphatized before coating	No change	Rusted[b]
39		
Sandblasted by ore coating	No change	No change
Sandblasted and phosphatized before coating	No change	Rusted[b]
40		
Sandblasted by ore coating	No change	No change
Sandblasted and phosphatized before coating	No change	Rusted[b]
41	No change	No change
Uncoated	No change	Rusted

[a] No rust, but numerous small blisters formed apparently by water penetrating through coating.

[b] During the first 15 days of immersion; afterward rusting lifted most of epoxy from bar.

Table 3. Electrical potential and resistance of bars embedded in concrete.

	24 Hours		3,480 Hours		
Coating[a]	Potential (MV)	Resistance (Ω)	Potential (MV)	Resistance (Ω)	Protective Rating[b]
1A	345.0	3.8×10^2	283.0	3.9×10^2	3
1B	408.8	7.0×10^2	362.4	8.2×10	
1-1	337.0	2.5×10^2	215.0	2.5×10^2	–
1-S	484.5	4.8×10^2	371.5	4.2×10^2	–
3A	285.6	3.1×10^2	432.4	2.2×10^2	3
3B	260.3	2.7×10^2	365.5	2.4×10^2	
4A	339.2	2.4×10^4	142.3	1.1×10^5	3
4B	130.0	1.0×10^5	115.5	1.4×10^4	
18	575.6	6.0×10^3	003.0[c]	1.0×10^4	3
19A	484.0	5.6×10^2	399.5	5.4×10^2	2
19B	438.0	6.1×10^2	282.0	6.0×10^2	
25	542.7	4.1×10^2	271.4[c]	5.1×10^2	1
27A	654.6	1.3×10^4	167.0[c]	7.2×10^4	3
27B	571.5	6.8×10^3	542.0	1.1×10^4	
28	461.5	5.2×10^2	262.8[c]	5.4×10^2	3
29A	376.3	6.4×10^2	163.0[c]	7.8×10^2	2
29B	403.4	6.6×10^2	360.5	5.4×10^2	
30A	058.0	1.0×10^5	N.C.[d]	2.1×15^5	1
30B	448.2	1.5×10^5	127.4[c]	1.6×10^5	
31A	359.8	1.5×10^3	038.5[c]	9.8×10^4	1
31B	092.2	9.8×10^3	013.5	6.2×10^4	1
38	392.7	3.2×10^2	165.7	4.1×10^2	3
39A[a]	513.0	4.9×10^2	348.0	4.7×10^2	3
39B[a]	536.2	5.0×10^2	402.0	4.8×10^2	
40A[a]	282.2	2.5×10^2	256.6	2.2×10^2	2
40B[a]	382.5	3.4×10^2	325.5	2.7×10^2	
40A	431.8	2.9×10^2	398.0	3.1×10^2	3
40B	377.0	2.8×10^2	316.9	2.3×10^2	
41A	540.5	6.0×10^3	432.2	1.3×10^4	2
41B	575.9	5.4×10^2	324.4	2.5×10^4	
Uncoated A	334.2	2.7×10^2	206.6	2.3×10^2	4
Uncoated B	264.0	2.6×10^2	180.3	2.2×10^2	

[a] A and B denote duplicate specimens.

[b] From reference 8, Table 8.

[c] Large shifts in electrical potential attributed to self-sealing of small holes in the silicone seal.

[d] No current flow and, therefore, no voltage reading.

cover the lower 13 in. (33 cm) of the concrete prism. Provided the silicone sealant did not leak, the bars were exposed only to those chloride ions penetrating the protective layer of concrete and nonmetallic coating. The specimens were inspected regularly for evidence of cracking, products of corrosion, or other signs of failure. Periodic measurements were made of the electrical potential in relation to the saturated calomel half-cell (9, 10) and of the electrical resistance between a platinum electrode in the solution and the bar. The results, given in Table 3, should be compared with those given in the earlier report (8, Table 5) for coated bars immersed in salt solution rather than embedded in concrete. The reason that the bars in solution had higher resistances than those embedded in concrete has not been explained. Furthermore, why the uncoated bars showed a passive potential of -296 mV while many of the coated bars showed a highly active potential is also unexplained. For these reasons, this method of evaluation has not been recommended in the prequalification tests, and some other method should be developed to determine the condition of coated bars in actual use.

