TRANSPORTATION RESEARCH RECORD 604

Bridge Decks: Corrosion, Cathodic Protection, and Pavement Seals

Notice

The project that is the subject of this report was approved by the
Governing Board of the National Research Council, whose members
are drawn from the councils of the National Academy of Sciences,
the National Academy of Engineering, and the Institute of Medicine.
The members of the committee responsible for the report were cho-
sen for their special competence and with regard for appropriate
balance.

This report has been reviewed by a group other than the authors
according to procedures approved by a Report Review Committee
consisting of members of the National Academy of Sciences, the
National Academy of Engineering, and the Institute of Medicine.

The views expressed in this report are those of the authors and do
not necessarily reflect the view of the committee, the Transportation
Research Board, the National Academy of Sciences, or the sponsors
of the project.

Contents

Design and Construction of Conventional Bridge Decks That Are Resistant to Spalling

Bruce F. McCollom, Bartlett and West, Lake Andes, South Dakota

An analysis was made to determine the length of time until spalling (induced by reinforcing steel corrosion) occurs and the relative cost for different conventional concrete bridge deck designs. Research results of others provided the data from which the time to corrosion was estimated. Three factors were considered: (a) frequency and rate of deicing salt application, (b) water-cement ratio of concrete, and (c) depth of concrete cover over reinforcing. The latter two were variables in deck design and construction. Decks with two combinations of water-cement ratio and clear cover each for two bridges were designed. From these, cost differences for labor and materials were determined. It was concluded that in Kansas conventional decks can be protected from spalling for a 50-year period. Also, life can be tripled for only a 2 percent cost increase by switching from decks with a 5-cm (2-in) cover and a water-cement ratio of 0.44 to decks with a 7.5-cm (3-in) cover and a water-cement ratio of 0.35.

Spalling of reinforced concrete bridge deck slabs is a serious and costly problem in Kansas, as it is in many other states. Much research on this problem has been and is being done. The type of spalling discussed in this paper is caused by corrosion of the reinforcing steel. Conventional bridge decks are those in which the only protection of the reinforcing steel from corrosion is provided by portland cement concrete. Further, conventional bridge decks are constructed in one course from a plastic concrete (5 to 10-cm or 2 to 4-in slump) that is placed, consolidated, finished, and cured by the usual methods.

RESEARCH RESULTS OF OTHERS

Although an ideal solution to the problem has not yet been defined, researchers have provided sufficient data for rational analysis of conventional bridge decks.

For conventional bridge decks, numerous researchers have shown that the length of time until corrosion, or spalling, is primarily a function of

1. The frequency and rate of deicing salt applications,

combined with the amount of time the deck is wet (the less time the better),

2. The water-cement ratio of the concrete (the lower the better), and

3. The depth of concrete cover over the reinforcing steel (the more the better).

It is assumed for the purpose of this discussion that the latter two items are variables but that the first item is fixed.

Quantitative evaluation of alternatives showed that better conventional bridge decks can and should be built. The two parameters that need to be quantified are (a) life of the bridge decks before serious spalling occurs and (b) cost (differential).

Numerous researchers have found that lowering the water-cement ratio or increasing depth of cover over the steel or doing both increases the life of bridge decks at a given frequency and rate of deicing salt applications. Therefore, three alternates were studied in which the water-cement ratio (w/c) and depth of cover were varied:

Alternate	w/c	Depth of Cover (cm)
A	0.35	7.6
B	0.44	5
C	0.49	2.5

Alternate C is typical of decks constructed from 1960 to 1965. The data that are most easily used to determine the years of life of alternates A and B are those of Beaton and Stratfull (1), Spellman and Stratfull (3, 4), and Clear and Hay (2). (Because their data were developed in U.S. customary units, SI units are not given for the variables in their equations or their figures.)

Analysis Based on Data of Beaton, Spellman, and Stratfull

Beaton and Stratfull (1) give the following equation:

$$R_t = \frac{10^{0.0442C} C^{0.717} S_i^{1.22} 1011}{K^{0.42} W_m^{1.17}} \qquad (1)$$

1

where

  R_t = estimated years to deterioration,
  C = sacks of cement per cubic yard of concrete,
  S_i = inches of concrete cover over reinforcing steel,
  K = chloride concentration in parts per million in environment, and
  W_m = total water contained in concrete mix as percentage of concrete volume (including that contained by aggregate).

Spellman and Stratfull (3) give the following equation:

$$D = (5.164C^{3.12})/W^{3.06} \qquad (2)$$

Figure 1. Effect of cement factor on time to active potential.

where

  D = average days to active potential,
  C = sacks of cement per cubic yard of concrete, and
  W = mixing water as percentage of concrete volume (not including that contained by aggregate).

(Depth of steel cover and chloride concentration were not variables in this research.)

Figure 1 is taken directly from Spellman and Stratfull (4) except that the water-cement ratio of the mixes has been added below the data points. The water-cement ratios were calculated from the mix proportion given in Table 1 of their paper (4).

One procedure used to estimate the life of Kansas bridge decks before serious spalling occurs is as follows.

1. Determine the values of S_i, C, W_m, and W:

Alternate	S_i	C	W_m	W
A	3	8.0	18.5	15.8
B	2	6.4	18.5	15.8
C	1	6.4	20.1	17.2

W_m values are based on the assumption of 890 kg/m^3 (1500 lb/yd^3) of coarse aggregate at 3 percent absorption.

2. Assume K to be constant.
3. Separate the effect of S_i from the effect of C and W_m in equation 1.

$$R_t \sim S_i^{1.22} \qquad (3)$$

$$R_t \sim (1.107C \; C^{0.717}/W^{1.17}) \qquad (4)$$

Figure 2. Spalling versus average chloride content.
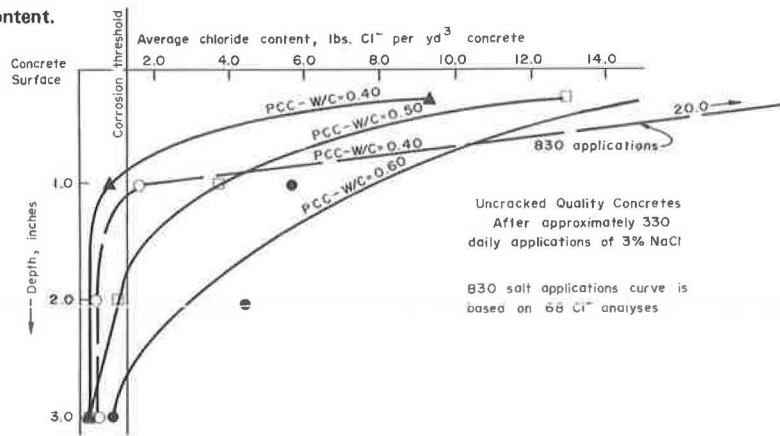
Figure 3. Spalling versus maximum chloride content.

Table 1. Cost increase by item.

Bridge	Size (m²)	Item	Difference in Quantity (kg)	Unit Cost ($/kg)	Increase ($)
1	608	Cement	41 355	0.03	1304
		Reinforcing	2 671	0.37	968
Total					2290
2	773	Cement	14 613	0.03	461
		Reinforcing	1 768	0.37	653
Total					1114

Note: 1 m² = 10.7 ft²; 1 kg = 2.2 lb.

Table 2. Cost increase per unit area and percentage.

Bridge	Average Cost/m² ($)	Item	Cost/m² ($)	Percent
1	1.95	7.5 versus 5 cm of cover	0.0186	1.0
		w/c = 0.35 versus 0.44	0.0139	0.7
Total			0.0325	1.7
2	2.79	7.5 versus 5 cm of cover	0.0056	0.2
		w/c = 0.35 versus 0.44	0.0073	0.3
Total			0.0129	0.5
Average			0.0232ᵃ	1.1

Note: 1 m² = 10.7 ft²; 1 cm = 0.39 in.

ᵃ $1.88/m².

4. Calculate the numerical values of the ratio of the length of time measures in equations 2, 3, and 4 and Figure 1 by using alternate C as the base.

5. Average the numerical ratio values produced by equations 2 and 4 and Figure 1 to yield a w/c effect value. Then multiply these values by the numerical ratio values obtained from equation 3 (the S_i effect value). The resulting combined values are as follows:

Alternate	S_i Effect	w/c Effect	Combined	Life (years)
A	3.82	2.60	9.9	99
B	2.33	1.33	3.1	31
C	1.00	1.00	1.0	10

6. Multiply the combined ratios by the life of alternate C, which is estimated to be 10 years. The as-built depth of cover of most decks constructed from 1960 to 1965 varies significantly from the specified minimum of 3.2 or 3.8 cm (1¼ or 1½ in). Variations in as-built water-cement ratio from the specified maximum of 0.44 or 0.49 liters/kg (5 or 5½ gal/sack) also are present. It is the author's judgment, based on observation of a number of these bridges on high traffic highways in northeastern Kansas, that 10 years is a reasonable estimate of the average time until serious spalling occurs for alternate C bridges. Serious spalling is defined here as spalling that receives maintenance in the form of patching. The estimated life of each alternate is given above.

Another procedure used to estimate life, with the same data, is as follows:

1. Calculate R_t values for each alternate by using equation 1 with K = 160 000 ppm (saturated solution).

2. Estimate the number of days during an average year in which a bare bridge deck in Kansas would be in a saturated condition. Based on average number of rain days of more than 0.25 cm (0.10 in) for the period 1941 to 1970, the number is estimated to be 50.

3. Multiply the R_t values from step 1 by 365/50.

These values are as follows:

Alternate	Life (years)
A	61
B	27
C	10

This method shows less difference between the alternates than the first method did because it is based only on equation 1, in which the effect of water-cement ratio is not so large as in equation 2 and Figure 1.

Analysis Based on Data of Clear and Hay

Figures 2 and 3 are taken directly from Clear and Hay (2) except that the results after 830 salt applications have been added. Clear and Hay state that 7 to 28 salt applications were required to induce rebar corrosion for a 2.5-cm (1-in) cover of concrete with a water-cement ratio of 0.50; this condition approximates alternate C. Interpolation by using Figure 2 indicates that rebar corrosion would not take place after 330 applications in alternate B or after 830 salt applications in alternate A under the test conditions. Clear and Hay's recommendations (dictated by interim findings, after 330 applications, of their research) were either (a) w/c = 0.40 concrete and 5 cm (2 in) of clear cover or (b) w/c = 0.50 concrete and 7.6 cm (3 in) of clear cover. They did not give a life expectancy in years for these combinations. One could infer, however, that alternate B would not be good enough and that alternate A would be better than necessary.

To convert the number of test applications to years until serious spalling occurs requires that the following factors be considered:

1. The amount of salt applied per year of bridge deck life versus a given number of test applications,

2. The effect of a given quantity of salt applied to a bridge deck under field conditions versus the same quantity applied in test applications,

3. The time lag between the start of corrosion and the time to serious spalling, and

4. The effect of not obtaining the specified maximum water-cement ratio or minimum depth of cover.

Based on estimates by maintenance personnel it is believed that Kansas bridge decks receive about 20 applications at 370 kg/2-lane km (1300 lb/2-lane mile) per year. In Clear and Hay's tests the top surface of the slabs was ponded to a depth of 0.5 cm (¹⁄₁₆ in) with a 3 percent solution of sodium chloride each afternoon. The slabs were flushed monthly with potable water. Most of the slabs were exposed to precipitation. From this information it was calculated that one test application places approximately the same amount of salt per unit area on the test slabs as one application by maintenance personnel does on Kansas bridge decks. Therefore, based on the quantity of salt, 20 test applications equal one year. Other equivalencies are as follows:

Test Applications (2)	Years of Salting Kansas Bridge Decks
830	41
330	16
7 to 28	½ to 1½

Clear and Hay discuss the difference between their procedure, which results in a wet-dry surface within a single day, and one involving continuous soaking. They base their discussion on the difference between those slabs protected from precipitation and those not protected. Preventing the natural washing action of precipitation and

the evaporating action of the sun had a definite adverse effect. They state that a continuous soaking procedure is a more stringent (although not necessarily superior) test. One could infer from their discussion that test applications are more stringent than field applications.

Spellman and Stratfull (3) found that their test specimens reached active potential in about three-quarters of the time it took for visible evidence of reinforcing steel corrosion (cracking). This time lag is probably not independent of depth of cover. It would logically be greater with more cover. The cover provided in their tests was a minimum of 2 cm ($\frac{7}{8}$ in).

In real bridge decks there is a finite probability that the specified maximum water-cement ratio will be exceeded and that the specified minimum depth of cover will not be obtained. The factors determining this probability are numerous and difficult to evaluate. In this analysis it was assumed that the in-place deck would be within the following tolerances essentially all the time: 0.03 water-cement ratio and 0.6-cm ($\frac{1}{4}$-in) depth of cover.

Having considered the Clear and Hay data in the light of the four factors just discussed, it was felt that they agree with the length of life estimated based on the Beaton, Spellman, and Stratfull data.

COMPARISON OF ALTERNATES

Based on the preceding analysis of the research results (1, 2, 3, 4), two conclusions were reached:

1. For the frequency and rate of salt applications used in Kansas, spalling should be eliminated (for practical purposes) within a 50-year design life if alternate A is adopted.
2. Alternate A should last three times as long as current alternate B.

To determine the difference in cost of alternates A and B, two bridges were designed for each of the alternates A and B. Bridge 1 is a 15-m, 20-m, 15-m (48-ft, 64-ft, 48-ft) continuous reinforced concrete haunched slab with a 12-m (40-ft) roadway, and bridge 2 is a 16-m, 150-m, 16-m (54-ft, 93-ft, 54-ft) continuous welded deck girder with an 8.5-m (28-ft) roadway. These bridges were chosen because they are typical of Kansas bridges and represent extremes as far as the effect of the proposed changes on cost. AASHTO specifications (including 1974 interims) were used. The exterior dimensions of the alternates of each structure are identical and are taken from actual bridges recently built in Kansas. The bridges as originally designed and built were of the alternate B type: They used grade 40 reinforcing and working stress design method. So that the alternates could be compared under conditions more typical of those currently used, both alternates A and B were redesigned by using grade 60 steel and the load factor design method. The depth of cover for alternate A was increased to a 7.6-cm (3-in) minimum by lowering the top reinforcing steel 2.5 cm (1 in) relative to alternate B. In the design, alternate A had fc' = 34.5 MPa (5000 psi) and alternate B had fc' = 27.6 MPa (4000 psi). The cost of alternate C is irrelevant at this point inasmuch as Kansas and most other states have already abandoned it.

In the cost comparison, labor and materials were considered separately. The significant material costs that differ between alternates A and B are given in Table 1. Labor costs do not differ significantly between alternates A and B. The formwork is identical, and the concrete mixes are the same slump; the only difference in reinforcing is that the top steel (negative moment) is one bar size larger in alternate A. The increased steel

costs are essentially due to the additional 2.5 cm (1 in) of cover, and the increased cement costs essentially are due to the lower water-cement ratio. Table 2 gives these costs.

IMPLEMENTATION

This system of bridge deck protection does not require any major adjustments in design or construction practices, at least in Kansas; hence, it is called conventional. It is only necessary to

1. Design bridges with a 7.5-cm (3-in) clear top cover instead of 5 cm (2 in); the thickness of decks should be the same.
2. Change the concrete specifications to a water-cement ratio of 0.35 instead of 0.44. Also, the specified minimum cement factors must be increased 25 to 30 percent so that mixes of the same slumps as currently being used will be obtained.

Contractors for the State Highway Commission of Kansas have constructed, by conventional methods, several bridge decks amounting to in excess of 7645 m³ (10 000 yd³) of the new concrete mixes. Several types of aggregate have been used, the weather has varied from winter to summer, and we have used both transit mix and remote central mix concrete. The in-place density and water-cement ratio have been checked extensively. This has shown that good consolidation (less than 2 percent entrapped air) is no more difficult to achieve with the new mixes than with the old. Likewise, it was shown that the probability of exceeding the specified maximum water-cement ratio is no greater for the new mixes than for the old. The only difficulty encountered that could be attributed to the new mix was in finishing. The top surface of the new mix does lose its wetness faster and is therefore more difficult to finish (at the same initial slump and under the same weather conditions). Two of the contractors adjusted their methods (no additional work force) and were able to achieve good finishes. The quality of the finish obtained by the third contractor was less than desirable on the new mix. Test cylinder strengths for the new mix after 28 days have averaged about 44.1 MPa (6400 lbf/in²) (specified air content 6 percent ± 2).

Kansas is now designing most new decks with 7.5 cm (3 in) of top cover, but none of these has been constructed yet.

CONCLUSIONS

1. In Kansas conventional reinforced concrete bridge decks can be protected from spalling for a 50-year design life by providing a 7.5-cm (3-in) minimum cover over the reinforcing steel and specifying concrete with a maximum water-cement ratio of 0.35. This can be done at a cost increase of 2 percent or less over the current practice of providing 5-cm (2-in) minimum cover and specifying a maximum water-cement ratio of 0.44.
2. For other conditions of deicing salt exposure, adding concrete cover and reducing water cement ratio (within practical limits) are a cost-effective way of increasing the life of conventional bridge decks. When bridge decks with 7.5 cm (3 in) of cover and a water-cement ratio of 0.35 are compared with decks with 5 cm (2 in) of cover and a water-cement ratio of 0.44, it is found that life can be tripled for a 2 percent, or less, increase in construction cost.

REFERENCES

1. J. L. Beaton and R. F. Stratfull. Environmental

Influence of Corrosion of Reinforcing in Concrete Bridge Substructures. HRB, Highway Research Record 14, 1963, pp. 60-78.

2. K. C. Clear and R. E. Hay. Time-to-Corrosion of Reinforcing Steel in Concrete Slabs. Offices of Research and Development, Federal Highway Administration, Rept. FHWA-RD-73-32, April 1973.

3. D. L. Spellman and R. F. Stratfull. Laboratory Corrosion Test of Steel in Concrete. Materials and Research Department, California Division of Highways, Research Rept. M&R 635116-3, Sept. 1968.

4. D. L. Spellman and R. F. Stratfull. Concrete Variables and Corrosion Testing. Materials and Research Department, California Division of Highways, Research Rept. M&R HRB 635116-6, Jan. 1972.

Electrochemical Removal of Chlorides From Concrete Bridge Decks

John E. Slater and David R. Lankard, Battelle Columbus Laboratories
Peter J. Moreland,* ICI Ltd., Runcorn, England

The spalling of concrete bridge decks is largely due to corrosion of the top layer of reinforcing steel. This corrosion is induced by chloride ions, which enter the concrete when deicing salt is applied to the deck surface. The construction of many new bridge decks is based on techniques that either prevent penetration of chloride or ensure resistance to chloride-induced rebar corrosion. However, the many salt-contaminated decks that now exist demand attention. Besides costly patching of spalls, only cathodic protection of the steel is currently available as a remedial measure. This paper presents the results of an investigation of an electromigration method for removing chloride from contaminated concrete decks, which prevents rebar corrosion. In this method, the chloride ion moves through and out of the concrete under an electric field applied between the rebar (cathode) and a surface anode. A preliminary laboratory investigation demonstrated the viability of the technique and identified optimum voltage (100 Vdc), treatment time (12 to 24 h), anode material (platinized titanium), surface electrolyte, and chloride fixant. A field trial on a 3 by 6.1-m (10 by 20-ft) section of chloride-contaminated bridge deck was conducted in which laboratory-optimized parameters were used. Under the best conditions, 90 percent of the chloride was removed from the concrete above the rebar; 88 percent was removed from the concrete immediately adjacent to the rebar. Potential measurements have shown that the previously actively corroding rebar became passive after treatment.

Deterioration of portland cement concrete (PCC) bridge decks is a serious problem in many parts of the United States and has resulted in high maintenance costs to keep the decks in a safe and serviceable condition. Although deck deterioration can take several forms, such as scaling, cracking, or spalling, during the last 10 years or so spalling has become the major contributor to bridge deck deterioration. The increased frequency of spalling of PCC bridge decks appears to be directly related to the increased use of deicing salts (primarily NaCl and $CaCl_2$) in recent years. Chloride ions accelerate the rate of corrosion of reinforcing steel in the concrete. The formation of corrosion products on the steel results in a buildup of stresses in the concrete, which is ultimately manifested as spalling of concrete on the deck surface.