Physical Properties

Tests were performed to determine the ability of coatings to resist rough handling during fabrication and placement in the deck. Before these tests were started, the film thicknesses and the number of holidays (pinholes not visible to the naked eye) were determined, and the bars were examined for evidence of damage or faulty application of the coating. These measurements were reported previously (8).

Impact Resistance—The impact resistance of the coatings was determined by a method similar to that of ASTM G 14-69T. In this test, a 4-lb (1.81-kg) tup terminating in a hemispherical nose ⅝ in. (15.9 mm) in diameter was dropped 30 in. (762 mm) onto the area between the deformations of the bar. [Based on the ⅝-in. (15.9-mm) nose diameter, the maximum area struck was 0.31 in.^{2} (2 cm^2).] The area of impact was visually examined, and the area of damage measured. With an impact of 120 in.-lb (13.6 N-m) not more than 0.15 in.^{2} (0.96 cm^2) should be damaged. Only 3 of the materials tested exceeded this value (Table 4).

Resistance to Abrasion—The abrasive resistance of the various coatings has been described previously (8). An abrasion loss of less than 100 mg/1000 cycles in the Tabor abraser (ASTM D 1044-56) with a 1,000-gram load per wheel should indicate satisfactory abrasive resistance.

Hardness—Five of the materials were tested according to ASTM D 1474-68. A 10-gram load was used. The Knoop hardness number (KHN) of each of these materials is given below. A KHN of 16 will give a satisfactory coating.

Code	KHN
22	20.7
29	19.8
30	6.7
31	20.7
39	21.2

Bend Tests—The coating materials were also evaluated by bending the coated bars to an angle of 120 deg over a mandrel with a 3-in. (76-mm) radius and examining the coating for cracking, chipping, scaling, or other damage. These results are also recorded in the earlier report (8, Table 8). Coated bars subjected to this test should show no visible cracks in the coating. Even if the coatings pass this test, they will not necessarily withstand the sharp bending and rough handling received during fabrication of the steel. However, the test is perhaps the simplest and most valuable for quality control purposes because insufficient curing of the coating, inadequate surface preparation (sandblasting) of the bar, excessive film thickness, or even use of the wrong epoxy may all become evident during this test.

Pullout Tests—One of the major items for concern in using coated reinforcing bars is that the bond between the steel and concrete may be reduced. Bond was evaluated by comparing the pullout strength of coated bars with that of uncoated bars. A procedure described by Mathey and Watstein (11) was used. Descriptions and results of those tests have already been reported (8, Table 9). Selected values from the earlier report are given in Table 5. The average bond strength of the uncoated bars with horizontal deformations at a slip of 0.002 in. (0.05 mm) at the free end of the bar is 1,008 lb/in.^{2} (6.9 MPa), whereas the strengths of 3 of the materials recommended for use are slightly above this value. The highest value obtained was for a bar coated with a coal tar epoxy (code 18) at 1,352 lb/in.^{2} (9.3 MPa). For comparison, a polyvinyl chloride coated bar (code 23) had a bond strength of only 25 lb/in.^{2} (0.17 MPa).

Creep—The creep or long-time slippage of the coated reinforcing bars in concrete under tensile stress can also be a matter for concern. Therefore, the creep of coated bars was compared with that of uncoated bars. Typical results are shown in Figures 2 and 3; bars coated with epoxy (code 31) exhibited even less creep than the uncoated bars. As might be expected, the bars heavily coated with polyvinyl chloride (code 30) show creep values several magnitudes greater than the uncoated bars. Also, coal tar epoxy had an excessive creep.

As far as we are aware, no such creep determinations of reinforcing steel in direct tension have been made; therefore, a brief description of the procedure will be given here.

The No. 6 reinforcing bars were embedded in the center of concrete prisms 10 by 10 by 12 in. (25.4 by 25.4 by 305 mm). These were mounted in frames as shown in Figure 4. The creep at the free end of the bar was measured by a dial gauge reading directly to 0.0001 in. (0.0025 mm) and mounted on a support attached to the top face of the concrete by bolts screwed into inserts cast in the concrete. The gauge was thus bearing on the exposed end of the reinforcing steel. At the loaded end of the specimen, 2 similar gauges were attached to a steel bar fastened to the lower face of the concrete by bolts secured to inserts cast in the concrete. These gauges bore on a steel yoke fastened to the reinforcing bar about 1 in. (25 mm) below the face of the concrete. This yoke was free to move in a recess in the base plate. The slippage (creep) of the loaded end of the reinforcing bar is then taken as the average reading of the 2 gauges.

The load on the bars was developed through large steel coil springs, also shown in Figure 4, which were first calibrated by measuring the load applied by a 60,000-lb (27,000-kg) capacity electromechanical universal testing machine versus the compressive displacement of the springs measured with dial gauges reading directly to 0.0001 in. (0.0025 mm). The setup for calibrations is shown in Figure 5. The compressive displacement versus load was nearly the same for all 24 springs tested as is shown in Figure 6.

The constancy of the stress in the rods was monitored by resistance strain gauges stated by the manufacturer to have negligible creep during a 1-year period.

CONCLUSIONS

This research shows that some powdered epoxy coatings applied by electrostatic spray techniques can prevent or greatly delay the corrosion of reinforcing bars in concrete. Many of the coated bars can give bond strengths under short-time and sustained loads equivalent to bond strengths of similar uncoated bars.

IMPLEMENTATION

From this research, specifications for the prequalification of any type of organic coating have been developed. Many of the manufacturers who submitted materials for this investigation may wish to modify those materials that failed to meet certain of the requirements by changing the composition of the coating material or some procedures such as curing methods or bar preparation.

Simple tests based on this research have also been developed for use as quality control specifications.

The Federal Highway Administration has suggested that states use coated bars on an experimental basis to determine the feasibility of such coatings, and portions of several bridge decks are now being constructed with epoxy-coated bars from 2 suppliers.

Table 4. Impact resistance of coatings on bars.

Code	Film Thickness (mils)	Damaged Area (in.^{2})	Type and Severity of Damage
2	5 to 15	0.110	Shattering and disbonding of coating propagating from area of impact
3	2 to 5	0.028	Only indention in coating and bar at impact area
4	10 to 20	0.082	Shattering and disbonding of coating propagating from area of impact
5	10 to 15	0.383	Large amount of shattering and disbonding of coating surrounding area of impact
10	10	0.079	Shattering and disbonding of coating at impact area
11	10 to 12	0.188	Shattering and disbonding of coating propagating from area of impact
16	2 to 4	0.038	Slight shattering and disbonding of coating at impact area
17	4	0.028	Slight shattering and disbonding of coating at impact area
18	4	0.038	Slight shattering and disbonding of coating at impact area
19	1	0.028	Only indention in coating and bar at impact area
22	25	0.234	Large amount of shattering and disbonding of coating surrounding area of impact
23	25	0.077	Large indention in coating
24	35	0.110	Large indention in coating
25	6 to 11	0.049	Shattering and disbonding of coating at impact area
27	8	0.077	Coating shattered at area of impact with slight propagating of shattering from impact region
28	1 to 2	0.038	Slight shattering and disbonding of coating at impact area
29	1 to 2	0.028	Slight shattering and disbonding of coating at impact area
30	15 to 18	0.110	Large indention in coating accompanied by slight cracking at impact area
31	8 to 9	0.110	Shattering and some disbonding of coating at impact area
32	4 to 6	0.049	Cracking in coating at impact area; slight cracking extending from impact region
33	3 to 4	0.028	Shattering of coating at impact area
38	2 to 4	0.038	Shattering of coating at impact area; slight cracking extending from impact region
39	2 to 4	0.028	Only indention in coating and bar at impact area
40	2 to 4	0.079	Shattering of coating at impact area; slight disbonding extending from impact region
41	3 to 7	0.038	Shattering of coating at impact area; slight cracking extending from impact region
42	3 to 4	0.028	Only indention in coating and bar at impact area
43	3 to 4	0.038	Smashing of coating at impact area; slight cracking extending from impact region

Note: 1 mil = 0.0254 mm, and 1 in.^{2} = 6.54 cm^2.

Table 5. Results of pullout tests.

Code	Avg Bond Strength (lb/in.^{2})	Code	Avg Bond Strength (lb/in.^{2})
Uncoated-H	1,008	31-H	1,056
18-H	1,352	39-H	1,121
23-H	25	41-H	1,046
25-D	986		

Note: 1 lb/in.^{2} = 0.07 kg/cm^2.

Figure 2. Creep at free end under 30,000 lb/in.^{2} load.