If we assume that deicing salts will continue to be used during the foreseeable future, then remedial steps must be taken if the problem is not to worsen. Recognizing this, the Federal Highway Administration and other highway agencies have initiated research and field programs to investigate the feasibility of a number of potential solutions to the problem, including (a) polymer impregnation of bridge decks, (b) cathodic protection of reinforcing steel, (c) protection of the reinforcing steel through the use of various coatings, (d) application of impermeable membranes or coatings to the concrete, and (e) sophisticated rehabilitation procedures involving removal of chloride-contaminated concrete to below-steel levels and subsequent protection of the steel (e.g., with epoxy coatings) before new concrete is placed. One means of providing extended service life to existing chloride-contaminated concrete decks that has not been adequately researched is removal of the chloride from the concrete. This paper describes that process.

A two-phase program was begun at Battelle Columbus Laboratories during July 1973 and was completed in September 1975. The initial laboratory phase was intended to evaluate the feasibility of the removal concept and to identify suitable procedures and equipment. This work was followed by a field phase in which the treatment concept developed in the laboratory was successfully applied to an existing bridge deck in Ohio.

LABORATORY EXPERIMENTS

The electrochemical removal concept involves the migration of chloride ions in concrete under the influence of an electrical potential gradient through the bridge deck concrete and into an electrolyte contained above it. The potential gradient is produced by applying a direct current source between the reinforcing steel and an electrode contained in the electrolyte above the bridge deck. An ion exchange resin contained in the electrolyte captures the chloride ions before they reach the anode. Thus, the evolution of chlorine gas is prevented, and corrosion of

*When this paper was written, Mr. Moreland was with Battelle Columbus Laboratories.

the anode is minimized.

The laboratory phase of the program involved studies of 76 by 152-mm (3 by 6-in) concrete cylinders and 117 by 152 by 23-mm (46 by 60 by 9 in) concrete slabs. The specimens were prepared with concretes in which pre-selected quantities of chloride had been added to the mix water. The initial work was directed at identifying

1. A suitable anode material,
2. A suitable ion exchange resin, and
3. A suitable electrolyte.

Then the effect of concrete and process variables on the electrochemical removal of chloride ion was investigated. The variables studied included

1. The magnitude of the applied electrical potential gradient,
2. The duration of treatment, and
3. The initial chloride content of the concrete.

Pretreatment and posttreatment measurements of chloride content at various levels in the concrete were made by using the techniques described by Berman (1).

The initial laboratory work showed that calcium hydroxide solution (0.1 normal concentration) was suitable surface electrolyte, platinized titanium was the optimum anode material, and Dowex 2-X8 was a suitable anion-exchange resin to capture chloride as it emerged from the concrete.

The amount of chloride removed for a given treatment time increased as the applied dc voltage increased. At 100 Vdc (the maximum voltage used in the program), treatment times of 16 to 48 h resulted in significant reductions in the chloride content depending on the initial chloride level and distribution. Treatment at 50 V was not sufficient to provide significant levels of chloride removal within a reasonable time period. The treatment time necessary for chloride removal increased as the initial chloride content of the concrete was increased. However, for concrete with a high chloride content (4.15 kg/m³ or 7 lb/yd³), substantial reductions in total chloride were achieved in a 24-h period.

Although the efficiency of the chloride removal technique was quite low, it did significantly reduce the chloride content of the concrete below what is now considered a threshold value for the corrosion of the reinforcing steel. Attempts to improve the efficiency of the treatment by changing the composition of the electrolyte solution were not successful.

The temperature of the concrete (laboratory specimens) during electromigration treatment at 100 V increased from 24 to about 52 C (75 to 125 F). The elevated temperature exposure had no obvious adverse effect on the integrity of the concrete. For the laboratory specimens, there appeared to be a chloride removal threshold. This residual chloride content was 0.02 percent or about 0.47 kg/m³ (0.8 lb/yd³) of concrete. A portion or all of this residual chloride is present in an insoluble form and hence is not amenable to easy removal by the electrochemical technique.

Given the positive results obtained in the initial experiments, the electrochemical removal investigation was continued on the large simulated bridge deck slabs. The experimental arrangement used is shown in Figure 1. The slabs were constructed in several lifts, with concrete containing Cl⁻ (added as NaCl to the mix water) above the top rebar mat. Concrete composition and rebar placement and size were the same as in the actual bridge deck selected for study.

Electrical power for the large slab work was supplied by a 5-kW portable generator. Voltage was controlled by a variable transformer, and a full-wave rectifier was used to convert the alternating current to direct current. Electrical connections were made to the reinforcing steel in the concrete and to a 61 by 76-cm (24 by 30-in) platinized titanium electrode (in expanded metal form) on the upper surface of the slab. A wooden dike, sealed with silicone, served to contain the ion exchange resin and the electrolyte solution (0.1 N calcium hydroxide solution).

The work on the large slabs established that significant quantities of chloride ion can be removed from thick concrete slabs by using the electrochemical treatment within a reasonable time period. However, current densities of at least 0.46 A/m² (5 A/ft²) were required for effective electrochemical treatment. Treatment times of 24 h appeared adequate to achieve a significant reduction in chloride ion content. The treatment can be either continuous or intermittent to achieve the same chloride extraction.

The removal of chloride ion was greatest in the concrete overlying the cathode (rebar). Significant but lesser reductions in chloride content were also achieved in concrete adjacent to the cathode areas. It was clear that the problems of scaling the treatment for application to large slabs would be minimal.

BRIDGE DECK EXPERIMENTS

For the electrochemical chloride removal experiments, the bridge deck had to contain substantial quantities of chloride but exhibit no evidence of surface spalling. With the cooperation of the Ohio Department of Transportation, a number of candidate bridges were identified and visited in fall of 1973. The bridges were examined visually for spalling, and electrochemical potential scans were taken to assess corrosion activity of the reinforcing steel.

The bridge selected for study was an 8-year-old, two-lane, steel girder bridge on the southbound lane of US-33 near Marysville, Ohio. The deck was completely free of surface spalls. As a matter of interest, a similar deck less than 100 m (330 ft) away, constructed at the same time by the same contractor, had about 60 percent of the surface repaired due to spalling. A 3 by 6.1-m (10 by 20-ft) area of the bridge deck on the west side lane was chosen for treatment.

Pretreatment Measurements

Pretreatment characterization data were obtained in the form of electrochemical potential scans, chloride analyses, delamination detection, and linear polarization electrode readings.

Potential Scans

Before the selected area was electrochemically treated, a number of potential scans were made of the deck (in November 1973, August 1974, and April 1975). Measurements were made on a 0.3-m (1-ft) grid in general accordance with the technique described by Stratfull (2) by using Cu/CuSO₄ as the reference cell. It has been established that steel showing a potential more negative than about -0.35 V is undergoing active corrosion.

During the 17-month measurement period, the active region of the treatment area increased from 18 to 55 percent. Figure 2 shows the potential profile of the selected section of the deck just prior to the electrochemical treatment (in April 1975).

Chloride Analyses

A number of cores were removed from the 3 by 6.1-m

(10 by 20-ft) area of the deck during the 18-month period preceding the electrochemical treatment. The location of the cores on the deck is shown in Figure 3. The average chloride content as a function of depth in the deck is given in Table 1.

An increase in chloride content at all levels in the deck was noted during the 17-month observation period. Approximate increases were as follows:

Deck Depth (mm)	Content (kg/m³)
0 to 25	2.4
25 to 51	1.2
51 to 76	0.3

Just before electrochemical treatment, the chloride ion content on the top 25 mm (1 in) of deck was more than 8.9 kg/m^3 (15 lb/yd³) while at 25 to 51 mm (1 to 2 in) the content was 2.4 to 5.9 kg/m^3 (4 to 10 lb/yd³).

Because of the rather large variation between specimens in measured chloride observed when duplicate cores were taken, we cannot speculate on the differences in chloride content between the high and low corrosion

Figure 1. Experimental setup for the electrochemical removal of chloride ion from simulated bridge deck slabs.
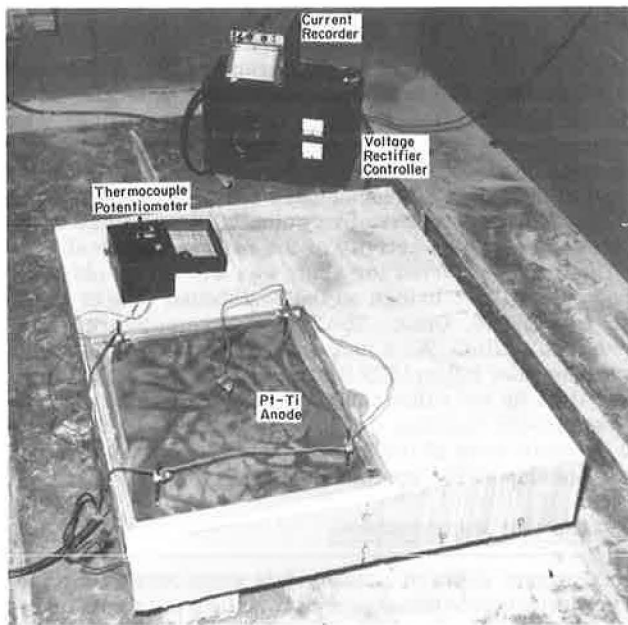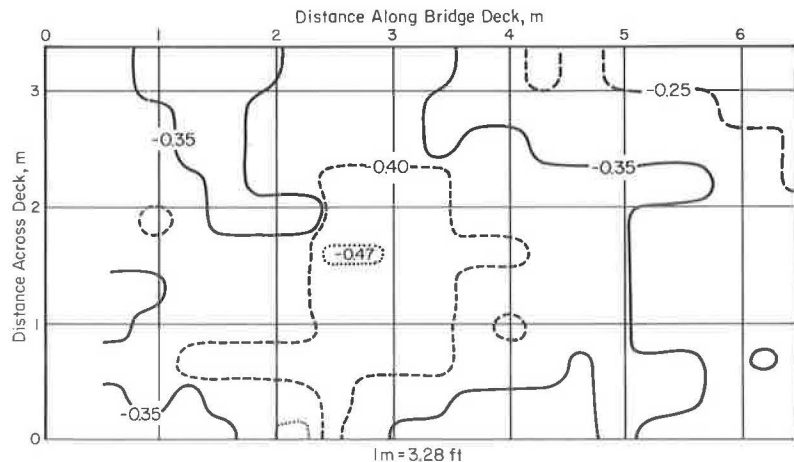

Figure 2. Pretreatment potential scan of bridge deck area (April 1975).

activity areas of the deck.

Delamination Detection

In April 1975, before the electrochemical treatment, maintenance personnel of the Ohio Department of Transportation examined the selected area of the bridge deck for delamination by using a sounding rod. There was no evidence of any delamination despite the significant areas of high corrosion activity.

Electrochemical Treatment of the Deck

After deck characteristics before treatment were identified, electrochemical experiments were begun on April 7, 1975. The total area treated was 18.6 m^2 (200 ft²). The treatment area was broken down into five 3.4-m^2 (40-ft²) sections (Figure 3) that were treated individually.

Wood ponding frames 50 mm (2 in) deep, which contained four 0.76 by 1.2-m (2.5 by 4-ft) compartments, were used to hold the ion exchange resins. After the deck was swept, the frames were sealed to the bridge deck surface with construction caulking compound. Weights were placed on the frames to prevent movement and to aid in maintaining a leakproof seal.

The electrolyte solution of saturated $Ca(OH)_2$ was placed in the ponding frame to a depth of 12 to 25 mm ($^1/_2$ to 1 in). Dowex 2-X8 ion exchange resin in the OH^- form was slurried into the electrolyte to a depth of 6 to 9 mm ($^1/_4$ to $^3/_8$ in). It was rather difficult to maintain a homogeneous dispersion of the resin in the electrolyte.

A 0.7 by 1.2-m (2.25 by 4-ft) platinized titanium anode was then placed in each of the four compartments within the ponding frame. The anodes were supported about 12 mm (0.5 in) above the bridge deck by small wood blocks. The anodes were kept covered with electrolyte solution at all times. Figure 4 shows the compartmentalized ponding frames and one of the generators used in the experiment. The frame in the background contains the electrolyte, resin, and anodes.

Electrical contact was made with the top layer of the reinforcing steel in the treatment area through 6 by 25 by 100-mm ($^1/_4$ by 1 by 8-in) mild steel studs that were arc welded to the rebars. The rebars were exposed by coring along the berm side of the treatment area. Direct current power leads were attached to the rebar contact studs and to vertical studs that were welded to the titanium anodes in each compartment. Electrical power was supplied by two trailer-mounted, gasoline-driven, direct current generators. A schematic of the experimental technique is shown in Figure 5.

Figure 3. Location of cores taken from treatment area.

CORE NUMBERS | DATE TAKEN
H-0, L-0 | Nov. 1973
H-1, H-2, L-1, L-2 | Aug. 15, 1974
7, 8, 9, 10 | Apr. 7, 1975, before electrochemical treatment
12 through 26 | Apr. 14, 1975, after electrochemical treatment
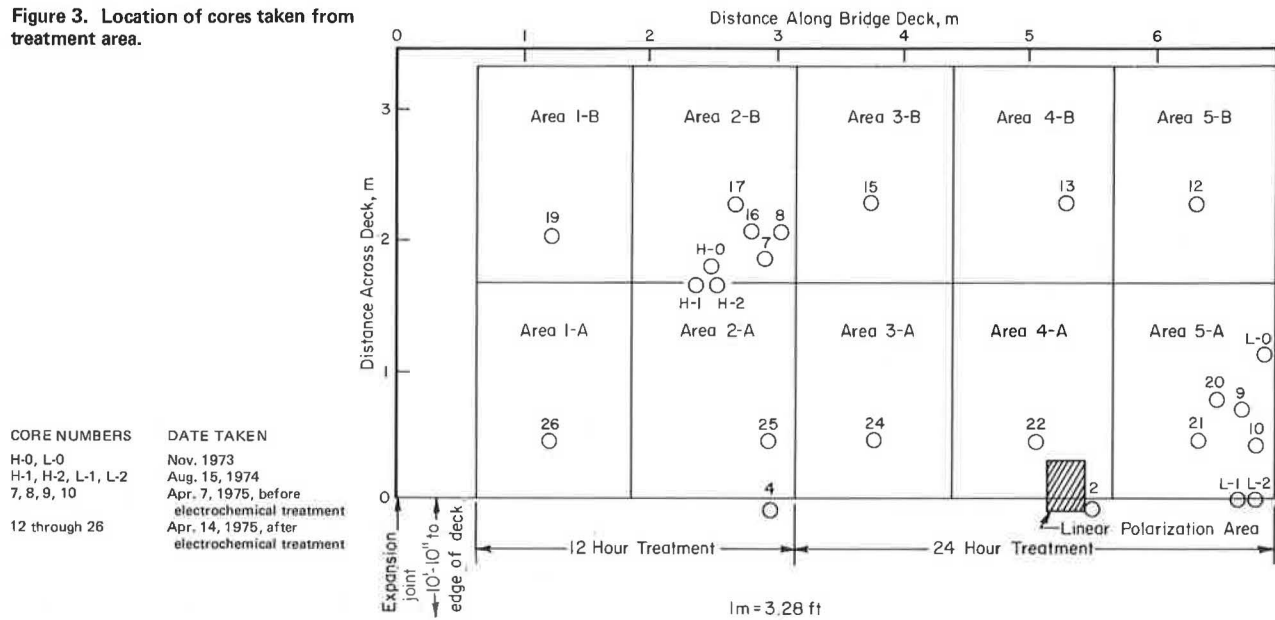
Table 1. Average chloride content of cores taken from bridge deck from November 1973 to April 1975 (before electrochemical treatment).

| Deck Depth (mm) | Chloride Ion Content[a] | | | | | |
| | Cores Taken on November 1973 | | Cores Taken on August 1974 | | Cores Taken on April 7, 1975 | |
	Percent	Kilograms per Cubic Meter	Percent	Kilograms per Cubic Meter	Percent	Kilograms per Cubic Meter
0 to 25	0.31	6.9	0.40	8.9	0.41	9.2
25 to 51	0.16	3.6	0.23	5.2	0.20	4.5
51 to 76	0.05	1.1	0.06	1.4	0.06	1.4
76 to 102	0.01	0.2	0.01	0.2	0.01	0.2
102 to 127	0.01	0.2	0.01	0.2	—	—

Note: 1 mm = 0.039 in; 1 kg/m³ = 1.69 lb/yd³.

[a]Based on dry concrete weight of 2242 kg/m³ (140 lb/ft³).

Figure 4. Chloride extraction hardware in position on bridge deck.
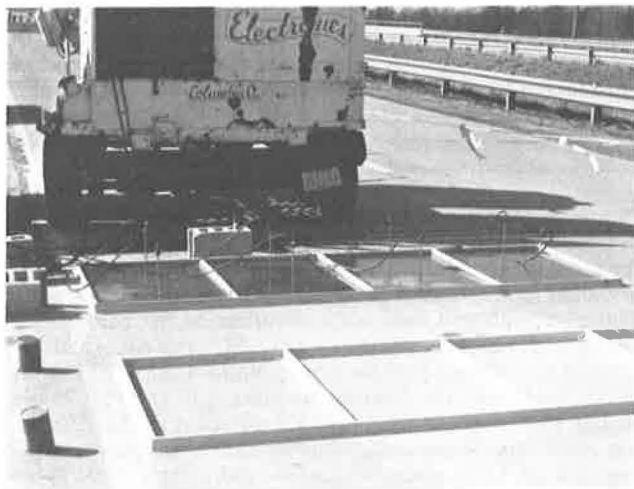
Each of the five treatment areas (Figure 3) was served by 2 dc generators, one generator supplying power to a 1.7-m² (20-ft²) area. In all cases, effort was made to maintain generator output at the maximum rated capacity of 120 V. In practice the voltage generally fluctuated between 100 and 120 V. Areas 1 and 2 were treated for 12 h, and areas 3, 4, and 5 were treated for 24 h.

Figure 5. Experimental technique used for electrochemical removal of chloride ions from a PCC bridge deck.
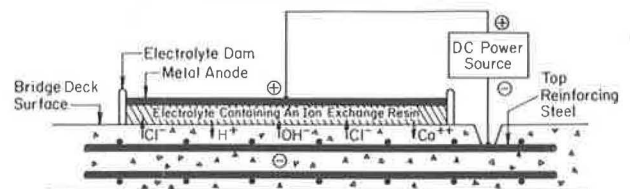
The ion exchange resin was omitted in the treatment of area 4. This resulted in a noticeable odor of free chlorine above the electrolyte ponds during the treatment period.

The temperature of the electrolyte ponds was measured during treatment periods with a thermometer. In addition, temperature of the concrete deck was measured by use of thermocouples inserted into small holes drilled into the concrete. Pavement temperature was monitored just above a rebar and about 3.8 cm (1½ in) below the concrete surface, midway between rebars. Two-person crews provided full-time monitoring of the experiments throughout the treatment period. Data were obtained on applied current and voltage and on slab and electrolyte temperature, as a function of treatment time.

After the electrochemical treatment, the electrolyte-resin solution was pumped off, and 7.6-cm (3-in) core samples taken from the treated area. The resin was returned to the laboratory for regeneration.

Table 2. Average chloride ion content of bridge deck before and after electrochemical treatment.

Deck Depth (mm)	Before Treatment		After Treatment				Average Cl⁻ Removed (%)	
			12-H		24-H			
	Percent	Kilograms per Cubic Meter	Percent	Kilograms per Cubic Meter	Percent	Kilograms per Cubic Meter	12 H	24 H
0 to 25	0.409	9.2	0.295	6.6	0.194	4.3	28	53
25 to 51	0.201	4.5	0.093	2.1	0.059	1.3	54	71
51 to 76	0.054	1.2	0.029	0.7	0.019	0.4	46	67
76 to 102	0.013	0.3	0.012	0.2	0.012	0.2	Nil	Nil

Note: 1 mm = 0.039 in; 1 kg/m³ = 1.69 lb/yd³.

Table 3. Amount of chloride removed from bridge deck.

Deck Depth (mm)	Treatment Area	Duration of Treatment (h)	Maximum Current (A)	Charge Passed[a]	Chloride Removed (%)
0 to 25	1A	12	47	54	44
	1B	12	45	56	36
	2A	12	100	100	29
	2B	12	50	54	15
	3A	24	96	70	89
	3B	24	87	70	91
	4A	24	35	—	19
	4B	24	28	26	24
	5A	24	53	45	62
	5B	24	30	13	21
25 to 51	1A	12	47	54	68
	1B	12	45	56	69
	2A	12	100	100	66
	2B	12	50	54	32
	3A	24	96	70	93
	3B	24	87	70	90
	4A	24	35	—	51
	4B	24	28	26	51
	5A	24	53	45	75
	5B	24	30	13	58

Note: 1 mm = 0.039 in.