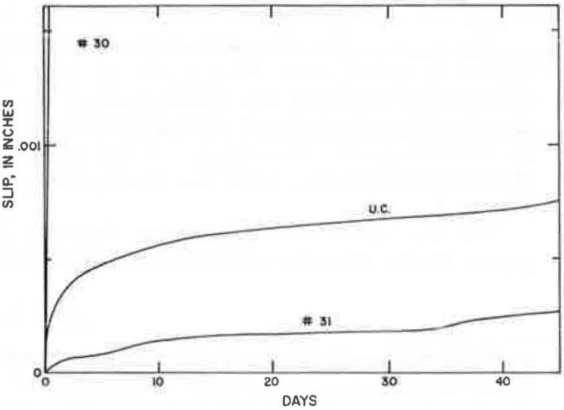


Figure 3. Creep at loaded end under 30,000 lb/in.^{2} load.

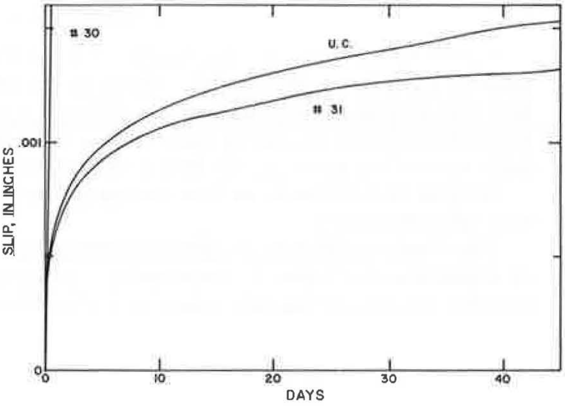


Figure 4. Assembly for measuring creep of bars in concrete.

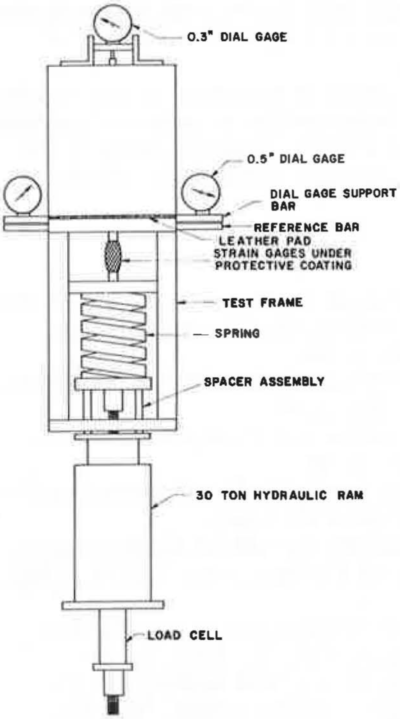


Figure 5. Spring calibration apparatus.

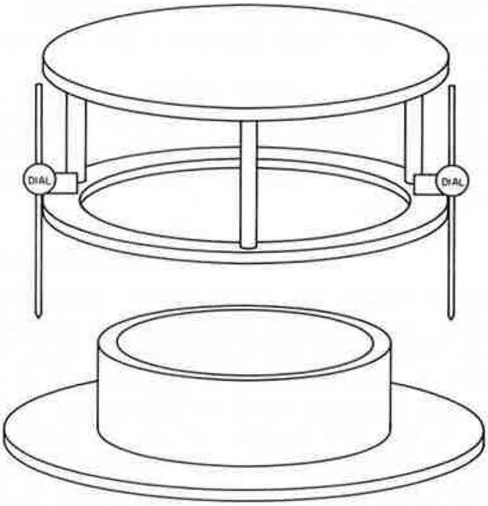
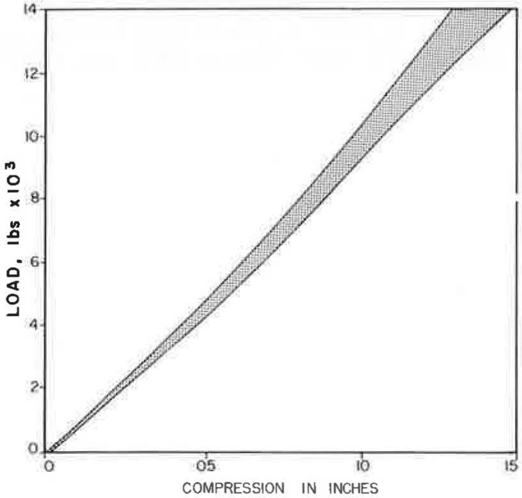


Figure 6. Range of compressive development versus load of all 24 springs used in tests.