[a]Calculated as the area under the current/time curve divided by treatment time. 2A arbitrarily assigned value of 100.

No major problems were experienced during the experiment. A careful posttreatment visual examination revealed that no cracks in the concrete occurred as a result of the treatment.

Posttreatment Measurements

The results of the electrochemical treatment were monitored through posttreatment measurements of chloride content, electrochemical potential measurements, and linear polarization (LP) electrode measurements.

Chloride Analyses

Posttreatment cores 7.6 cm (3 in) in diameter were taken from the treated areas (Figure 3). These were cores 12, 13, 15, 16, 17, 19, 20, 21, 22, 24, 25, and 26. Cores 16 and 20 were taken 12.7 cm (5 in) deep. All other cores were taken only to the level of the top reinforcing steel (about 50 mm or 2 in). Most of the cores were then sectioned into 25-mm-thick (1-in) slices although cores 15, 16, and 17 were sectioned in 12-mm (½-in) slices to permit a better discrimination of the chloride concentration profile.

In several cases, the concrete immediately adjacent to the reinforcing rod (about 6 mm or ¼ in in radius) was analyzed separately.

Table 2 gives a summary of chloride analyses before and after treatment. Although there was some variation in measured chloride from specimen to specimen, a reasonable assessment of effect of treatment param-

eters on chloride removal could be made.

Deck Depth of 0 to 25 mm (0 to 1 In)

The average pretreatment chloride ion content in the top 25 mm of the treated area of the deck was 0.409 percent (9.2 kg/m³ or 15.5 lb/yd³) as measured on cores 7, 8, 9, and 10. For an initial chloride content of 0.409 percent, the reduction in chloride as affected by the treatments is given in Table 3 for the various treatment areas. The average chloride removed in the top 25 mm (1 in) was 31 percent in 12 h and 51 percent in 24 h.

A general correlation existed between maximum current and charge passed and the amount of chloride removed. Certain treatment areas did not conform to this generalization (areas 2A and 2B).

Deck Depth of 25 to 51 mm (1 to 2 In)

The average chloride ion content at a deck depth of 25 to 51 mm (1 to 2 in) in the treated area was 0.201 percent (4.5 kg/m³ or 7.6 lb/yd³) as measured in cores 7 and 9. For a pretreatment value of 0.201 percent, the reduction in chloride in the 25 to 51-mm concrete level as affected by the treatment is also given in Table 3.

As for the first 25 mm, there was a general correlation between maximum current and charge passed and the amount of chloride removed. The average chloride removed at a depth of 25 to 51 mm (1 to 2 in) was 59 percent in 12 h and 70 percent in 24 h. A comparison of the data for the two depths shows that a significantly greater percentage of chloride was removed at the 25 to 51-mm level than at the 0 to 25-mm level.

Concrete Adjacent to Reinforcing Steel

Chloride analyses were run on the concrete enclosing the reinforcing steel (about 6 mm or ¼ in in radius) both before and after treatment. The pretreatment data were obtained from cores 2 and 4 taken from the berm section. The posttreatment data were obtained on the remnants of cores 26 (area 1A), 25 (area 2A), 22 (area 4A), and 21 (area 5A). These results are given in Table 4. For the cores analyzed, the average reduction in chloride content in the concrete immediately adjacent to the reinforcing steel was 79 percent. Significantly, the posttreatment cores were from some areas that did not show exceptionally high chloride extraction at the 0 to 25 and 25 to 51-mm (0 to 1 and 1 to 2-in) levels (Table 3).

Potential Scans

Potential scans were taken on the deck 24 h after the final section was treated and then 1 week, 1 month, and 3 months after treatment. The potential scan after 24 h (Figure 6) shows the effects of treatment on the potential. The Roman numerals show chronological order of treatment. The first section treated (5 days before the

scan) exhibits passive potentials. This is in direct contrast to the active values shown by this section before treatment. The second section treated shows a range of potentials from extremely active to moderately passive. This indicates that the extreme cathodic polarization applied to the steel during treatments can take days to decay. Sections III and IV show similar effects. The last section treated, section V, shows potentials well within the cathodic protection range (more active than -0.85 V) for steel.

The scan obtained 1 week after treatment is shown in Figure 7. The obvious difference is the decay of the cathodic polarization in sections II, IV, and V and the maintenance of passive potentials in these areas. The exception to this is the band of polarization along the midsection of treatment sections II and IV.

The results from the 1-month scan are shown in Figure 8. At this time, the vast majority of the deck exhibited a potential well within the passive range: No readings on the treated surface were more active than -0.30 V. The most passive potentials were exhibited by steel in treatment sections II and V.

Table 4. Chloride content of bridge deck concrete at rebar interface before and after electrochemical treatment.

Treatment Area	Before Treatment		12-H Treatment		24-H Treatment	
	Percent	Kilograms per Cubic Meter	Percent	Kilograms per Cubic Meter	Percent	Kilograms per Cubic Meter
Berm	0.091	2.0	—	—	—	—
Berm	0.058	1.3	—	—	—	—
1A	—	—	0.013	0.3	—	—
2A	—	—	0.012	0.2	—	—
4A	—	—	—	—	0.018	0.4
5A	—	—	—	—	0.021	0.5
Average	0.075	1.7	0.013	0.3	0.020	0.5

Note: Sample about 6-mm-thick ($\frac{1}{4}$-in) concrete surrounding upper half of rebar. 1 kg/m³ = 1.69 lb/yd³.

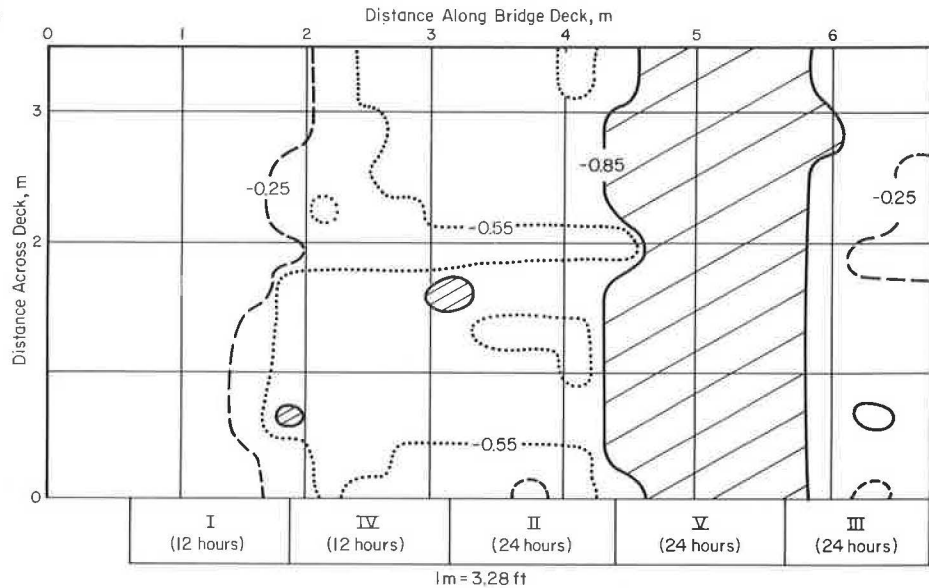**Figure 6. Potential scan of treatment area 24 h after final treatment.**

1 m = 3.28 ft

Figure 7. Potential scan of bridge deck 1 week after final treatment.
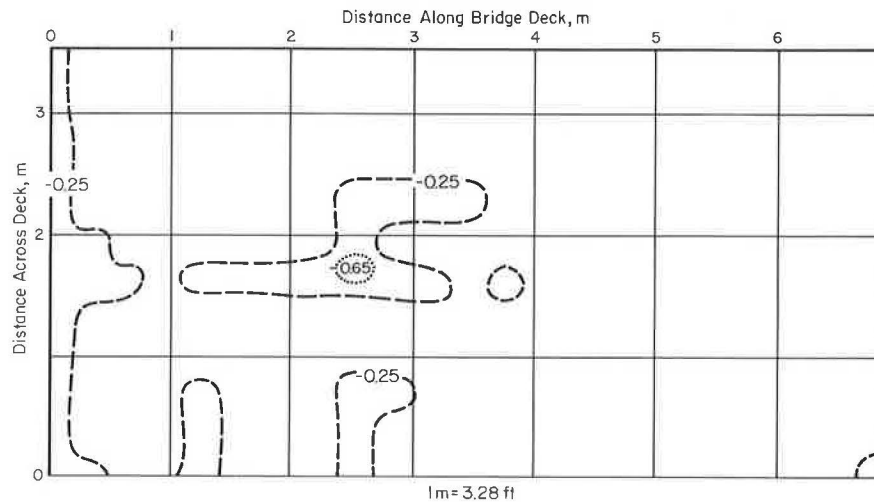

1 m = 3.28 ft

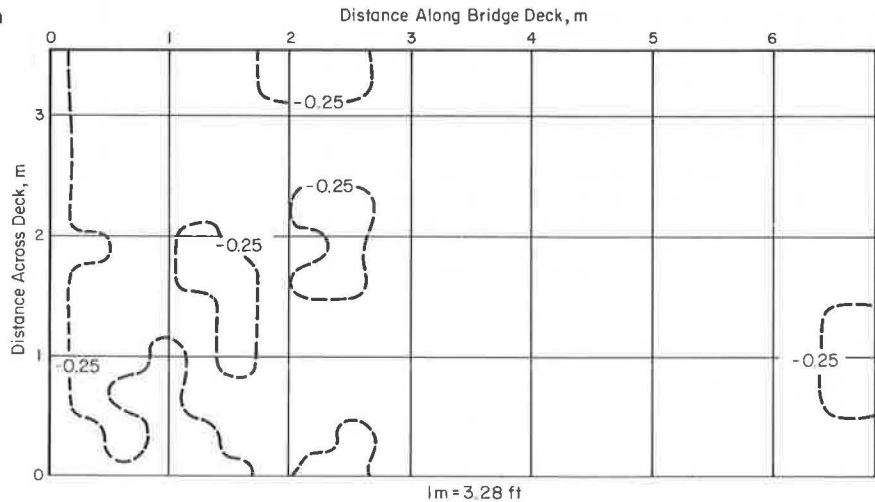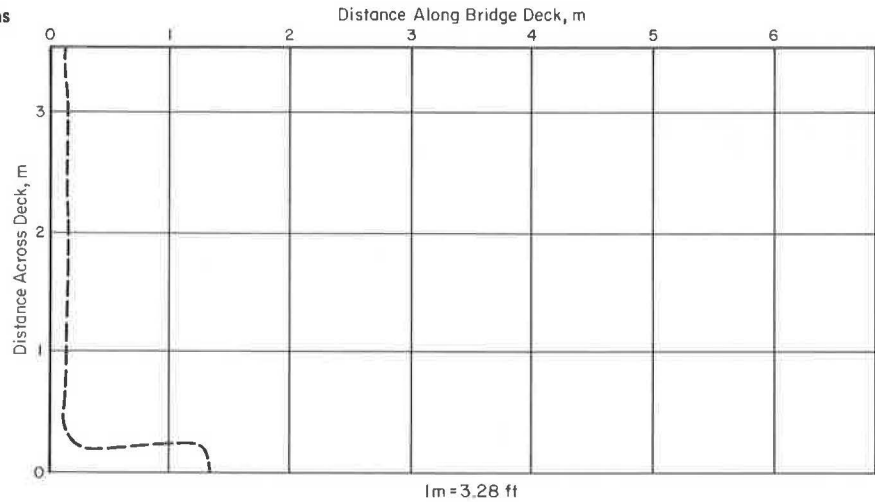Figure 8. Potential scan of bridge deck 1 month after treatment.

Figure 9. Potential scan of bridge deck 3 months after treatment.

The 3-month scan (Figure 9) shows that potentials of the treated area were within the passive range. A check of potentials outside the area, however, revealed that active corrosion of the steel was continuing in such areas, possibly enhanced by the large adjacent passive area generated by the electrochemical treatment.

Linear Polarization Readings

Corrosion rates of the reinforcing steel as deduced from LP electrodes were measured prior to treatment and 1 week, 1 month, and 3 months after treatment. The results were as follows:

Date	Corrosion Rate (m/year)	Date	Corrosion Rate (m/year)
April 7	0.5	May 23	0.6
	0.5		0.5
April 21	1.1	July 15	0.4
	0.8		0.3

Thus, 1 week after treatment, the linear polarization electrodes were showing a corrosion rate almost twice as high as before treatment. However, 1 month after treatment, the corrosion rate had dropped to that observed before treatment.

The reasons for this unexpected behavior may be the destruction of the passive oxide film on the rebar by the cathodic treatment and its subsequent growth after treatment.

DISCUSSION OF RESULTS

The principle of using electromigration methods to extract chloride ion from salt-containing reinforced concrete has been shown effective in laboratory experiments and on a bridge deck in the field. For the concrete situated vertically above the reinforcing steel, an average of 31 percent of the chloride was removed after 12 h and 51 percent after 24 h of treatment. For concrete at the 25 to 51-mm (1 to 2-in) level, these values were 59 percent and 71 percent. Peak extraction values of more than 90 percent were obtained.

Because the chloride content of the concrete immediately adjacent to the reinforcing steel is the determining factor in initiation and continuation of corrosion, the reduction in chloride level at this location to below 0.02 percent is extremely significant. The literature suggests that this is close to or below the lower limit for initiation of corrosion (3, 4). The potential scan before the treatment showed that more than 55 percent of the treatment area rebar was actively corroding. Posttreatment scans (up to 3 months after treatment) showed that all active corrosion had ceased, and the rebar exhibited passive

potentials. Linear polarization results also support this conclusion. The combination of cathodic polarization of the rebar and chloride removal probably led to this effect. Cathodic polarization effectively stopped corrosion during treatment, and the level of chloride after treatment was insufficient to reinitiate the corrosion.

The long-term effect of the extraction procedure on rebar corrosion is not yet known. After treatment of a full-size bridge deck, the concrete surface would be waterproofed to ensure that further application of de-icing salt would not cause chloride penetration. (This was not practicable in the field experiment described here.) However, because not all the chloride is removed, there may be a tendency for chloride redistribution by diffusion under concentration gradients. The effect of this process on long-term initiation of rebar corrosion is unclear at this time.

The hardware used in the field experiment is probably not wholly suited for scale-up to treatment of a full-size deck, particularly if that deck shows sharp transverse or longitudinal gradients. Further development of surface electrolyte containment procedures is necessary to optimize the technique.

CONCLUSIONS

1. The extraction of chloride ions from salt-contaminated concrete by electromigration techniques has been shown to be feasible.
2. Optimum applied voltages, treatment times, surface electrolyte, and anode material were determined from laboratory investigations.
3. Treatment of a 3 by 12.2-m (10 by 40-ft) section of a bridge deck removed up to 90 percent of the chloride present in the concrete above the top reinforcing steel.
4. Pretreatment and posttreatment potential scans showed that all active corrosion of the reinforcing steel in the treatment area was halted after chloride extraction.

ACKNOWLEDGMENTS

REFERENCES

1. H. A. Berman. Journal of Materials, Vol. 7, No. 330, 1972.
2. D. L. Spellman and R. F. Stratfull. Chlorides and Bridge Deck Deterioration. HRB, Highway Research Record 328, 1970, pp. 38-49.
3. R. F. Stratfull, W. J. Jurkovich, and D. L. Spellman. Corrosion Testing of Bridge Decks. Paper presented at 54th Annual Meeting, TRB, 1975.
4. K. C. Clear. Evaluation of Portland Cement Concrete for Permanent Bridge Deck Repair. Federal Highway Administration, Rept. FHWA-RD-74-5, 1974.

Discussion

T. R. Cantor, Port Authority of New York and New Jersey

Conceding that the corrosion of rebars could be arrested by the method described, the paper states that the porosity of the deck will be increased. This porosity will cause the deterioration of the deck-rebar system, and therefore the whole procedure would appear not to be useful. In addition, some of the data and methodology are open to question and could result in misleading conclusions.

DECK DETERIORATION

A reinforced concrete bridge deck functions because the concrete on top carries the load in compression (its strongest mode) and the reinforcing bars carry the tension load (weakest mode). If the function of the upper concrete is lost, there could be no great incentive to preserving the lower tensile steel, which, as I understand it, is the thrust of the paper under consideration.

When the concrete above the rebars is originally properly placed and cured, it is relatively impermeable. In the natural course of events, the concrete becomes more porous for a variety of reasons, among which are loading, temperature stresses, vibration, and freezing and thawing. With time, the pores, fissures, capillaries, crevices, and cracks open and enlarge. These openings collect debris such as dirt and chemicals. Chlorides can certainly be detected among the accumulations. In any case, a damp porous pavement that contains chemicals has all the elements required for the classic electrolytic cell so it is quite reasonable to expect that chloride ions could be encouraged to migrate under the urging of a 100-V potential and so be expelled from the deck.

REBAR CORROSION

For steel to corrode, moisture and oxygen are essential. All other chemicals can only influence the reaction (conversion of iron to iron oxide) by acting as a catalyst in one way or another. Even with all essential materials present, steel will not corrode under specific inhibiting conditions. Because chloride is not a corrosion product, it must in some manner act as a catalyst or improve the conductivity or encourage the formation of local anodes, which means that only traces are required to promote the reaction.

Given the relatively slow corrosion rate of steel in concrete, just small quantities of chloride should be more than adequate to encourage corrosion. The paper states that, at best, only 90 percent of the chloride is removed, which still theoretically leaves ample chloride to promote corrosion.

The authors claim that the steel was apparently reduced to a passive state at least for a limited time on the basis of half-cell potentials. Questions can be raised on the subject of passivity. For example, where did the calcium, sodium, and hydrogen released at the rebar go? Another consideration is that, inasmuch as nothing has been done to perpetuate the passive condition, it seems safe to assume that, in a few months to half a year, the

corrosion would become active again.

POTENTIAL MEASUREMENTS

From theoretical considerations and experience, half-cell potential measurements can only be made where the concrete has some degree of porosity, which then suggests that some degree of deterioration is present or that the concrete is not of the very best impervious quality. Therefore, if half-cell potentials can be measured at all, whether or not corrosion is taking place, the concrete must be porous and susceptible to further increasing porosity and damage.

The paper indicates that the concrete was more porous after the electrical treatment. This is significant since the molecular volume of chloride is larger than the molecular volume of either oxygen or water. Obviously, if chloride can come out, there are sufficiently large channels for oxygen and water to enter the concrete. The presence of water and oxygen should start or continue the corrosion of the rebar after a relatively short time, which suggests that we would be right back where we started.

SEALING

The paper recognizes the increasing porosity problem and suggests that a procedure for overcoming it could be the application of a deck sealant. Sealing of decks is always a good procedure, but, because corrosion can only proceed in the presence of oxygen and water, if the deck is to be sealed why bother to remove the chlorides?

SALT DATA

The methodology used and the data reported seem inadequate to the situation. Unless the data in Tables 1 and 2 are being misinterpreted or the samples are not comparable because of their distance from each other (we have found appreciable differences in chloride content a few inches apart), it is difficult to understand the higher chloride content in August 1974 than November 1973 and no appreciable increase over August in April 1975. Additionally, if the cores were not sliced dry, considerable chloride could be washed from the sample if water was used during slicing.

My own work indicates that, from the end of one salting season in April to the next salting season in October, up to 75 percent of the salt was washed out of the deck at all depths by normal spring and summer rains. A washing procedure might have been used to reduce the chloride before potential application.

SAMPLE PREPARATION

It seems that the sample preparation was inadequate even though the undesirable laboratory procedure of introducing salt did not affect the field results. Nevertheless, it is difficult to understand the deliberate use of saltwater in the preparation of concrete specimens since the samples cast certainly are not normal concrete.

If the object were to load hardened concrete with salt, that could have been easily accomplished by infusing salt solution under vacuum into the specimen, drying, and repeating as often as desired. By this technique, we have achieved salt loadings of more than 120 kg/m^3 (200 lb/yd^3).

SUMMARY

The need to salvage the many deteriorating bridge decks is generally accepted, but there is small chance of success if the problem of deterioration is not well understood. The method suggested could do more harm than good because the deck becomes more porous and hence more vulnerable to deterioration. If sealing is then required, why bother with the whole procedure, since sealing itself cuts off the source of oxygen and moisture, without which corrosion cannot occur? The duration of any passivity achieved may be relatively short, so again the question of the benefit of the procedure has to be raised. Further doubts are raised by the salt data and the questionable practice of preparing specimens with saltwater. Finally, for repairs to be effective the bridge deck should be returned to its original design concept with good concrete working in compression over rebars working in tension.

Authors' Closure

The research described in our paper was undertaken on the basis of the widely held premise that the primary cause of concrete bridge deck spalling is the chloride-induced corrosion of the top level of reinforcing steel in the deck. It was therefore assumed that the removal or immobilization of the chloride ions in the concrete could halt or reduce corrosion of the reinforcing steel. Inasmuch as Cantor does not concur with the original premise, his criticism of the chloride removal concept is understandable.

DECK DETERIORATION

Cantor has interpreted the thrust of the paper to be one of "preserving the lower tensile steel" in a concrete bridge deck. From the viewpoint of preservation, our interest lies in preserving the concrete above the top level of reinforcing steel; this concrete is destroyed as a result of chloride-induced corrosion of the top level of reinforcing steel.

Cantor states that, in the natural course of events, concrete becomes more porous because of loading, temperature stresses, vibrations, and freezing and thawing. It is generally understood that properly cured concrete becomes less porous with age as cement hydration progresses. The factors he mentions have no effect on concrete porosity per se; they can cause gross cracking of the concrete deck, but these cracks are not essential for the migration of chloride ion into the concrete.

REBAR CORROSION

We do not believe that the mechanism of aqueous corrosion of steel outlined by Cantor is correct. The pH of the solution in which the steel is exposed has a dominant role in defining the nature and extent of corrosion. For example, in dilute sulfuric acid, steel corrodes rapidly, whereas even in the presence of air steel is resistant to corrosion in alkaline solution with pH > 12. This latter situation in fact obtains in chloride-free concrete; in such an environment, the steel forms a thin oxide film on its surface that protects it from further corrosion. The steel is said to be in the passive condition. The action of chloride apparently is to destroy the protective qualities of this film, and rapid corrosion can then ensue. The manner in which the chloride breaks down the film is as yet uncertain; the breakdown occurs initially at discrete points on the steel surface, which spread laterally; these are the anodes to which Cantor refers. The concentration of chloride necessary to cause film

breakdown is known to increase as the pH increases; more than trace quantities of chloride are necessary to cause the film breakdown process at the pH of moist concrete (12.5).

In the reported work on the bridge deck, the best removal achieved was a 90 percent reduction in the total chloride contained in the top 52 mm (2 in) of concrete. Cantor is concerned that this situation "theoretically leaves ample chloride present to promote corrosion." What is important here is the distribution of this residual chloride, inasmuch as any chloride at the rebar surface can induce (or reinduce) corrosion. We showed that chloride ion removal was always greatest from the concrete immediately adjacent to the reinforcing steel. Chloride concentrations at this level were obviously reduced by treatment to values below which corrosion would continue or reinitiate within a 3-month time interval (from the potential measurements). The crucial factor here is possible time-dependent redistribution of residual chloride after treatment, which may lead to renewed corrosion. This possibility is the focus of current research.

Cantor acknowledges that the removal treatment apparently reduced the steel to a passive state but is concerned that "inasmuch as nothing has been done to perpetuate the passive condition..., in a few months to half a year, the corrosion would become active again." As we discussed in the paper, we recognized from the outset that, once the chloride ion was removed by the electrochemical treatment, it would be necessary to seal the concrete (e.g., membranes or polymer impregnation) to prevent subsequent ingress of new chloride as a result of further applications of deicing salt. The possibility of residual chloride redistribution has already been discussed.

POTENTIAL MEASUREMENTS

With regard to measurement of half-cell potential, we agree strongly that such measurements can only be made when the concrete is an electrolytic conductor. However, concrete in normal service is a conductor. It only ceases to be a conductor when all free moisture has been removed. Cantor believes that half-cell measurements can only be made where the concrete has some degree of porosity. He further says that the presence of this porosity suggests that some degree of concrete deterioration is present. We must assume that the deterioration to which Cantor refers is gross cracking of the concrete, because all concrete has a measurable porosity. The ability to measure half-cell potentials in our opinion in no way confirms that the concrete is of poor quality or is deteriorating, as Cantor suggests. Cantor states that the paper "indicates that the concrete was more porous after the electrical treatment." This conclusion is not stated in the paper, for we made no measurements of concrete porosity either before or after treatment. Some oral discussion on possible porosity problems occurred after presentation, when unpublished Kansas data were discussed that indicated an increase in the porosity of concrete after electrochemical treatment.

SEALING

Cantor states that "sealing of decks is always a good procedure, but, because corrosion can only proceed in the presence of oxygen and water, if the deck is to be sealed why bother to remove the chlorides?" The bulk of opinion at this time indicates that the sealing of existing salt-contaminated decks is not always good practice. This is related to the presence of moisture in the deck and the diffusion of oxygen from the lower surface of the deck. Thus, oxygen and moisture are not excluded by the sealing process and corrosion will continue in the presence of chloride.

The Federal Highway Administration in fact has recommended that membranes not be placed on a deck if the chloride content is higher than 0.89 kg/m^3 (1.5 lb/yd^3) at the rebar level.

SALT DATA

Cantor's remarks concerning our methodology of measurement of chloride in the concrete are (a) the nonuniform distribution of chloride in field concrete and (b) the wet coring method which, it is claimed, could wash considerable chloride from the sample. First, we stated that we took into account the large specimen-to-specimen variation in measured chloride content. Second, the amount of chloride washed from a concrete core during the 5-min coring process is negligible.

Cantor's statement that 75 percent of salt is washed out from bridge decks by normal spring and summer rains is surprising. So far as we know, this phenomenon has never been reported in the literature by other workers. In fact, work on flushing techniques on chloride-contaminated concrete at Battelle, not reported in this paper, led to the conclusion that the technique was ineffective in removing the chloride.

SAMPLE PREPARATION

The practice of adding chloride ions to the laboratory concrete samples was an expedient to provide chloride ions in the free water phase of concrete. Drying and vacuum impregnation of the large slab specimens used in the program were not a possibility.

The similarities in results from the electrochemical removal treatments in the laboratory and field lead us to conclude that this expedient did not compromise our experimental approach.

SUMMARY

The concluding statements made by Cantor reaffirm that his interpretation of the causes of concrete bridge deck spalling is quite different from the more usually held view. Thus, he questions the need for and the possible problems attendant on prevention of the deck spalling problem based on electrochemical removal of chloride ion.

Our work was undertaken on the assumption that chloride in concrete decks is undesirable and leads to corrosion of the reinforcing steel and that the condition of the deck can be stabilized or improved by removing the chloride and by then taking steps to prevent subsequent ingress of further chloride. The objective of the research, to develop a technique for removing chloride ions from concrete bridge decks, was met. This is the necessary first step in evaluating the utility of this unique approach for preventing bridge deck spalling. As we state in the paper, however, the overall technical and economic viability of this approach remains to be demonstrated.

Fundamentals of Corrosion

R. P. Brown and R. J. Kessler, Florida Department of Transportation

Knowledge of the fundamental concepts of corrosion is necessary to understand the corrosion process. Because the scope of the corrosion concept is broad, this paper presents only the basic principles involved in the corrosion process. Those things that affect the corrosion process are defined and classified. Corrosion is divided into two classifications: wet corrosion and dry corrosion. This paper deals primarily with wet corrosion and covers the corrosion mechanism, forms of attack, and the effects of environmental factors. The basic parts of a corrosion cell are explained. Three types of corrosion cells, the principles of polarization, and the rate of corrosion are discussed. Examples are used throughout the paper to illustrate applications of the various principles. Forms of corrosion such as uniform and pitting attack are presented. Environmental influences discussed include biological corrosion, stray current, and differential environments.

Corrosion is one of the major problems facing engineers today. The seriousness of the problem is illustrated by the fact that corrosion costs $15 billion per year in the United States. This can be further illustrated by the fact that 40 percent of the steel produced each year is used to replace corroded metal (1). Because corrosion is such a serious problem, its fundamentals must be understood.

Corrosion is "the deterioration of a substance (usually a metal) or its properties because of a reaction with its environment" (2). This deterioration can be classified into two categories: dry corrosion and wet corrosion. Dry corrosion occurs in the absence of a liquid phase or above the dew point of the environment. In this case, a metal reacts with vapors and gases usually at high temperatures. These reactions can produce a scaling and tarnishing attack on exposed metals. In wet corrosion, a metal usually reacts with aqueous solutions or electrolytes. Wet corrosion causes the greatest amount of deterioration of materials (3).

This report relates the fundamentals of corrosion and emphasizes corrosion mechanisms, forms of attack, and environmental influences. Because of the magnitude of the subject, it is impossible to cover all aspects in detail in this paper.

CORROSION MECHANISM

Corrosion occurs by chemical or electrochemical means. An example of a chemical reaction is the dissolving of a metal by an acid. An electrochemical reaction involves metals, chemicals, and water that combine to form cells capable of generating electricity. Through the action of these cells, metals are reverted back to their native compounds by using the electricity as a source of energy. Figure 1 shows that electrochemical cells contain four basic components: electrolyte, anode, cathode, and conductor. The electrolyte is the current-carrying medium, such as water or soil. The anode (positive electrode) is in contact with the electrolyte, and corrosion will occur through an oxidation reaction that involves the formation of ions and the release of electrons (4). The cathode (negative electrode) is also in contact with the electrolyte, but no corrosion occurs because of a reduction reaction, which is the acceptance of electrons generated from the anode (5). The conductor connects the anode and cathode to complete the circuit for current flow (4). In the wet corrosion process, all corrosion cells that are formed contain these four basic parts.

Inasmuch as the corrosion cell is electrical in nature, it operates proportional to Ohm's law, which states that "the amount of flowing current (corrosion loss) decreases as the resistance of the circuit increases and current flow increases as the potential difference between the anode and cathode increases. Thus the amount of corrosion experienced is proportional to the electrolyte's ability to react, the potential difference between anode and cathode, and the amount of resistance in the external metal circuit" (6, p. 4).

Several factors influence the electrolyte. They are ionization, resistivity, pH, and temperature. The flow of electricity through an electrolyte depends on the presence of tiny electrically charged particles called ions. These ions form the basis for the conversion of metals from a metallic form to a corroded form. Ions are formed by the addition or removal of an electron

from the basic atom. The removal of an electron yields a positively charged ion, while addition yields a negatively charged ion. Ions are also formed by dissociation, which is the breaking up or splitting of a compound into its elements. Another factor affecting the electrolyte is resistivity, which is a measure of an electrolyte's ability to resist or oppose the flow of electric current. Low resistivity means that electricity will flow freely. High resistivity means that the electrolyte is a poor conductor (7). A third factor affecting an electrolyte is the degree of acidity or alkalinity of the solution, i.e., the relative amounts of hydrogen ions, hydroxyl ions, or pH. And, finally, temperature has a pronounced effect on electrolytes. As temperature increases, the resistivity of the electrolyte is lowered. Conversely, as the temperature decreases, the resistivity increases.

Another important factor affecting a corrosion cell is the difference in potentials of the metal. The driving force for current and corrosion is the potential developed. The electromotive series (Table 1) indicates the known potentials of the various metals (3). The difference in these potentials gives some indication of their effect on the corrosion cell.

The resistance of the external circuit connecting the anode and cathode has a pronounced effect on the corrosion cell. When the resistance in this circuit is high, the rate of current flow is low and the amount of corrosion is reduced.

In the wet corrosion process there are three basic types of cells: galvanic, concentration, and electrolytic. The galvanic cell (Figure 2) is one in which the anode and the cathode are of different metals or one in which dissimilar conditions exist in the same metal (7). In this cell the electrolyte is homogeneous and the dissimilarity in the metals causes a potential difference, which induces current flow. The common flashlight battery is a good example of a galvanic cell. Current is generated by connecting the zinc case and carbon center post through a light bulb. The current flowing from the zinc case through the electrolyte filler to the carbon rod eventually corrodes the zinc, and the battery must be replaced. In the concentration cell (Figure 3), the anode and cathode are the same material while the electrolyte is nonhomogeneous (7). Current is established by different oxygen concentrations, different kinds of chemicals, or different concentrations of the same chemical in the electrolyte. The difference in the electrolyte can be illustrated by material along a buried steel pipeline. Conditions vary from wet and swampy to dry and sandy. The electrolytic cell (Figure 4) contains all the four basic components of a corrosion cell plus an external source of electric energy (7, pp. 24-48). This source is connected in the conductor between the anode and cathode to cause current flow. Corrosion takes place as an electrolytic cell when a rectifier impressing direct current is used for corrosion control on pipelines.

The three cells all contain the four basic components and induce current flow. As this current flows, deposition of hydrogen builds up a film on the cathode surface and begins to resist the flow of current. This reduction in current flow due to the resistance of the hydrogen film is known as polarization (4). If the hydrogen film is not removed either by evolution or by some reaction involving oxygen, the corrosion rate will be reduced. To understand what is happening during a corrosion process, it is helpful to know the rate at which a metal deteriorates.

Metals corrode at known rates. These rates are based on kilograms of metal lost per current discharged in a year. Some examples follow:

Metal	Rate (kg/A/year)
Lead	32.7
Zinc	10.0
Aluminum	3.6
Steel	9.1

FORMS OF CORROSION

The many forms of wet corrosion are created by the various environments and types of corrosion mechanisms. Two of the most common forms of corrosion attack are uniform and pitting corrosion. Other forms include galvanic, crevice, intergranular, stress, fatigue, erosion, fretting, and selective leaching.

The most common form of corrosion attack is uniform corrosion. It is a chemical or electrochemical reaction that proceeds uniformly over the entire exposed surface of a large area. It occurs on metal surfaces that are homogeneous in chemical composition or that have homogeneous microstructures (8, pp. 3-32).

Another common form of attack is pitting corrosion. It is an extremely localized attack that results in holes in the metal. "A pit is defined as a cavity or hole with the surface diameter about the same as or less than the depth" (3). It is one of the most destructive forms of attack because it causes failure by perforation and only a small percentage of weight loss of the entire structure. A unique type of anodic reaction takes place in a pit. The corrosion process produces conditions that are both reactive and necessary for the continuing activity of the pit. Pitting corrosion can be initiated by metallurgical inhomogeneities. Pits are randomly distributed and have varying depth and size. Maximum pit depth information is the most useful in estimating the service life of a metal attacked in this manner (8).

Galvanic corrosion can result from the potential difference between dissimilar metals when they are in contact and surrounded by an electrolyte. This type of corrosion can produce either a uniform or pitted surface.

Crevice corrosion is localized and is caused by different concentrations of ions or dissolved gases located within crevices and other shielded areas on exposed metal surfaces. The region deep within a crevice becomes the anode and corrodes freely. A cathodic reaction takes place at the mouth of the crevice. Even in a homogeneous metal, a potential difference may exist. Metal ions are formed within a crevice, and high acidic conditions exist. This can lead to continuing activity of the corrosion process (8).

Another form of localized corrosion is called intergranular corrosion. This corrosion occurs at and adjacent to minute grain boundaries, but the grain itself experiences relatively no corrosion. Intergranular corrosion can be caused by impurities at the grain boundaries, enrichment of an element, or depletion of one of these elements in the grain boundary areas. Therefore, the grain boundary becomes anodic to the surrounding grains. As a result, the alloy at the grain boundary corrodes and may also lose its strength (3). Stainless steels that are improperly heat treated and do not contain special stabilizing alloying additions are susceptible to intergranular corrosion (8). Intergranular corrosion can be recognized by crack patterns along the grain boundaries.

The combination of tensile stress and corrosive medium produces a process known as stress corrosion. The tensile stress can be residual or external. Stress corrosion occurs in the form of intergranular or transgranular cracks. More damage is caused when the two are combined than when either acts alone. This type of corrosion usually causes failure without warning. Ex-

amples of stress corrosion are areas surrounding a weld and the stressed areas adjacent to rivets and bolts.

Corrosion fatigue is probably an extension of stress corrosion. "Fatigue is defined as the tendency of a metal to fracture under repeated cyclic stressing. Corrosion fatigue is defined as the reduction of fatigue resistance due to the presence of a corrosive medium" (3). Therefore, corrosion fatigue is related to the failure of the mechanical properties rather than to appearance.

When the rate of corrosion on a metal accelerates because of relative movement between a corrosive liquid and the metal surface, erosion corrosion occurs. It occurs in the form of grooves, gullies, waves, rounded holes, and valleys. The relative movement exposes new reactive locations on a metal, which become anodic to the relatively large surrounding cathodic area. At these locations rapid corrosion can occur. Equipment exposed to moving fluids are subject to erosion corrosion. Some of these are piping systems, valves, pumps, propellers, impellers, heat exchanger tubing, turbine blades, and equipment subject to spray (3).

Another form of corrosion involving movement is fretting, which occurs at contact areas between materials under load subjected to vibration and slip. Fretting occurs, for example, when two parts such as bushings and bearings are closely fitted. Fretting leaves pits or grooves in the metal surrounded by corrosion products (3).

Figure 2. Galvanic cell.
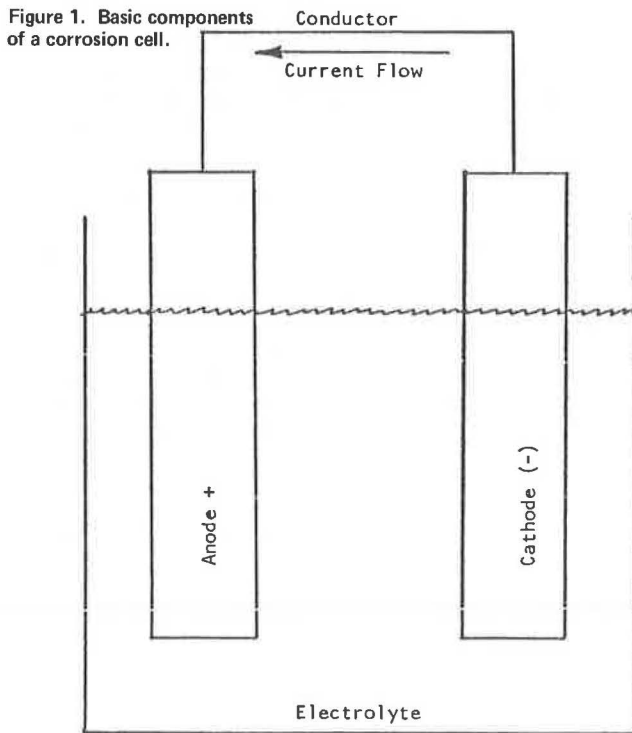
Figure 1. Basic components of a corrosion cell.

Table 1. Electromotive series.

Type	Element	Electrode Potential[a] (V)
Active or anodic	Magnesium	-2.363
	Aluminum	-1.662
	Zinc	-0.763
	Chromium	-0.744
	Iron	-0.440
	Cadmium	-0.403
	Nickel	-0.205
	Tin	-0.136
	Lead	-0.126
	Hydrogen	-0.000
Noble or cathodic	Copper	+0.337
	Silver	+0.799
	Platinum	+1.200
	Gold	+1.498

[a]Electrode potential versus normal hydrogen electrode at 25°C (77°F).

Figure 3. Concentration cell.

Figure 4. Electrolytic cell.

Figure 4. Electrolytic cell.
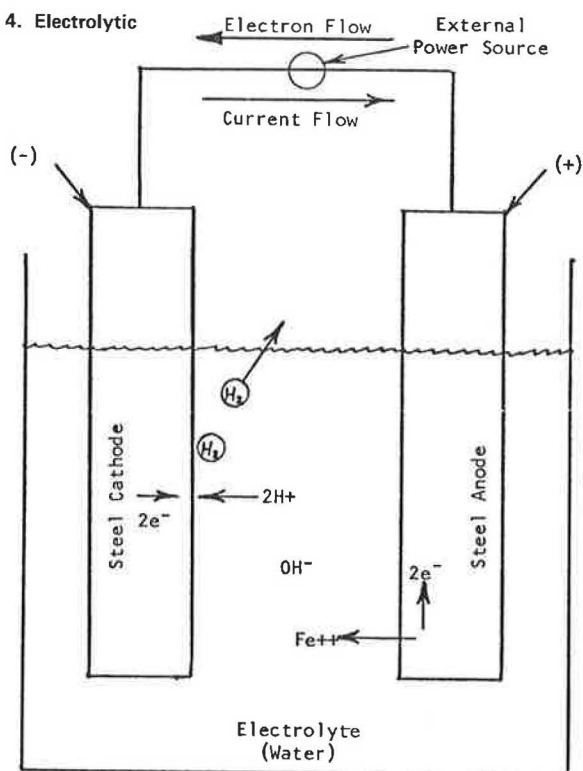
Selective leaching is a form of corrosion in which one or more elements are removed from an alloy and become electrochemically active. When an alloy consists of highly dissimilar elements, selective leaching is possible (8). One of the most common forms of selective leaching is graphitization. An example of graphitization is the removal of iron from cast iron pipe, leaving graphite.