ACKNOWLEDGMENTS

The authors wish to thank Erick Anderson for his skillful and painstaking preparation and handling of the pullout and creep tests. We also wish to thank all the producers and fabricators who kindly furnished the coatings and coated bars for this series of tests.

The contents of this report reflect the views of the Office of Research of the Federal Highway Administration, which is responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policy of the Department of Transportation. This report does not constitute a standard, specification, or regulation.

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INITIAL CORROSION SURVEY OF THE BAY AREA RAPID TRANSIT SYSTEM

Peter L. Todd, Bay Area Rapid Transit District, Oakland, California

The 75-mile (120-km) Bay Area Rapid Transit System has a dc traction power supply that uses the continuously welded steel running rails for its negative current return. The rails are mounted on insulating fasteners to minimize leakage current. However, high values of stray current have been measured during the initial operation of the system. Testing has demonstrated that the negative return grounding at traction substations and its interconnection with other grounding has been the major cause of stray earth currents. An alternate method of negative return grounding through diodes has been tested and is being installed. The diodes block exchange currents, but allow leakage and fault current return to the traction rectifiers.

•THE BAY AREA Rapid Transit (BART) System, like many other direct-current traction systems, uses its tracks for negative return of traction current. Since these 4 rails extend for 75 miles (120 km) and are physically close to the earth, they can be the source of stray earth currents (1). Such stray currents are a major cause of electrolysis of buried metals in the vicinity of direct-current traction systems (4). This paper discusses the nature of the BART system earth current problem and what is being done to solve it.

DESCRIPTION OF ELECTRIFICATION SYSTEM

The running rail negative return consists of the 4 main-line steel rails that are continuously welded and bonded at 1,000-ft (305-m) intervals. This circuit element has a resistance of about 2.5 mΩ/1,000 ft (8.2 mΩ/km) of track. The rails are insulated from the earth and structures except at traction substations. The rail insulation is provided by the rail mounting. Insulating pads and clips are used to mount the rails on concrete ties, and special insulating fasteners are used to mount the rails on the concrete surfaces of aerial structures and subway inverts. Wood ties and ballast are used for rail support at switches and through seismic zones.

The 37-traction power substations are located where loading and voltage drop dictate. The BART system is center fed, so most substations are located adjacent to passenger stations. These traction substations are supplied from twin 34.5-kV cables. Transformer rectifiers at the substations convert the 34.5-kV, 3-phase, 60-Hz energy to 1,000-dc energy. The circuit to the transit vehicles is made by a 1,000-V third rail, sliding contact shoes on the cars, car wheels, and the track negative return. As noted before, the traction rectifier negative is grounded at each substation.

The twin 34.5-kV cables are fed at 7 points by the local electric utility company. At grade, the cables are direct buried and lead covered. On aerial structures and in tunnels, the cables are in 2 nitrogen-filled pipes.

The 75 miles (120 km) of main-line double track are supported on several types of structure, all of which can be damaged by stray current electrolysis. About 20 miles (32 km) are in tunnel, 27 miles (43 km) are in aerial structures, 24 miles (39 km) are at grade on exclusively occupied right-of-way, and 3.6 miles (5.8 km) are in the Trans-Bay Tube. In addition to the main line, there are 3 storage yards in the East Bay.

Publication of this paper sponsored by Committee on Corrosion.

The vehicles are powered by 4 chopper-controlled dc traction motors. Each car draws about 1,000-A current at the start and about 200-A current at full speed. The vehicles are equipped with regenerative breaking so that a decelerating train can feed energy to a nearby accelerating train. This latter feature conserves part of the energy that would otherwise heat the breaking resistors.

TESTING AND MODIFICATIONS

Each traction rectifier is equipped with 2 shunts: 1 measures total rectifier current and 1 in the rectifier ground connection measures current over paths other than the rail negative return. Initial measurements made with a single train operating and only part of the line energized showed 10 to 20 percent of the local rectifier current in the ground connection. These are extremely high leakage current values, and further investigation has shown that the greatest part of the current is caused by connections of the substation ground mat to other structures and not by leakage from the rails.

For instance, a conduit connected to a traction substation ground mat at one time carried hundreds of amperes. Passenger station lighting and equipment power is supplied by a 480-V utility service, which is separate from the traction service. In this case the conduit connected the traction substation ground mat to the passenger station ground mat, which was connected to a water main with a faulty dielectric union. Where possible, we are eliminating connections between the traction substation grounds and passenger station service grounds.

We made initial coordinated tests with the local gas supply utility. Its gas mains cross and parallel the BART system right-of-way. In at least one case, in the test area, a main is within 10 ft (3 m) of a traction substation ground mat. For these tests a single train was cycled over the A-line segment. The time and location of train starts were recorded onboard while chart recorders monitored potentials and current flow on gas pipes and at traction substations. Two tests were made: 1 with traction substation negative return ground connections in place and 1 with these ground connections opened. With ground connections in place, current flow between traction substation ground mats was recorded. With ground connections open, voltages between rails and ground were recorded. Analysis of the charts enabled us to determine current paths, and the tests showed that lifting the traction rectifier's connections to ground markedly reduced stray current carried on the gas lines.

Current pickup on a gas main revealed grounded connections at a switch on the aerial structure. Investigation revealed contacts of the running rail fasteners and the restraining rail fasteners at the switch. The restraining rail fasteners, which mount a rail outboard of the running rail for containment of car wheels in case of derailment, are, of course, tied to the reinforcing steel. Contacts between running rail and restraining rail fasteners have been eliminated on the system.

The current exchanges between traction substation ground mats of as high as 15 percent of current supplied to the train seemed surprisingly high given the ground mat resistances and the open circuit rail drop voltages. The 34.5-kV cable pipes provide a path for these exchange currents. These pipes are grounded at the traction substations and are supported on the tunnel liners and aerial structures. Analysis and testing of the aerial structure case show that these pipes carry the exchange currents. These conclusions apply to the pipes wherever they are found on the system.

The aerial girders are isolated from other structures to withstand seismic forces. One end of the girder has a horizontal pin in a plastic sleeve, and the other end is mounted in vertical dowels in plastic sleeves. Elastomeric pads between vertical and horizontal joint surfaces provide a degree of electrical isolation between girders and between girders and support columns. The 34.5-kV pipes are hung from the girders and anchored and grounded at every fifth column. Figure 1 shows the circuit.

The pipes and their ground connections every 300 ft (91 m) form an extended ground conductor whose calculated resistance (2) is equivalent to the 4-rail negative return circuit.

The driving voltage for the substation ground current exchanges, as for all stray traction current, is the voltage drop in the running rail negative return. The traction

Figure 1. Solidly grounded substations.

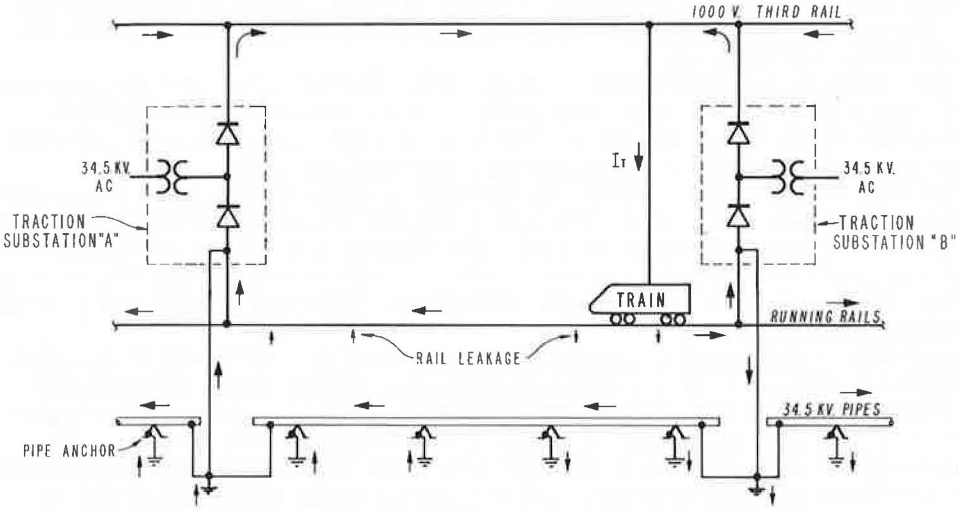
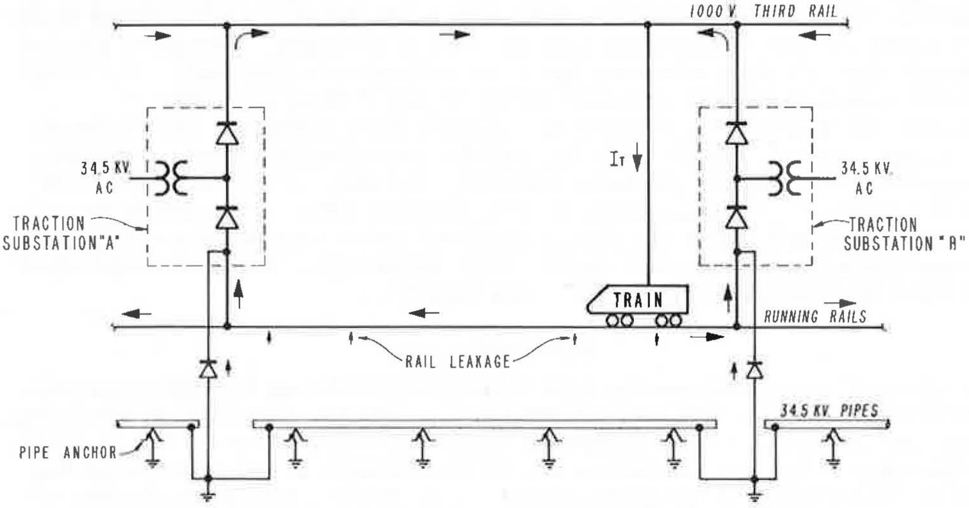