ENVIRONMENTAL INFLUENCES

The forms of corrosion discussed can be influenced by the nature of their surrounding environment. An aggressive or electrochemically active environment can greatly increase the rate of corrosion of a metal. Some of the environmental factors influencing this rate are biological corrosion, stray current, and differential environments.

Biological corrosion is not actually a form of corrosion. However, the activity of living organisms (bacteria, algae) can increase the corrosion process by influencing the anodic and cathodic reactions and the protective surface films. It can also create corrosive conditions by producing deposits (3). Two types of microorganisms can cause this type of corrosion. These are aerobic (oxygen using) and anaerobic (nonoxygen using). Of the two types, anaerobic organisms are most responsible for corrosion of iron and steel.
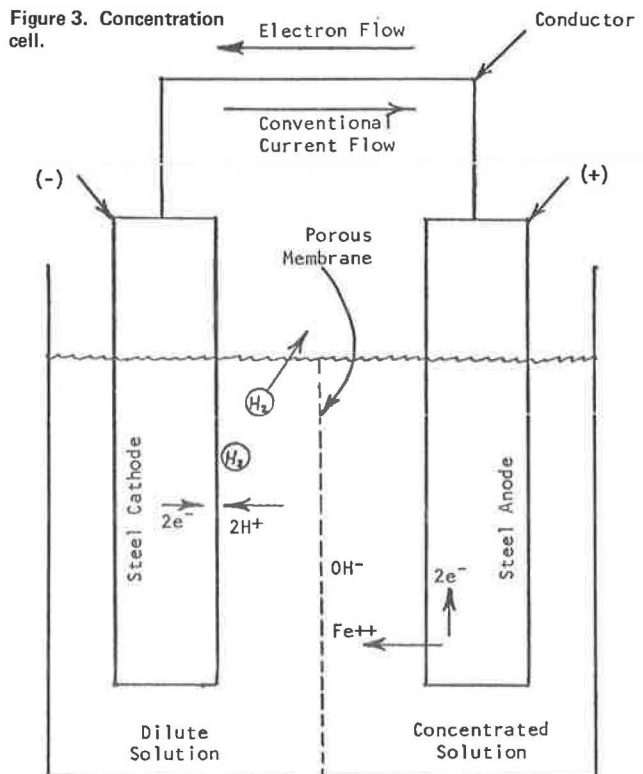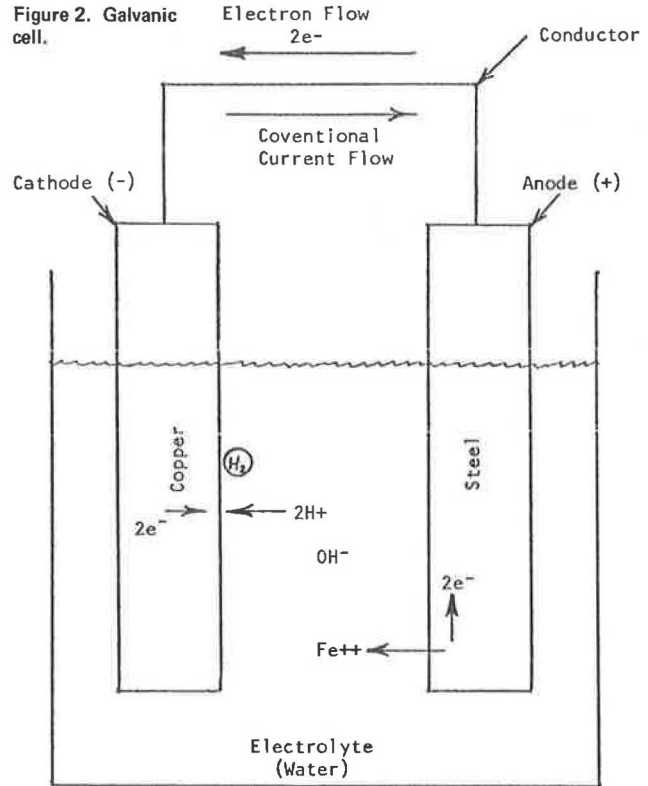
Another environmental influence is stray current, which is caused by direct current emanating from an outside source entering and being discharged from a metal surface. At the point where stray current enters the metal surface, the area becomes the cathode. The area of discharge becomes the anode, and corrosion occurs. Sources of stray direct current include electric railways, railroad signal systems, cathodic protection systems for pipelines, direct current industrial generators, and direct current welding equipment. Under certain conditions, alternating currents can become partially rectified by passing through and off of metal structures,

and the partial direct current component can cause corrosion.

Differential environment also affects the corrosion process. It occurs as a concentration cell when a metal is in contact with an electrolyte that varies in composition from point to point or varies in concentration from point to point along the surface of the metal. Examples of these variations in environments include the conditions present along a buried pipeline, a steel pipe in contact with both soil and water, or the interface between a steel reinforcing bar and the surrounding concrete.

SUMMARY

This paper discusses the fundamentals of corrosion. The paper defines terms and basic electrochemical principles that determine the corrosion process. These definitions should not be considered the only correct ones. Other definitions can be found that may be as good or even better. The principles of corrosion mechanism, forms of attack, and environmental influences have been used to simplify the various aspects of corrosion.

REFERENCES

1. R. W. Staehle. Editorial. Corrosion, Vol. 31, No. 1, Jan. 1975, pp. 1-4.
2. A. deS. Brasunas. Scope and Language of Corrosion. Basic Corrosion Course, National Association of Corrosion Engineers, Houston, 1970.
3. M. G. Fontana and N. D. Greene. Corrosion Engineering. McGraw-Hill, New York, 1967, pp. 1-115, 282.
4. O. C. Mudd. Control of Pipe-Line Corrosion. Corrosion, 1945, pp. 2-4.
5. F. L. LaQue. Introduction to Corrosion. Basic Corrosion Course, National Association of Corrosion Engineers, Houston, 1969, pp. 2-4.
6. W. T. Bryan, ed. Designing Impressed Current Cathodic Protection Systems With Durco Anodes. Duriron Co., Dayton, 1970.
7. Corrosion Control. In Engineering and Design Manuals, U.S. Army Corps of Engineers, Aug. 1, 1962.
8. F. H. Cocks, ed. Manual of Industrial Corrosion Standards and Control. ASTM, Special Technical Publ. 534, 1973, pp. 3-32.

Pipe Corrosion and Protective Coatings

Bob H. Welch,* Transportation Research Board

Fifty-eight pipe culvert installations were sampled for durability characteristics; the samples were assigned a relative pipe rating value for the purpose of numerical analyses and correlation with corrosive environmental soil and water conditions. The independent variables used included chemical composition of the water and soil, pH, total soluble salt content, minimum resistivity, resistivity at the naturally occurring soil moisture content, resistivity using the corresponding field water, and age of the pipe. Results indicate the total soluble salts is a more significant factor than any single soluble salt content in predicting the performance of pipe material. All soil sites examined eventually reached a soluble salt content of 0.8 percent. The corrosive effects of the solubles peak at approximately the 5 percent level. The effects of pH and minimum resistivity are found to be higher at the lower soluble salt content (<1.5 percent), and both lose their dominance at higher salt concentrations. Minimum resistivity, in particular, loses its effect on pipe life expectancies at a solubles content greater than 1.5 or 2 percent. The criterion used to predict pipe performance correlates very well with field observations and varies only in areas beyond the limits of the selection criteria.

Durability is one of the important factors that must be considered when a particular type of culvert pipe material is selected to be used in a given environment. Durability greatly influences the service life of a culvert; it is often the main criterion for choosing a particular material, as well as its thickness and the protective coating that it should have. Experience of many engineers and studies by various investigators have indicated that specific environmental characteristics of the culvert backfill soil and runoff waters greatly influence the corrosion performance of various pipe materials and coatings. There are some discrepancies about the environmental parameters that cause material corrosion and their relative quantitative effects.

The guidelines and recommended criteria for selecting culvert material are based on environmental conditions that are most prevalent in Utah soils, i.e., an average resistivity of approximately 11.0 Ω·m, an average pH of 8.3, and an average soluble salt content of 1.5 percent (alkaline soils principally with low sulfate contents). In addition, few drainage structures in Utah have continuous water runoffs; many are in semiarid climates where drainage flows are intermittent. Therefore, the findings and conclusions presented here should be extrapolated to other environmental limits with caution.

A random sampling of pipe materials for corrosion and abrasion analysis was chosen so that a variety of pipe materials and environmental surroundings and a wide span of time in place would be included. Pipes whose history of placement and specifications were not complete were eliminated from consideration. Six categories of pipe materials were evaluated: reinforced concrete, corrugated galvanized steel, aluminum alloy, bitumen-coated corrugated galvanized steel, bitumen-coated asbestos-bonded corrugated galvanized steel, and structural plate corrugated galvanized steel.

When the pipes were inspected, the following information was recorded on forms: type of pipe, height of backfill, degree of corrosion, type of corrosion, location of corrosion, visual observations, slope of pipe and channel, degree of erosive scour, topographic description, degree of abrasion, and any additional remarks about the condition of the pipe or its environmental surroundings. Then a 10-cm-diameter (4-in) core was drilled out of the pipe. For consistency among core samples, the samples were taken 3.6 m (12 ft) from the pipe end and 15 deg up from the pipe invert.

Soil samples were taken from the soil side of the culvert and placed in waterproof containers to be analyzed in the laboratory at a later date. Where runoff waters were discharging through the culvert, a corresponding water sample was also obtained. Pictures were taken of the surrounding topography, the invert, and the soil side of the pipe.

Soil samples were analyzed for the following: percentage of natural moisture, total soluble salts, soil pH, minimum resistivity, silicon dioxide, iron oxide, aluminum oxide, calcium oxide, magnesium oxide, soluble sodium oxide, insoluble sodium oxide, soluble potassium oxide, insoluble potassium oxide, chlorine, carbon dioxide, sulfates, and organics. Soil samples received

from drainage structures having a runoff also were tested for minimum resistivity by using the field water instead of distilled water. Water samples, where available, were analyzed for sulfate, chlorine, calcium oxide, magnesium oxide, sodium oxide, potassium oxide, and carbon dioxide content in parts per million.

Core samples were first cleaned of any loose debris and then visually evaluated by three people and assigned a tentative pipe rating (PR) on a scale from 10 (excellent) to 0 (failure). These cores were then randomly measured in five locations for thickness to the nearest 0.0025 cm (0.001 in) and weighed. An average of the five thickness measurements was used as the thickness number. These samples were stripped of their zinc coatings and were again measured and weighed. The tentative pipe rating evaluations were reviewed based on visual observations of condition of the core metal of each sample. Final pipe ratings then were assigned each specimen after the field notes at each location, photographs of each location, and observations of the 10-cm (4-in) cores were compiled. This PR became the final number designating the relative degree of corrosion sustained by each pipe, which was used in numerical analysis.

The statistical and numerical analyses of data on the soil and water samples were categorized in three main areas. First a simple correlation coefficient matrix was determined by using all variables from all samples for possible correlation or dependency between environmental parameters. The pipe ratings, pipe ages in place, before and after core thicknesses, before and after core weights, design material weights and thicknesses, and combinations of their ratios were included in the data analysis. Then the results from this analysis were used to determine correlations between combinations of single independent parameters. Finally, multiple linear regression analysis was performed on both the single parameters and the groups of parameters.

Inasmuch as PR = 2 indicates that a pipe needs maintenance or replacement (not necessarily structural or hydraulic failure), PR = 2 was set as a constant in the equations, which were then solved for age. Inasmuch as the corrosion process may not be linear with respect to time, the age scale was adjusted so the equation adequately described the correlation of a PR = 2 and age to the condition of PR = 2. From these equations, the constant was adjusted for various types of coatings and metal thicknesses.

For each of implementation, these equations then were plotted for use by materials engineers as a guide in selecting pipe culvert materials.

Results presented are primarily geared toward the useful design life of underground culvert materials. These results may also be applicable to similar underground installations such as storm drains, cross drains, side drains, or bin walls exposed to underground, long-term deterioration by the immediate soil environment.

Inspection of culvert pipes throughout Utah indicated that the durability criteria of corrosion and abrasion should be given adequate consideration in the design and planning phases of highway development in conjunction with structural, hydraulic, construction, material availability, and economic factors.

At the test sites selected throughout Utah, there are no acidic soils (pH < 7). The pH of all soils is in the alkaline range. The only areas where acidic attack on underground pipe materials may be a problem are very isolated and unique instances. In these areas the runoff waters could possibly be acidic; however, the soil remains predominantly alkaline. Some general observations regarding pipe corrosion and durability based on investigations of several pipe sites are as follows:

1. Pipe extremities (the outer 1.8 to 2.4 m or 6 to 8 ft) corrode at a much faster rate than the interior of the pipe;

2. The exterior circumference of the pipe corrodes at approximately the same rate; and

3. Corrosion was a problem predominantly on the exterior or soil side of the pipe and not on the invert side.

In general, because of the predominantly flat topography and basically arid or semiarid climates that characterize the sites examined and because only six of the locations had a continuous year-round water flow, abrasion and scouring did not seem to be a problem for most pipe installations. Sediment buildup was a more serious problem than scour or invert abrasion in the majority of instances.

Specific results of the statistical analysis of the data obtained from the soil and water around each pipe location include a simple correlation matrix for each class of pipe and all pipe classes together. Table 1 gives the simple correlation coefficients for each class of pipe. Aside from the more widely accepted independent variables of age and soil pH, the following variables are used:

Variable	Symbol
Independent	
Minimum soil resistivity	R
Total soluble salts	SS
Natural moisture content	NM
Dependent	
Pipe rating	PR
Metal loss	ML
Highest pipe rating	HPR
Lowest pipe rating	LPR
Highest pipe rating -	
lowest pipe rating	HPR-LPR

The results of this analysis indicate that no single or group of single parameters of age, SS, or pH adequately explains the deterioration process of underground drainage structures. From the table, the single most important parameter, if used by itself to describe pipe performances, is the minimum soil resistivity.

Figure 1 shows pipe ratings versus resistivity. Although resistivity may be the single most important variable, based on the widely scattered data in Figure 1 and field experience, it alone is not reliable enough to explain pipe corrosion.

To expand the simple correlation matrices to include combinations of SS, pH, R, and age together with analysis of chemical components of the water and soil seems necessary to better explain the corrosion phenomenon. Several combinations of these variables were analyzed; those used are summarized by Welch (1).

Because no single soil parameter or groups of independent soil parameters adequately explain or can be used to predict pipe performance, a multiple linear regression analysis was performed. Two equations, each significant at the 0.05 level and containing the environmental parameters that can easily be evaluated for future pipe locations, were selected as the most suitable to represent the interaction of these environmental parameters and pipe performance. The formula for concrete is

$$\text{Log PR} = 0.66 + 0.18 \log \left[R/(SS \times pH \times age) \right] \qquad (1)$$

and that for plain corrugated steel pipe is

$$PR = 9.25 + 0.15 SS + 0.007 \left[R/(SS \times pH) \right]$$
$$- (0.0013 \times SS \times pH \times age) - 0.06 pH^2 \qquad (2)$$

A pipe rating of two is considered a pipe failure from the durability or corrosion viewpoint. Only two pipes sampled actually had this rating. PR = 2 does not constitute a structural failure but rather an impending failure for which maintenance or replacement should be initiated. The pipes may not necessarily corrode at a uniform rate once corrosive attack has begun or when the pipe material nears the failure point, as shown in Figure 2. The rate of corrosion $\partial PR/\partial t$ for a particular pipe material at a given location is a function of the pipe's surrounding soil and water conditions. This report does not document the increasing rates of corrosion

Figure 1. Minimum soil resistivity.
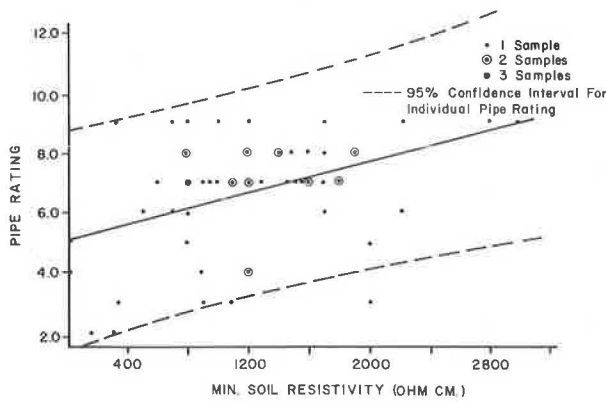
Figure 2. Time adjustment for multiple linear correlation analysis.

Figure 3. Material selection criteria.
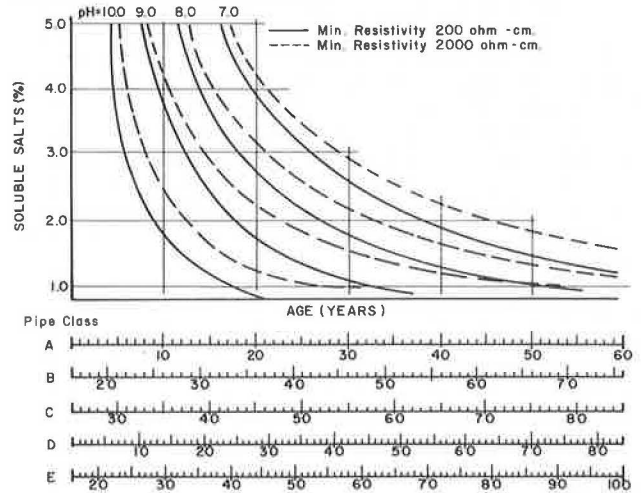
Figure 4. Concrete material selection criteria.

$SO_4 < 0.5$ % use Type - II Cement
$SO_4 \geq 0.5$ % use Type - V Cement

Table 1. Simple correlation coefficients of various pipe materials.

Pipe	Variable	PR	HPR	LPR	HPR-LPR	ML	Age	SS	pH	R	NM
Concrete	PR	1.000	0.712	0.934	-0.401		-0.231	-0.493	0.188	0.684	
	HPR		1.000	0.408	0.349		0.321	-0.109	0.245	0.591	
	LPR			1.000	-0.691		-0.173	0.613	0.104	0.584	
	HPR-LPR				1.000		-0.078	0.503	0.089	-0.134	
	Age						1.000	0.441	0.121	-0.357	
	SS							1.000	0.342	-0.353	
	pH								1.000	0.142	
	R									1.000	
Asbestos-bonded,	PR	1.000				-0.245	0.008	0.632	0.632	0.605	-0.409
bitumen-coated	ML					1.000	0.557	-0.577	0.265	-0.366	0.058
corrugated	Age						1.000	-0.616	0.077	0.164	0.394
steel	SS							1.000	-0.213	0.229	-0.253
	pH								1.000	-0.489	0.141
	R									1.000	0.031
	NM										1.000
Bitumen-coated	PR	1.000				-0.742	-0.140	-0.485	0.235	-0.095	0.669
corrugated steel	ML					1.000	0.482	0.946	-0.472	-0.541	-0.011
	Age						1.000	0.577	0.420	-0.180	0.176
	SS							1.000	-0.485	-0.744	0.305
	pH								1.000	0.725	-0.311
	R									1.000	-0.792
	NM										1.000
Plain corrugated	PR	1.000					-0.126	0.039	-0.308	0.428	
steel	Age						1.000	-0.191	0.102	-0.194	
	SS							1.000	-0.260	-0.221	
	pH								1.000	-0.004	
	R									1.000	
All samples	PR	1.000					-0.206	-0.084	-0.212	0.444	-0.313
	Age						1.000	-0.164	0.096	-0.287	0.107
	SS							1.000	-0.248	-0.217	0.046
	pH								1.000	0.081	-0.095
	R									1.000	-0.190
	NM										1.000

with respect to time; the approach is presented here because it does provide a rational explanation of equations 1 and 2 with respect to service history. Because the equation developed by the multiple linear regression method and the majority of data fall between pipe ratings of 4.5 and 8 and because actual pipe deterioration is not linear with respect to time, the projected failure time is some time factor (t - Kt) greater than the actual failure time projected by the equations. Based on the observations of pipes in service, the K constant is 0.01 for equation 1 and 0.15 for equation 2.

Using this criterion to compare service life with pipe rating in two situations, the data collected in this study and results of 40 independent pipe tests have shown these selection procedures to be accurate to ±3 years at 0.05 significance level. Figure 3 shows equation 2 plotted against the following pipe scales:

Symbol	Pipe
A	Corrugated galvanized steel
B	Bitumen-coated corrugated galvanized steel, aluminum alloy, corrugated steel coated with pitch resin adhesive
C	Asbestos-bonded bitumen-coated corrugated steel
D	Plain corrugated steel structural plate
E	Bitumen-coated corrugated steel structural plate, aluminum alloy structural plate
F	Types 2 and 5 portland cement concrete

Figure 4 shows equation 1 plotted for type 2 portland cement concrete pipe. It should be noted that Figure 4 for concrete pipe is found to work in Utah's alkaline soils except at three locations. The soil at these locations had a sulfate content of 0.5 percent or higher. Therefore, in soils containing more than 0.5 percent sulfate, a type 5 cement is recommended.

From Figure 3, for pipe class D, the effects of resistivity and pH on service life are much greater at a lower soluble salt range (<2 percent) than at higher soluble salt ranges. More than 2 percent soluble salts indicate that resistivity becomes a secondary factor affecting durability, and the effects of pH are slightly reduced. The scale for soil solubles runs from 0.8 percent to 5.0 percent because the relative effects of a total soluble salt content above 5 percent are not appreciably greater than at the 5 percent level. A minimum level of 0.8 percent is recommended because (a) soluble salts below 0.8 percent are not the primary contributing factors to corrosion (pH and resistivity are) and (b) the older pipe locations inspected have accumulated higher salt content levels than may have originally existed at these locations because of applications of deicing salt, for example (2).

Our test data and other data accumulated from soil testing throughout Utah were compared to the iso maps presented by Meshgin (3) for possible use in lieu of a complete soil analysis at each pipe location. These data also were compared to the very detailed surface iso maps provided by the U.S. Department of Agriculture (4). Soil characteristics, particularly soluble salts, pH, and minimum resistivity, vary too much from one location to another to use the iso maps effectively without a large error in proper pipe material selection. Therefore, in the preconstruction phase of highway design, the materials engineer should sample soils in pipe culvert locations to identify potentially aggressive areas. However, the iso maps could be helpful in providing an indication of the soil conditions in the corresponding drainage basin.

To apply the field data on deterioration of pipes made of various materials to new culvert materials and coat-

ings requires a relatively rapid laboratory method for evaluating potential durability or resistance to corrosion. The results of the salt chamber, ozone, and electrolytic cell test determinations (1) indicate that these methods are not suitable for rapid evaluation of durability and corrosion resistance.

The mudpack test (1) after 8 weeks' exposure did cause noticeable deterioration of some pipe samples. Flakes of the galvanized steel coating and some local pitting occurred on the plain corrugated steel pipes and the plain corrugated structural steel plate pipes. The aluminum alloy cladding turned dark after 7 days' exposure and formed a uniform rough oxide coating after 3 weeks' exposure. This condition of the aluminum alloy remained unchanged throughout the entire 8-week exposure period. On all of the uncoated steel pipes, a salt-like crystalline structure built up around the samples on top of the mudpack. The corrugated steel pipe coated with pitch-resin adhesive had some adhesion loss along the edges but showed no signs of change on the coated side; however, the invert side did lose approximately 5 percent of the coating, and the remaining material had lost a considerable amount of its adhesive properties.

The bitumen-coated corrugated steel pipe sample showed no corrosion beneath the bituminous coating where it was totally intact after the mudpack test. Near the edges where the core metal had been exposed, the bituminous coating lost adhesion to the metal. The bituminous coating also had a tendency to flow at a temperature of 38° C (100° F) after the 8-week period. The asbestos-bonded bituminous coatings remained in good condition throughout the test and showed only slight adhesion loss near the edges where the base metal had been exposed.

Metal structures subjected to potential aggressive attack from alkaline soils can be identified if minimum resistivity, pH, and total soluble salts are known. By analyzing their combined effects, acceptable predictions can be made about the resistance of the steel to corrosion. At lower soluble salt contents, the rate of corrosion is highly dependent on the minimum resistivity and pH whereas high salt contents will in themselves be the principal corrosion-causing agent.

Deterioration of concrete pipes also is highly dependent on pH, soluble salts, and minimum resistivity in alkaline soil environments. However, the sulfate content in amounts of more than 0.5 percent may be the principal deterioration agent.

A more extensive laboratory mudpack testing program to quantify the relative effects of temperature, resistivity, pH, type of water, duration of test, sulfates, wetting-drying, and total soluble salts on various pipe materials should be undertaken. Included with these parameters, an electrolytic cell test should be applied to the various materials for optimum results. Based on the results of this investigation, the mudpack test is recommended for quantifying in a relatively short period of time the comparative durability characteristics of various pipe culvert materials. In conjunction with this recommendation, a universal method of pipe evaluation of culvert materials should be developed so information concerning corrosion or abrasion may be more fully used by design and materials engineers in areas where the information has not been accumulated. Only with this type of liaison can the optimum use and exchange of data describing laboratory and field durability be implemented and a single set of reliable corrosion standards be adopted by all highway agencies.

REFERENCES

1. B. H. Welch. Pipe Corrosion and Protective Coat-

24

ings. Utah State Department of Highways, Nov. 1974.

2. K. M. Jacobs. Culvert Life Study. Maine Department of Transportation, Technical Paper 74-1, Jan. 1974.

3. K. Meshgin. Pipe Durability Study in the State of Utah. Utah State Department of Highways, Aug. 1966.

4. Soil Survey, Roosevelt-Duchesne Area, Utah. U.S. Department of Agriculture, Series 1940, No. 24, 1959.

Laboratory Corrosion Tests of Galvanized Steel in Concrete

George A. Hill, D. L. Spellman, and R. F. Stratfull, Division of Structures and Engineering Services, California Department of Transportation

Concrete test specimens containing galvanized and black reinforcing steel were partially immersed in saturated salt solution. The results were that (a) corrosion began at about the same time for both the galvanized and black (mild) steel exposed in the same concrete system; (b) the greater the thickness of zinc was, the earlier concrete cracking occurred; and (c) in concrete of high quality, galvanized steel caused cracking earlier than black steel. Observations indicate that galvanizing is either inefficient or ineffective in preventing rusting of underlying steel in concrete. Most significant in postponing concrete cracking caused by corrosion of black or galvanized steel was increasing the cement factor. There does not appear to be a half-cell potential value per se that discloses the corrosion activity of galvanized steel in concrete.

Several sources report various aspects of the use of galvanized steel (1, p. 69) as a means for postponing concrete distress due to corrosion of the black reinforcing steel (5 through 9). However, the various reports (1, 2, 3, 4, 9) do not consistently indicate a significant benefit to using galvanized instead of plain reinforcing steel in concrete subjected to salt (chloride ion) contamination.

Zinc is a widely accepted rust-protective coating in a normal atmospheric environment. Because of this demonstrated property, galvanized steel rebars have been used in concrete to reduce the rate of steel corrosion in the presence of salt.

The environment under which a galvanized steel rebar exists in concrete with a pH of about 12.5 and is subject to attack by chloride ions is one about which comparatively little is known. The corrosion of zinc is inhibited through the formation of a mixture of zinc compounds such as the oxide, hydroxide, and carbonate. In addition, the zinc can serve sacrificially to protect steel exposed through scratches and cracks. To find how these protective measures react in portland cement concrete in an aggressive environment is the purpose of this research.

TEST PROCEDURE

The test procedure (8, p. 28) was to partially immerse the reinforced concrete specimens to a depth of 8.9 cm ($3\frac{1}{2}$ in) in a saturated solution of sodium chloride and in plain tap water. The level of solution and water was maintained by periodically adding water to compensate for that lost by evaporation and by having an excess of salt in the bottom of the tanks. Each addition of water was thoroughly stirred into the solution. All tests were performed in the laboratory, which had an air temperature of about 22°C (72°F). The relative humidity was assumed to range between 30 and 45 percent.

The specimens consisted of No. 4, or 1.3-cm-diameter ($\frac{1}{2}$-in), reinforcing steel bars embedded in concrete bars 11.4 cm ($4\frac{1}{2}$ in) wide by 6.4 cm ($2\frac{1}{2}$ in) thick by 38.1 cm (15 in) long. The amount of cover was designed to be not less than 2.5 cm (1 in).

The cement used was ASTM type 2, modified low-alkali, which complies with California State standard specifications.

Eighty replicate specimens were cast from 10 batches of 279 and 418-kg/m³ (5 and $7\frac{1}{2}$-sack) concrete for the laboratory tests. Half of each batch was cured with steam plus water, and the other half was cured with water only. Twenty replicate samples of either black or galvanized steel were used in each test of cement factor and curing.

In addition, 10 specimens were cast from 335-kg/m³ (6-sack) concrete. All had galvanized steel reinforcement. Half of the specimens were moist cured, and the remainder were steam cured.

Those specimens that were cured with steam plus water were subjected to a temperature of 59 ± 3°C (138 ± 5°F) for approximately 16 h and then submerged in tap water at 25 ± 2°C (73 ± 3°F) for a total cure period of 28 days. The water-cured specimens were submerged in tap water for 28 days before testing. The concrete mix data are given in Table 1.

In conjunction with corrosion testing, concrete absorption tests were performed in accordance with Test Method Calif. 538-A. Some details of the absorption test have been reported (8). Essentially, the test consists of oven drying the concrete specimens at 110°C

(230°F) and then submerging them in water. Table 2 gives the results of the concrete absorption tests.

After the specimens were placed in the testing solutions, electrical half-cell potential measurements were made thrice weekly. The purpose of these measurements was to determine when a corrosive quantity of salt had penetrated to the metal surface and to determine, if possible, an active (corroding) or passive (noncorroding) potential for the galvanizing existed.

For the galvanized steel specimens, the reinforcing steel was first sandblasted, weighed, and then galvanized. The weight of the zinc coating was calculated from the difference of the before and after galvanizing weights. The hot-dip galvanizing of the steel complied with the California standard specifications dated January 1969 and ASTM designation A 123.

It was intended that the thickness of the galvanized coating be the equivalent of 610 g/m^2 (2 oz/ft^2) to provide an average thickness of about 0.086 mm (0.0034 in) of zinc. However, weight measurements showed that the average amount of zinc deposited was equivalent to about 915 g/m^2 (3 oz/ft^2) and varied from 739 to 1420 g/m^2 (2.60 to 4.65 oz/ft^2).

RESULTS OF TESTS OF CONCRETE WITH GALVANIZED AND BLACK STEEL

As shown in Figures 1 through 4, the time to initiation of corrosion (as measured by the change in half-cell potentials) of galvanized and black steel in comparative concrete environments is similar irrespective of cement factor or method of curing. The difference in the time to cracking of the concrete caused by the corrosion of either steel or zinc showed that, in relatively porous (279 kg of cement/m^3 of concrete or 5-sack) moist-cured concrete, the mean time for 20 galvanized specimens to crack was 315 days, and in the concrete containing 20 black steel bars the mean time to cracking was 175 days. For the 279-kg/m^3 (5-sack) steam-cured specimens, the mean time to concrete cracking was 243 days for the galvanized steel and 124 days for the black steel.

In the dense (418-kg/m^3 or 7½-sack) moist-cured concrete, the mean time for the 20 galvanized steel specimens to crack the concrete was 549 days, while only 7 out of 20 black steel specimens had cracked at the end of the 622-day test period. The last galvanized specimen cracked in 678 days.

In the case of the galvanized steel specimens partially immersed in Sacramento city tap water (about 20 to 40 ppm Cl), three moist-cured concrete specimens out of the five cracked about 7.6 cm (3 in) above the waterline after about 600 days of testing.

At the conclusion of the test period of 1700 days, all 10 tap water specimens were opened for inspection of the galvanizing. On all but one specimen, relatively minor rust spots were observed on the zinc surface. The one exception was where rust was absorbed by the steam-cured concrete and was on the surface of the metal for a distance of about 0.32 cm (⅛ in). Zinc corrosion products were observed at the tap waterline and below, but above the waterline the zinc was generally free of corrosion products.

Three concrete samples from the 418-kg/m^3 (7½-sack) concrete were chemically analyzed and found to contain about 100 ppm chromate as CrO_3. This is estimated to be greater than that found necessary in cement pastes to prevent hydrogen gas evolution from zinc (9). Minor evidence of gas evolution was observed at the concrete-zinc interface, and many of the galvanized steel bars exposed showed no evidence of mortar porosity or of it sticking to their surfaces. The source

of the chromate was found to be in the aggregate and the cement.

A chemical analysis of the concrete section immersed in tap water showed about 3.3 kg of chloride ion per cubic meter of concrete (5.6 lb/yd^3). The chloride content of the atmospherically exposed concrete was found to be about 0.95 kg/m^3 (1.6 lb/yd^3). The source of the chloride in concrete was not determined. For the underwater concrete sections exposed to the saturated sodium chloride solution, the average amount of absorbed chloride was 37.6 kg/m^3 (63.3 lb/yd^3).

Table 2 gives concrete absorption test results. Based on the consistency of results, differences in concrete absorption do not appear to have affected any comparative test results.

As a further evaluation of the relationship between zinc and concrete cracking, the 20 galvanized specimens in each test were ranked in ascending order of actual weight of zinc and then separated into two groups. The two groups consisted of 10 that had the lightest weight of zinc in the particular test series and the other group of 10 that had the heaviest weight of galvanizing. Then the average of days to concrete cracking for each group in each test series was calculated and tabulated (Table 3). In each case the group of bars with the heaviest galvanizing cracked first.

DISCUSSION OF RESULTS

The test data showed that, for greater weights of zinc coating, the concrete cracked earlier. Also, because heavier weights of zinc coating resulted in earlier concrete cracking, it is obvious that the corrosion products of zinc exert sufficient pressure to crack a 2.5-cm-thick (1-in) concrete cover.

The potential of the galvanized bar in initially salt-free concrete was found to vary between -0.19 and -0.75 V SCE (saturated calomel half cell). This voltage difference of 0.56 V might result in localized corrosion of the zinc coating.

In highly salt-contaminated concrete the potential of the galvanized bar varied between -0.62 and -1.02 V SCE, which could cause sections of zinc to have a galvanic voltage difference of 0.40 V—a difference that could induce localized corrosion of itself. When zinc is in concrete of variable salt contamination (potential at 1.02 V SCE) (Figure 2) and a potential of -0.19 V SCE (Figure 5), then the corrosion of zinc in the salt-contaminated concrete could be initiated by a maximum of about 0.83 V driving the associated current flow.

For steel in concrete specimens exposed to a saturated sodium chloride solution, it was reported (16) that the mean potential of corroding black steel was -0.36 V SCE to a maximum mean of -0.48 V SCE. Therefore, when black steel in salt-contaminated concrete begins to corrode (potential of -0.36 to -0.48 V SCE) and is electrically interconnected to galvanized steel in relatively low salt concrete (minimum potential of -0.19 V SCE), then the reversed polarity zinc might cause the black steel to have accelerated corrosion as a result of the 0.17 to 0.29-V differential.

For the average indicated range of half-cell potentials of zinc (this report) and black steel in salt-free concrete (8, 16), it seems likely that the black steel would normally tend to cause accelerated corrosion of galvanizing in salt-free concrete.

Under certain conditions, the polarity of zinc can reverse, which may cause accelerated corrosion of black steel. For example, at temperatures of about 60°C (140°F) or higher in aerated hot waters, it was determined (12) that zinc does not act as a sacrifical coating but becomes noble and induces pitting of steel. It was

also found that waters high in carbonates increase the tendency of the polarity reversal of zinc to iron couple (13, p. 330)

It was reported that the formation of ZnO as a corrosion product of zinc is responsible for the polarity reversal instead of the porous $Zn(OH)_2$ or the basic zinc salt, which is normally anodic to iron (14, p. 16). The compound ZnO is reported to be a semiconductor that in

Table 1. Concrete mix variables.

| Variable | Mix (kg/m³) | | |
	279	335	418
Maximum aggregate size, cm	1.9	1.9	1.9
Actual cement content, kg/m³	277	332	418
Slump, cm	7.6	6.4	7.6
Net water/cement (by weight)	0.63	0.51	0.43
Gross water/cement (by weight)	0.72	0.59	0.47
Air (entrapped), percent	2.15	2.6	1.6

Note: 1 kg/m³ = 0.062 lb/ft³. 1 cm = 0.39 in.

Table 2. Concrete absorption (percent by volume).

Cement (kg/m³)	Cure	Galvanized Bars	Black Steel Bars
279	Moist	15.30	15.23
	Steam	15.53	15.64
335	Moist	14.83	—
	Steam	14.84	—
418	Moist	13.84	13.83
	Steam	13.42	13.48

Note: 1 kg/m³ = 0.062 lb/ft³.

Figure 1. Potentials of black and galvanized steel in 279-kg/m³ concrete (moist cured).
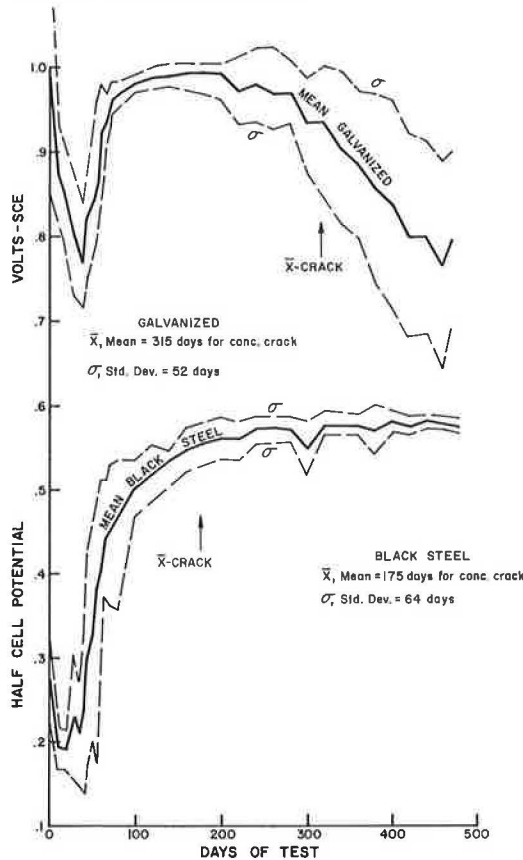

Figure 2. Potentials of black and galvanized steel in 279-kg/m³ concrete (steam cured).
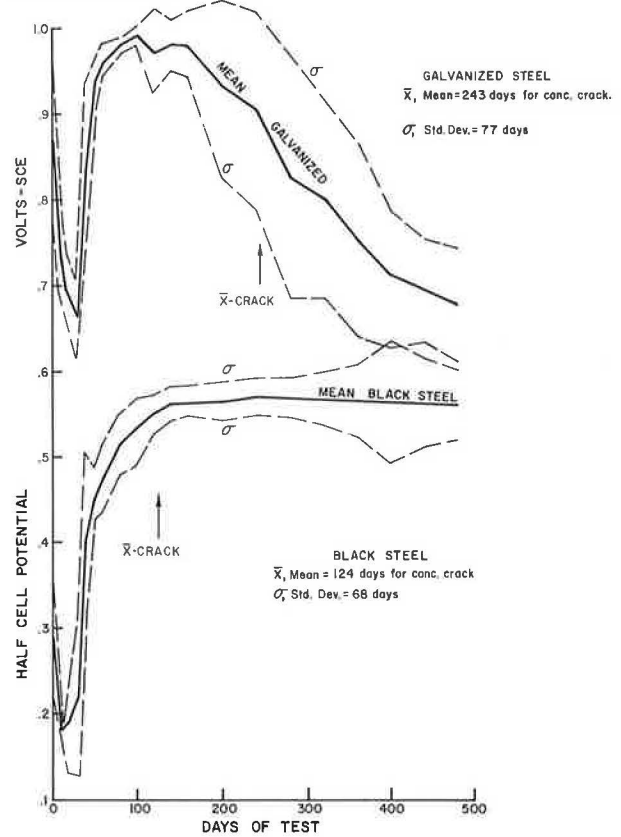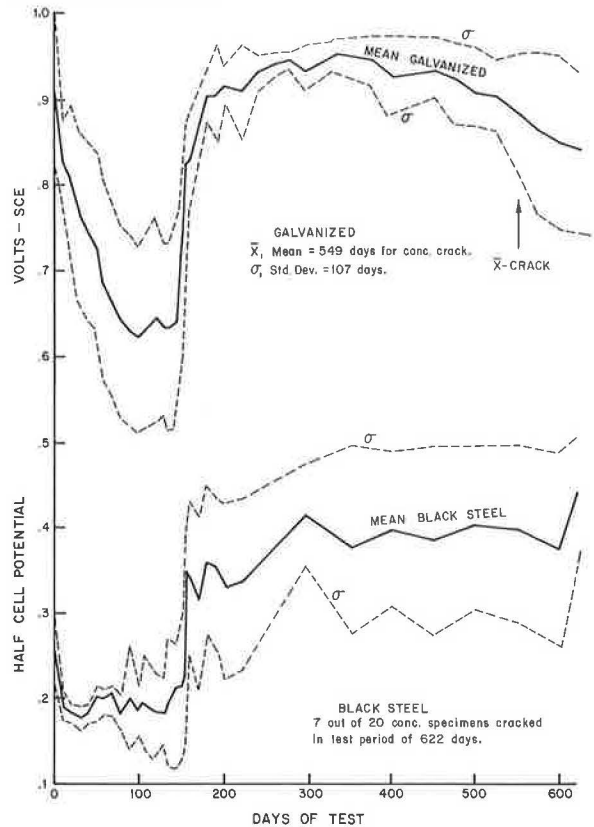

Figure 3. Potentials of black and galvanized steel in 418-kg/m³ concrete (moist cured).

aerated waters acts as an oxygen (O_2) electrode whose potential is noble to zinc and iron (15). A noble potential may accelerate the corrosion of metals with a less noble potential.