Figure 2. Diode-grounded substations.



power system is designed so that about 30 percent of an accelerating train's power requirement is supplied from the 2 substations adjacent to the substation with maximum loading. Our tests indicate that half of this current or about 15 percent does take the ground path between substations.

Since the extended circuit elements, running rails, 34.5-kV pipes, and the earth are connected only at the substations, a break in the negative return ground connection should eliminate the exchange currents. This is exactly what was observed in the test with traction substation negative return ground connections opened.

The rail drop voltages will of course appear at these open connections. These voltages exist between rails and ground. When the rails are grounded, the voltages appear between the rail grounds and remote ground. The voltages also appear between the passenger station platforms and the running rails, if platform and rail are not connected. Any attempt to bond rails and platform will transfer the potentials to other and perhaps more hazardous locations.

The magnitude of these rail-to-ground voltages is from 10 to 30 V with the present 4-car revenue operation. If the ground connections are opened, these voltages may double; but the values are still comparable to those measured in other transit properties. We used field data and a network analyzer to develop a clearer picture of the distributed negative return circuit characteristics. This study showed that under certain fault conditions the voltage between rail and ground can be quite high, and it is necessary to consider touch potentials and provide for safety.

As a consequence of this study, we tested diode grounding. Figure 2 shows this technique. The diodes in the negative return ground connection permit fault current and rail leakage current to return to the substation, but block exchange currents over the ground mat connections. Our tests indicate that the ground currents should be reduced by several orders of magnitude with this type of grounding. The stray current would then be due to leakage over rail fasteners and inadvertent grounds. The latter have proved easy to locate and eliminate during testing of diode grounding.

Of course, we still have the problem of rail to platform voltages. Our platforms are reinforced concrete, and the steel has not been intentionally bonded. The measured resistances between the platform surfaces and ground are as low as 100 Ω/ft^2 (92.9 Ω/m^2) when wet. This insulation level is marginal when one considers the voltages that may develop between car side and platform under some system conditions. We are now testing membrane surfaces for the platform edge. These surfaces would provide both slip resistance and electrical isolation (3).

SUMMARY

High values of stray earth current have been measured during the initial operation of the dc traction system. The tract negative current return is mounted on insulating fasteners, so little leakage is expected or measured from this source. The negative return grounding at traction substations and its interconnection with other grounding has been the major cause of the earth currents. An alternate method of negative return grounding through diodes has been tested and is being installed.

ACKNOWLEDGMENT

This project was performed with assistance from the U.S. Department of Transportation. The contents of this report reflect the views of the author, who is responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the Bay Area Rapid Transit District or the Urban Mass Transportation Administration. The author wishes to acknowledge the work and contributions of R. H. Miller, D. R. Stanley, O. Kramer, and E. F. Wargin, members of the staff at the Bay Area Rapid Transit District.

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