It was also reported that, when zinc is used as an anode in water or dilute NaCl, the current output decreases gradually because of the insulating corrosion products formed on the zinc surface. In one series of tests, the current between zinc and iron decreased to zero in 60 to 80 days, and a slight reversal of polarity was reported (15, p. 204).

Based on these reports by others, it is obvious that zinc will not always be a sacrificial metal and thus inhibit or reduce the corrosion of black steel.

In other studies it was found that, when zinc was placed in concrete, the high alkalinity of the cement caused it to react and evolve hydrogen gas (9, 17).

Table 3. Weight of zinc and concrete cracking.

Cement (kg/m^3)	Cure	Days to Concrete Cracking		Weight of Zinc[a] (g/m^2)	
		Average	Standard Deviation	Average	Standard Deviation
279	Moist	327	56	895.4	46.7
		302	48	1038.1	71.6
	Steam	248	82	873.7	41.5
		239	75	979.9	36.6
418	Moist	600	56	901.6	43.5
		498	123	1124.2	173.0
	Steam	390	69	916.0	55.4
		381	182	1137.1	106.9

Note: $1 kg/m^3 = 0.062 lb/ft^3$; $1 g/m^2 = 0.0033 oz/ft^2$.

[a]For 10 bars.

Figure 4. Potentials of black and galvanized steel in 418-kg/m^3 concrete (steam cured).
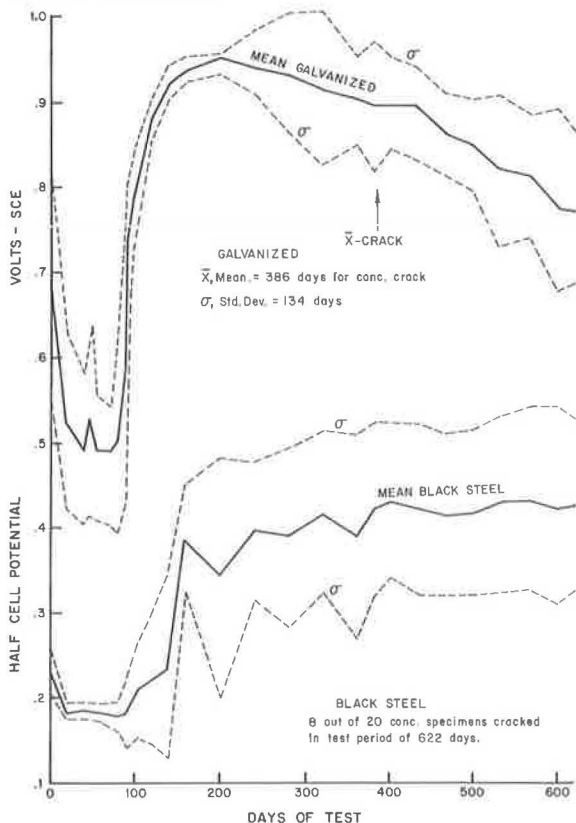
When mixing cement with distilled water, researchers found that the resultant pH was 12.8—well into the alkaline range—which is highly corrosive and causes hydrogen evolution from the zinc (9, 18). It has also been reported that, if more than 100 mg/kg of CrO_3 is added to the mixing water (17), the evolution of hydrogen gas will be inhibited.

In short time tests, it was reported that the corrosion rate of zinc is lowest at a pH range of 7 to 12 and is quite rapid when pH is 12.5 (15) or near the values of 12.6 or 12.7 (9).

In these tap water tests, black steel as a control for the galvanized steel specimens was omitted because, in previous tests of concrete with the same cement factor as that used in this test, the authors observed no corrosion of the bars. Also, because this test was to evaluate galvanizing it was originally thought that there would be no corrosion of the zinc and that the galvanized specimens would be a control for the specimens exposed to the salt. Such was not the case; however, the test of the galvanized specimens in tap water did demonstrate the wide range of half-cell potentials that could be expected in zinc, and the measured values are subject to a great deal more interpretation and investigation than anticipated.

SUMMARY AND CONCLUSIONS

Galvanized and Black Steel

When the steel-reinforced concrete specimens used in the tests were partially immersed in saturated sodium chloride solution, the chloride ion penetrated the concrete and caused both the galvanized steel and the black steel to begin to corrode at essentially the same time. In a relatively porous concrete, corrosion-caused concrete cracking required a longer time when the steel was galvanized than when it was not. In a high-quality struc-

Figure 5. Potential of galvanized reinforcing steel in 335-kg/m^3 concrete in tap water.
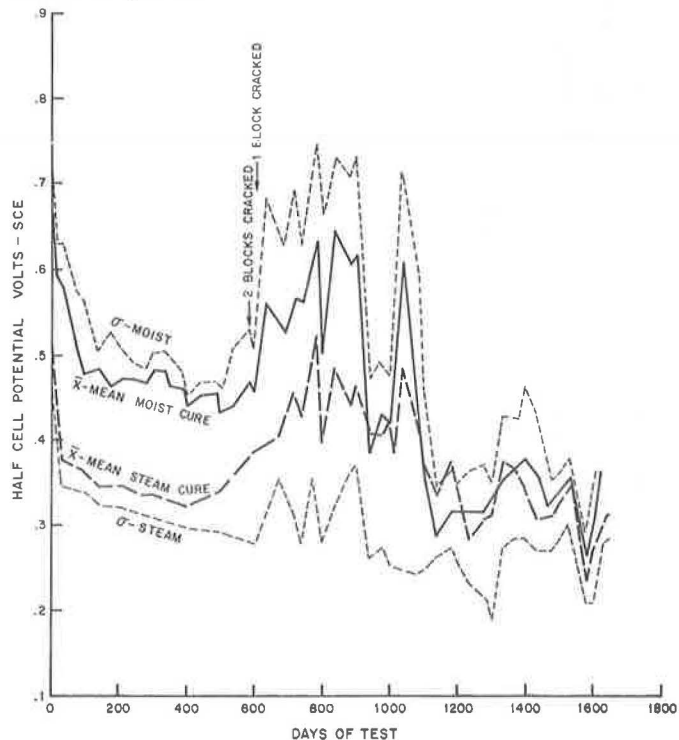
tural grade concrete, corrosion of the galvanizing has-
tened the time to cracking. It appears that the porosity
of the concrete can govern the time to cracking: The
greater the porosity is, the greater will be the amount
of zinc corrosion products that can be absorbed before
bursting pressures develop.

The test results showed that, irrespective of the con-
crete quality or method of curing used, thicker coatings
of zinc (up to 1420 g/m² or 4.65 oz/ft²) resulted in a
shorter time to corrosion-caused concrete cracking.
This test result indicates that the greater the amount of
galvanizing used, the greater will be the amount of cor-
rosion products available to cause concrete cracking.
This could indicate that the galvanizing corrodes rapidly
in salt-contaminated concrete. Otherwise, there should
be no difference in time to cracking for different thick-
nesses of zinc.

Electrical potential measurements indicate that gal-
vanized steel can assume electrically noble potentials in
low-salt concrete and initially serve as a cathode to
cause accelerated corrosion of both galvanized and black
steel in highly salt-contaminated concrete. The amount
of corrosion that occurs will depend on many variables
such as polarization and anode to cathode area. On the
basis of data plots, it was estimated that the lower mean
potential of galvanized steel in concrete exposed to tap
water was about -0.36 V SCE with a standard deviation
of about 0.12 V (Figure 5). However, it was not deter-
mined whether this or other potentials indicate a passive
or noncorroding condition for the galvanized steel. But
zinc and steel corrosion products were observed on the
surfaces of the steel bars. As a result, in concrete of
variable salt concentration, there does not seem to be a
definitive half-cell potential that would clearly indicate
an active (corroding) or passive (noncorroding) condition
of the zinc.

It appears that zinc behaves similarly to steel in salt-
contaminated concrete; it corrodes and causes concrete
cracking.

Zinc does not have a consistent half-cell potential in
concrete whereby it can be depended on to be a sacri-
ficial metal and protect steel from corrosion by galvanic
action.

In salt-contaminated concrete, the data (Figures 1
through 4) indicate that galvanized steel can have a half-
cell potential that ranges between -0.62 and -1.02 V
SCE (potentials within the limits of standard deviation).
It is assumed that such potentials indicate an active or
corroding state for the galvanized steel.

Also, the potential measurements show that galvanized
steel can, under some conditions, initiate corrosion of
itself in salt-free concrete. The potential of zinc in rel-
atively low salt-containing concrete can vary between
about -0.19 and -0.75 V SCE (Figure 5), which can result
in a galvanic voltage difference of 0.56 V to initiate local-
ized corrosion of itself. The amount and rate of corro-
sion will depend on many variables.

In tap water galvanized steel corroded initially in
relatively salt-free concrete and, after about 600 days
of test, caused three out of five moist-cured specimens
to crack (Figure 5). The five steam-cured specimens
did not have concrete cracking.

Concrete Curing

In this series of tests, as well as in those previously
reported (8,16), there is a more rapid penetration of
chloride into steam-cured concrete and earlier corrosion
of the embedded steel as compared to a water-cured con-
crete of the same quality.

Concrete Absorption

Concrete absorption or porosity, per se, indicated that
it had a significant effect on the results of the corrosion
behavior of galvanized steel. In highly absorptive con-
crete, the time to cracking caused by corrosion of gal-
vanized steel will differ greatly from that of black steel.
However, concrete absorption is not a reliable indicator
of the protective qualities of concrete when the value is
controlled by variables other than cement factor (8).

It is likely that the corrosion products are partially
absorbed by the adjacent mortar or aggregate in highly
absorptive concrete, which postpones an internal pres-
sure buildup by the corrosion products that can cause
rupture of the concrete.

REFERENCES

1. I. Cornet and B. Bresler. Corrosion of Steel and
   Galvanized Steel in Concrete. Materials Protection,
   Vol. 5, No. 4, April 1966.
2. D. A. Lewis. Some Aspects of the Corrosion of
   Steel in Concrete. First International Congress on
   Metallic Corrosion, London, April 1961.
3. D. F. Griffin. Effectiveness of Zinc Coatings on
   Reinforcing Steel in Concrete Exposed to Marine En-
   vironments. Civil Engineering Laboratory, U.S.
   Navy, Port Hueneme, Calif., Technical Note N-1032,
   July 1969; Second Supplement, June 1971.
4. C. E. Bird and F. J. Strauss. Metallic Coatings
   for Reinforcing Steel. Materials Protection, Vol. 6,
   No. 7, July 1967.
5. W. Halstead and L. A. Woodworth. The Deteriora-
   tion of Reinforced Concrete Under Coastal Condi-
   tions. Trans., South African Institute of Civil Engi-
   neering, April 1955, pp. 1-20.
6. B. Tremper, J. L. Beaton, and R. F. Stratfull.
   Causes and Repair of Deterioration to a California
   Bridge Due to Corrosion of Reinforcing Steel in a
   Marine Environment—Part II: Fundamental Factors
   Causing Corrosion. HRB, Bulletin 182, 1958, pp.
   18-41.
7. Concrete Bridge Deck Durability. NCHRP, Synthe-
   sis 4, 1970.
8. D. L. Spellman and R. F. Stratfull. Concrete Var-
   iables and Corrosion Testing. HRB, Highway Re-
   search Record 423, 1973, pp. 27-45.
9. C. E. Bird. The Influence of Minor Constituents in
   Portland Cement on the Behavior of Galvanized Steel
   in Concrete Corrosion Prevention and Control. July
   1964, pp. 17-21.
10. T. Ishikawa, I. Cornet, and B. Bresler. Electro-
    chemical Study of the Corrosion Behavior of Galva-
    nized Steel in Concrete. Proc., Fourth International
    Congress on Metallic Corrosion, Amsterdam, Sept.
    7-14, 1969, pp. 556-559.
11. R. F. Stratfull, W. J. Jurkovich, and D. L. Spell-
    man. Corrosion Testing of Bridge Decks. Paper
    presented at 54th Annual Meeting, TRB, 1975.
12. G. Schikorr. The Cathodic Behavior of Zinc Versus
    Iron in Hot Tap Water. Trans., Electrochemical
    Society, Vol. 76, 1939.
13. R. Hoxeng and C. Prutton. Electrochemical Behav-
    ior of Zinc and Steel in Aqueous Media. Corrosion,
    Vol. 5, 1949.
14. P. J. Gilbert. The Nature of Zinc Corrosion Prod-
    ucts. Journal of Electrochemical Society, Vol. 99,
    1952.
15. H. H. Uhlig. Corrosion and Corrosion Control.
    Wiley, New York, 1964.
16. R. F. Stratfull. Half-Cell Potentials and the Cor-
    rosion of Steel in Concrete. HRB, Highway Re-

search Record 433, 1973, pp. 12-21.

17. K. A. Christenson and R. B. Williams. Solving the Galvanic Cell Problem in Ferro-Cement. Tech. Rept. No. 2, NR Contract N 00014-69-A-0200-1007, Project NR 032522, July 1971.

18. B. E. Roetheli, G. L. Cox, and W. B. Littreal. Effect of pH on the Corrosion Products and Corrosion Rate of Zinc in Oxygenated Aqueous Solutions. Metals and Alloys, March 1932, p. 73.

19. H. Woods. Durability of Concrete Construction. ACI Monograph 4, 1968, p. 115.

20. R. F. Stratfull. A Report on the Investigation of the Concrete Embedded Steel in the Facilities Located at the U.S. Naval Station. U.S. Navy, March 1960.

Discussion

Daryl E. Tonini, American Hot Dip Galvanizers Association, Inc.

My comments refute the highly speculative and largely unsupported conclusions put forward by Hill, Spellman, and Stratfull. The comments address the following subjects: test conditions, zinc corrosion products, and coating thickness in some detail.

TEST CONDITIONS

Although they do not mention it in the test procedures, the authors do comment about the lack of control specimens in the experiment. Their rationale for omitting control specimens is unsatisfactory and most disturbing. Also, I am concerned about the variability introduced into the experiment by the use of 10 different batches of concrete, even when made to the same nominal specification. In addition, no mention is made of how the bars were assigned to the test prisms. It appears that they were assigned systematically and not randomly, as should have been required. Therefore, it seems that the design of this experiment may have been so poorly controlled at the outset as to raise serious doubts about the validity of the results and conclusions.

Besides, the test conditions themselves were apparently so poorly controlled that they produce extreme distortions of the real-world conditions that the authors should have attempted to simulate. A chloride level of 37.6 kg/m^3 (63.3 lb/yd^3) does not appear to be typical at bar level for either a full-scale structure such as a bridge deck or that found in other experiments of this type. In addition, differential cell effects generated under the exposure conditions used for this study appear to have been totally unaccounted for in the data presented in the paper.

The critical reader who is familiar with the work of other researchers in this field must question test conditions that led to the cracking performance found by the authors, particularly the black bar specimens, which apparently did not crack even after 622 days of exposure. For example, Clear and Hay (21), using "the utmost in [cement] quality control," found that, for "typical bridge deck concrete," i.e., cement factor of 7.0, w/c = 0.50, and 2.5 cm (1 in) of cover, the time to corrosion for black steel was only 1 week in most cases for a surface application of a total of only 0.6 kg (1.4 lb) of salt. For the test specimens cited by Clear and Hay, rust stains appear as soon as about 15 weeks and no later than 45 weeks. It would have been very enlightening had our

authors been more thorough and conducted further analyses. They would have then been able to give the reader some quantitative indication of the conditions that led to the highly unusual performance for black steel.

Also, from the data presentations adopted by the authors, it is evident that they assumed that the data on time to cracking were normally distributed and that they were therefore justified in using mean and standard deviation values in the presentation of the data. This assumption is open to serious question and must be justified before the arguments presented can be considered. Judging from the relatively large values presented for standard deviation, the authors should have felt impelled to examine the nature of their data more carefully before adopting such a statistical format for their results. Without a presentation of the authors' data, one can only speculate on the more precise nature of the statistics of the experiment. However, from the very nature of this type of experiment and from what is apparently typical of the statistics of related corrosion experiments, the assumption of normal distribution statistics is highly suspect.

ZINC CORROSION PRODUCTS

The authors suggest several times that the formation of zinc corrosion products generates bursting pressures, which have a direct relationship to the time to cracking. Neglecting for the moment the authors' unsupported correlation between a time to cracking criterion and the underlying corrosion mechanism, let us examine some of the features of the corrosion model that has been suggested. The authors state "... it is obvious that the corrosion products of zinc exert sufficient pressure to crack a 2.5-cm-thick (1-in) concrete cover." Unfortunately, what appears to be obvious to the authors was in no evident way measured in the experiment described. Even a casual reading of Reis, Mozer, Bianchini, and Kesler (22) or Houston, Atimtay, and Ferguson (23) should have dissuaded the authors from using such a simplistic model, particularly since they failed to support their assumption with data from the experiment. As further reinforcement for this point, Cornet and Bresler (24) show that a significant difference in time to cracking (as well as extent of cracking) in test prisms can be attributed to differences in rebar geometry (e.g., deformed or plain bars) and that cracking during the earlier stages of exposure can be unrelated to corrosion effects for either black or galvanized bars.

The authors also speculate, again without any indication that measurements were made, that zinc corrosion products migrate away from galvanized bars to account for their apparent improved performance in more porous concrete material. This assumption may be refuted with the authors' own data. From the absorption data given in Table 2 and Figures 1 through 4, it can be shown that the porosity data cannot always be correlated with time to cracking criteria. It can be demonstrated that the data presented for the 279-kg/m^3 and 418-kg/m^3 (5 and 7½-sack) mixes are consistent in this regard.

Further, when one examines the relative magnitude of changes in absorption that are suggested by the authors as the reason for differences claimed in performance as measured by time to cracking, one finds that, for the 279-kg/m^3 (5-sack) material, a change of only 1.5 percent in absorption appears to produce a 23 percent change in time to cracking. For the 418 kg/m^3 (7½-sack) material, a change of 3.1 percent in absorption appears to produce a 23 percent change in time to cracking, in the opposite direction. For a relationship that is so highly leveraged (even if it was consistent), I am curious about how accurately absorption was measured with regard to the relatively small magnitude of change that was cited.

Further, one is led to assume from the absorption testing temperature specified that the porosity measurements were made on samples prepared for that purpose and not made on the rebar test specimens. There is no mention of any controls to ensure that the concrete in the specimens was properly consolidated or of any measurements made of the unit weight of the specimens to determine how well consolidation was effected in the preparation of the samples.

How can the authors seriously suggest that their major conclusion can even be remotely ascribable to cement factor and absorption when they have apparently overlooked (or failed to control) the consolidation of their specimens? Clear and Hay (21) have convincingly demonstrated that a variation of as little as 5 percent in the in-place density can result in increasing chloride ion penetration through a 2.5-cm (1-in) concrete cover by a factor of 6.33.

It is also interesting to note that Stark and Perenchio (25) found no correlation between cement factor and galvanized rebar performance in any of the full-scale or simulated bridge deck structures they investigated. Inasmuch as scale factors in laboratory tests can often produce misleading results, particularly if the laboratory tests are poorly designed or executed, they generally cannot be given the same credence as tests on full-scale structures. Stark and Perenchio concluded that "... this investigation shows that galvanized steel clearly outperformed the untreated steel where a corrosive environment exists as defined by chloride ion content and condition of the steel."

COATING THICKNESS

The most anomalous assertion in the paper is that galvanized material with lighter coating weight performs better than does material with a heavier coating. This is particularly troubling since the conclusion is the result of a pseudo-statistical argument. It is supported only by speculation and not by any apparent analysis of the specimens themselves.

The conclusion regarding coating weight is based on a simple comparison of arithmetic means after the data were ordered into two populations based on coating weight. If valid, this comparison must satisfy at least a basic test for significance between the difference of mean values. Applying the universally accepted Student's t-test to the means clearly shows that something is very wrong with the authors' conclusion on coating weight. At a 99 percent confidence level (a common criterion for such testing), all the tests fail to show any statistically significant differences between the mean values used by the authors. At a 95 percent confidence level, three out of four comparisons fail to show any differences between the mean values cited. Based on this, the coating weight conclusion presented by the authors is seriously flawed and is rejectable. What is puzzling is why the authors did not include electroplated zinc bars with coating weights of a lesser order of magnitude into their test matrix. It appears that such an examination would have resolved this question much more directly than was attempted by the authors.

In the Stark and Perenchio report, there are no differences in galvanized rebar performance that could be attributed to coating weight. They were dealing with the almost exact same range of coating weights as were the authors.

The space available for these comments does not permit a detailed discussion of other defects in the Hill, Spellman, and Stratfull paper. However, a close examination of their arguments concerning potentials and potential reversal shows deficiencies of a magnitude

similar to those cited here. It is hoped that these topics will be addressed by other commentators.

In conclusion, the Hill, Spellman, and Stratfull paper is flawed to the extent that it should be essentially rejected and the experiment reperformed under controlled conditions.

REFERENCES

21. K. C. Clear and R. E. Hay. Time to Corrosion of Reinforcing Steel in Concrete Slabs: Volume 1—Effect of Mix Design and Construction Parameters. Federal Highway Administration, Rept. FHWA-RD-73-32, April 1973.
22. E. E. Reis, Jr., J. D. Mozer, A. C. Bianchini, and C. E. Kesler. Causes and Control of Cracking in Concrete Reinforced With the High Strength Steel Bars—A Review of Research. Univ. of Illinois, Urbana, T&AM Rept. 261, April 1964.
23. J. T. Houston, E. Atimtay, and P. M. Ferguson. Corrosion of Reinforcing Steel Embedded in Structural Concrete. Center for Highway Research, Univ. of Texas at Austin, Research Rept. 112-1F, March 1972.
24. I. Cornet and B. Bresler. Corrosion of Steel and Galvanized Steel in Concrete. Western Region Conference, National Association of Corrosion Engineers, Phoenix, Oct. 1964.
25. D. Stark and W. Perenchio. The Performance of Galvanized Reinforcement in Concrete Bridge Decks. International Lead Zinc Research Organization, Skokie, Ill., Project ZE-206, Final Rept., Oct. 1975.

Albert R. Cook, International Lead Zinc Research Organization, Inc.

Tonini's comments seem to be well founded. I too believe that important conclusions reached by the authors are wrong, that they are not supported by the data presented, and that in any case the data obtained cannot be relied on because the tests were poorly designed, not subject to comparison with control tests, and not supported by essential data concerning the materials used. Also, many of the data can be used to support entirely contrary conclusions.

I make these forthright comments because I believe it would be a pity if, because of this laboratory study, galvanized steel was neglected for bridge deck reinforcement. Galvanized reinforcement has been proved to extend the life of a wide range of reinforced concrete structures. Core tests of a 21-year-old bridge deck made by the Portland Cement Association showed that in concrete, with chloride levels above those considered aggressive to untreated steel, a galvanized coating was still protecting the basic steel, and between 60 and 75 percent of the original galvanized coating remained after 21 years.

The results reported in this paper are not consistent with previous laboratory tests nor with practical experience. It is reasonable therefore to question the test rather than the subject of the test.

Accelerated tests in general are notoriously inconclusive and are only indicative of what might happen under the conditions under which the test was made. A corrosion engineer will seldom rely on such data without having demonstrated that the kinetics of the reactions can at least be related to those taking place in practical situations.

Of paramount importance to the corrosion resistance

of galvanized reinforcing steel is the nature of the galvanized coating. No data have been provided concerning this important variable. The authors have, however, pointed out that a wide variation in coating weight was noted. This indicates lack of control of the galvanizing operation or inadequate control of the composition of the basis steel or both.

Let us consider the summary and conclusions. The authors state, "The greater the porosity [of the concrete] is, the greater will be the amount of zinc corrosion products that can be absorbed before bursting pressures develop." No data are presented that show the porosity of the concrete. We cannot agree with the view that a 10 percent difference in absorption, which the authors noted between a mix containing 279 kg/m³ (5-sack mix) and a mix containing 418 kg/m³ (7½-sack mix), considered in the absence of permeability data, justifies the speculation that this indicates a difference in porosity that could account for differences in pressure, which in turn are caused by an unknown amount of corrosion product.

Concrete cracking is caused by a number of factors unrelated to corrosion of the reinforcement. The authors could have examined the galvanized specimen after testing to confirm the presence and composition of zinc corrosion products and determine the degree of any zinc absorption. Instead, they were content to rely on pure conjecture concerning this fundamentally important point.

The authors also state that thicker coatings of zinc result in a shorter time to cracking. They write that this could indicate that galvanizing corrodes rapidly in salt-contaminated concrete. They ignore the evidence in the literature that the diameter of any reinforcing bar, the depth of cover, and the time to cracking are related. It seems more likely that the effect reported is due to geometry rather than to any special corrosion susceptibility of the zinc.

The authors appear to be relating the open-circuit potentials that they have seen at different times for zinc in low-salt concrete and the open-circuit potentials of steel in lightly salt-contaminated concrete. However, it is closed-circuit potential, among other things, that is important to the course of any corrosion and not open-circuit potential. They correctly state that any corrosion that might occur will depend on many variables. It would be more realistic if they had pointed out that these variables are so poorly defined by their available data that their remarks are entirely speculative. In my view, such far-fetched speculation has no place in such a paper.

A series of papers by Gouda and Mourad (26, 27, 28) stresses the effect of differences in pH, salt concentration, surface condition, carbon dioxide, and oxygen concentration on the corrosion of untreated (black) steel reinforcement. Hill, Spellman, and Stratfull attempted to simulate, in an accelerated manner, the corrosion of reinforcing steel in a bridge deck by taking concrete specimens 38 by 6.4 by 11.4 cm (15 by 2½ by 4½ in) and placing them to act as a wick, immersed to a depth of 8.9 cm (3¼ in) in concentrated salt solution. This salt solution was able to evaporate on penetrating the specimen, carrying with it usual amounts of dissolved carbon dioxide and oxygen, and also it was able to build up a level of salt contamination and of chloride concentration cells far more severe than would have been likely in practice. All this produces conditions in no way comparable to what might have occurred in practice.

Also, the quality control exercised in the investigation can be questioned. Similar specimens were placed in tap water, and, even in tap water, the specimens built up 1.9 kg/m³ (5.6 lb/yd³) of chloride ion. They found that atmospherically exposed concrete specimens contained 0.55 kg/m³ (1.6 lb/yd³) of chloride. Where did

all this salt come from? In one northern state, about 2.42 kg/m³ (7 lb/yd³) of chloride ion is the maximum expected. Clear and Hay (21) suggest that 0.38 to 0.45 kg/m³ (1.1 to 1.3 lb/yd³) of chloride ion is the threshold level for chloride-initiated corrosion of untreated reinforcing steel in concrete. Yet the authors refer to their specimen as being relatively salt free. The presence of these unexplained large amounts of chloride in the tap water specimens and in the atmospherically exposed specimens justifies serious doubt about the manner in which the concrete was prepared. Furthermore, the consolidation of the mix can be critical to the behavior of the concrete and its reinforcement. Equal consolidation of specimens is therefore vital to ensure the validity of comparative tests. This variable was not judged worthy of mention. Furthermore, to ensure comparable results of course requires that the concrete be uniform in composition from one specimen to another, and yet 10 separate mixes were prepared for the mixes containing 279 and 418 kg/m³ of cement (5 and 7½ sack).

Regarding paragraph 4, galvanized steel is generally accepted as an excellent protective coating for steel. In 1973, 1.5 Tg (1 535 000 metric tons) were used for this purpose. Those wise enough to make use of it in this fashion would not quarrel with the authors' statement, "Zinc behaves similarly to steel.... It corrodes...." It has however been shown to increase the life of the steel out of all proportion to the increased cost incurred, and that is why such a tremendous amount of zinc is used for corrosion protection.

The authors suggest that zinc may reverse its polarity and cause accelerated corrosion of black steel. Given the miniscule amount of data relating to half-cell potentials that they feel justifies placing such a suggestion in the open literature and given the findings of others that in the presence of chloride ion, or calcium or silicate ions, no such reversal has been seen, and given that cases of reversal of zinc potential in practice have been involved with domestic water falling within certain limits of composition and usually with elevated temperature and under pressure of oxygen, I can only characterize their statements as irresponsible.

It is entirely beyond reason to draw any analogy between corrosion processes taking place under neutral conditions in domestic water and what might take place in concrete. Even in the case of distilled water, two investigators (29) concluded that "The cathodic depolarization arising from oxygen at atmospheric pressure is insufficient to bring about polarity reversal of zinc-steel couple" and that "The presence of chloride ions in solution reduces any tendency towards enoblement of zinc." The authors covered 3 to 5 ppm (Cl⁻) and 100 ppm (Cl⁻).

Also in the summary, the authors "assume" that, based on half-cell potential, the galvanized steel in their salt-contaminated concrete is in an active or corroding state. Yet in a previous paragraph they state that this was "not determined."

I confess that I am unable to comprehend what the authors are getting at in the last paragraph of their results. Do they seriously suggest that, for example, when a rebar with 7.92 g/m² (3 oz/ft²) of zinc showed 390 days to concrete cracking with a standard deviation of 69 days and a rebar with 9.85 g/m² (3.73 oz/ft²) showed 381 days to concrete cracking with a standard deviation of 182 days, these data can be used to help form any conclusion whatsoever? Surely impartial examination of this set of data, given the limits of error noted, cannot bolster the authors' view that galvanized bars with heaviest galvanizing will crack first.

We feel that publication of this paper may cause wrong conclusions to be formed by readers who are not able to assess the questionable statistical methods used and who

may be unused to evaluating the extent to which laboratory tests can be applied in practice.

We prefer to rely on the many other tests and long practical experience to support the use of galvanized steel in concrete. One example would be the 2-year tests and practical experience that justified the statement by an area engineer in Bermuda: "The use of fresh water, Stateside aggregate, adequate cover and galvanized steel is required to provide concrete in Bermuda guaranteed to have no rust," and it should be borne in mind that a good deal of reinforced concrete in Bermuda is exposed fully immersed or partially immersed in seawater.

REFERENCES

26. V. K. Gouda and H. M. Mourad. Differential pH Cells. Corrosion Science, Vol. 14, 1974, pp. 681-690.
27. V. K. Gouda and H. M. Mourad. Differential Salt Concentration Cells. Corrosion Science, Vol. 15, 1974, pp. 307-315.
28. V. K. Gouda and H. M. Mourad. Differential Surface Condition Cells. Corrosion Science, Vol. 15, 1974, pp. 317-328.
29. D. J. Hubbard and C. E. A. Shanahan. Corrosion of Zinc and Steel in Dilute Aqueous Solutions. British Corrosion Journal, Vol. 8, Nov. 1973.

I. Cornet, University of California, Berkeley

The paper by Hill, Spellman, and Stratfull presents information that demonstrates the need for further research on corrosion of metals in concrete.

A few minor items may be noted. In Table 3 the 279-kg/m^3 (5-sack) concrete steam-cured specimens average 275 and 239 days, which would give an arithmetic mean of 257 days to concrete cracking. Figure 2, however, gives a mean of 243 days. For the same Table 3, the 418-kg/m^3 (7$^1\!/_2$-sack) concrete moist-cured specimens average 600 and 498 days, which would give an arithmetic mean of 549 days to concrete cracking. Figure 3 gives a mean of 475 days. For purposes of this discussion it is assumed that table values are correct, and an arithmetic mean is intended in the figures.

The authors are perhaps unduly negative and pessimistic about their findings. Their data show substantially a two to one improvement in performance of galvanized steel over black steel in prism specimens of 279-kg/m^3 (5-sack) concrete exposed to saturated sodium chloride solution. Moreover, there is clear indication that an optimum thickness of zinc coating might give even greater improvement over black steel, as shown in Figure 6. Black steel is 0 g/m^2 (0 oz/ft^2) in this figure. For the black steel each observed point is the average of 20 specimens; for the galvanized, each observed point represents 10 specimens. Predictions of the average days to cracking for black steel specimens are taken from earlier publications by Spellman and Stratfull (8, 30), based on data shown in Figure 7. Observe that there is excellent agreement between the experimental observations and the predicted time to cracking for the 279-kg/m^3 (5-sack) concrete specimens.

The authors are to be congratulated on demonstrating a two to one improvement in corrosion resistance for the 279-kg/m^3 (5-sack) concrete. Such concrete may be regarded as average for construction practice. Control of water-cement ratio may be very important in securing

best performance, but practical improvements and economies may be obtainable by galvanizing. The authors should publicize this work on the 279-kg/m^3 (5-sack) concrete.

It is more complicated to analyze the performance of galvanized steel in the 418-kg/m^3 (7$^1\!/_2$-sack) concrete. Figure 6 shows clearly that there may be an optimum thickness of galvanizing to give greater corrosion resistance in concrete. Obviously there is need for more research on the effect of thickness of zinc on corrosion resistance of reinforcement. Comparisons between galvanized and black steel reinforcement require prediction of when the black steel reinforced specimens would have cracked, because tests were terminated before all of the black steel specimens had cracked. Fortunately Spellman and Stratfull have provided a prediction equation for the average days to concrete cracking:

$$C = 1.12P + 115 \tag{1}$$

where P = average days to active potential. The predicted C = 289 days is shown in Figure 6. Incidentally, Spellman and Stratfull state that "the time to the active potential of steel in concrete is mathematically related to the time to concrete cracking due to corrosion" (8). They furthermore state that "visual observations not only are of questionable accuracy depending on the observer but also are a more time-consuming and expensive procedure than is the measuring of half cell potentials" (8). Accordingly, one can place considerable confidence in the predicted time to cracking of concrete due to corrosion of black steel.

It is not really necessary to predict the time for cracking on the 418-kg/m^3 (7$^1\!/_2$-sack) concrete with black steel. Spellman and Stratfull have published data for a 446-kg/m^3 (8-sack) concrete with no admixtures, for both moist and steam curing. Days to concrete cracking reported for the 446-kg/m^3 (8-sack) concrete are all confined to specimens cast vertically. However, Spellman and Stratfull (30, p. 14) state that "the effect of orientation of the steel to the time to an active potential appears to be relatively minor." They also state that "the time to the active potential... is mathematically related to the time to concrete cracking due to corrosion" (30, p. 33). The data are given in Table 4, and the points for moist cure are shown in Figure 7.

The galvanized reinforcement does better than the black in the statistics given in Table 4, but it is not clear why the two to one improvement in performance found for the 279-kg/m^3 (5-sack) concrete is not maintained in the 418-kg/m^3 (7$^1\!/_2$-sack) concrete. Incidentally, a 279-kg/m^3 (5-sack) concrete even with a 0.63 ratio of water to cement can hardly be called porous, particularly in the absence of permeability data. An air content of 2$^1\!/_2$ percent falls in the range of normal nonporous concrete.

If Hill, Spellman, and Stratfull had continued the tests until cracking for black reinforced concrete with the 418-kg/m^3 (7$^1\!/_2$-sack) concrete, the average data points might have fallen far from the prediction equation line shown in Figure 7. This might have happened if something had gotten into the concrete. For example, a 335-kg/m^3 (6-sack) concrete with admixture No. 1 falls way off the curve. Or the visual observations of cracking may have been of questionable accuracy, as stated by Spellman and Stratfull (8). If one did rely on such observations, one would, in effect, be discarding entirely the concept developed by Spellman and Stratfull (8) that "the time to active potential of steel in concrete that is partially immersed in a saturated sodium chloride solution is mathematically related to the time to concrete cracking due to corrosion."

It is appealing to have such a useful concept. As

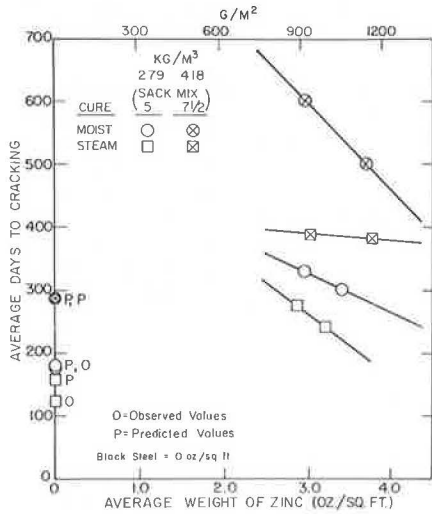Figure 6. Time to cracking versus weight of zinc.

Figure 7. Days to concrete cracking versus days to active potential (black steel).
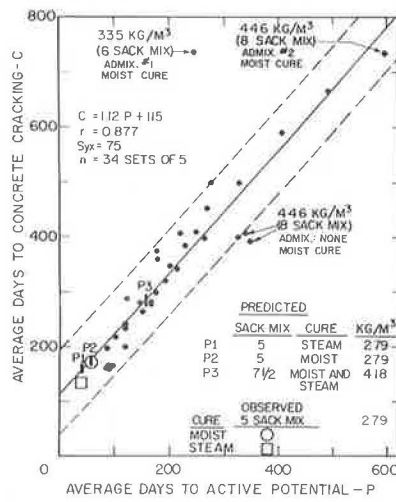

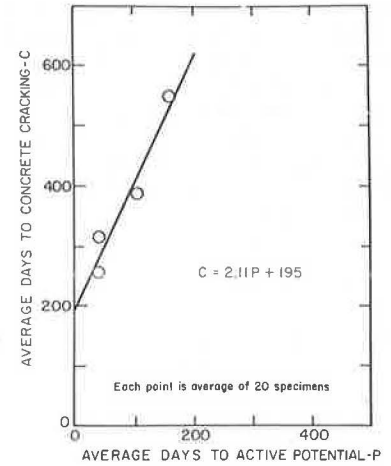Figure 8. Days to concrete cracking versus days to active potential (galvanized steel).

Table 4. Average days to active potential and concrete cracking.

Steel	Concrete (kg/m³)	Cure	Observed Days to Active Potential	Days to Concrete Cracking	
				Observed	Predicted
Black	279	Moist	55	175	177
		Steam	40	124	160
	418	Moist	155	622	289
		Steam	155	622	289
	446[a]	Moist	348.6, 339.0	390.8, 406.8	505.4, 494.7
		Steam	212.4, 174.8	341.2, 296.8	352.9, 310.8
Galvanized	279	Moist	40	315	279
		Steam	40	257	279
	418	Moist	160	549	532
		Steam	105	387	417

Note: 1 kg/m³ = 0.062 lb/ft³.

[a]For slumps of 5.1 and 10.2 cm (2 and 4 in).

Spellman and Stratfull have shown, one can save time and money in laboratory tests by obtaining a potential-time history, noting when the potential of the black steel becomes active and calculating the time to concrete cracking by using equation 1. An attempt to obtain such an equation for galvanized steel is shown in Figure 8. Figures 1 through 4 show that galvanized steel starts out at a potential of -0.7 to -1.0 V SCE in these tests. The galvanized steel passivates in a few days and reaches a potential of -0.5 to -0.75 V SCE. The chloride ion then permeates the concrete. In a few weeks, the chloride ion reaches the reinforcement and attains a sufficiently high concentration to break down the passivity. The potential then rises to become active at -0.80 to -0.85 V SCE. This is an arbitrary selection from the curves, but it does permit us to obtain a time to research active potential and to derive a correlation equation:

$$C = 2.11P + 195 \qquad (2)$$

Note that galvanized steel has some tendency to repassivate or at least to change potential considerably after some period of corrosion. Note also that Hill, Spellman, and Stratfull present a complicated picture in Figures 1 through 4. These are mixed potentials for zinc, zinc-iron alloy, and steel in chloride solution in concrete, and the curves are averaged curves. One must be cautious in going from such averaged data to an individual case.

It is regrettable that the authors could not at this time give information on their tests of corrosion of zinc electroplated steel reinforced concrete prisms exposed to saturated sodium chloride solutions. These were designed to have 152, 305, and 610 g/m² ($\frac{1}{2}$, 1, and 2 oz/ft²) of zinc. Apparently tests were discounted when the specimens failed to crack when they were expected to. Specimens that were cut open were found to have corrosion products other than the white zinc corrosion products anticipated. Still it might be illuminating to give details on these tests. We understand that the coatings may have been cadmium rather than zinc, which unfortunately was discovered long after the tests were terminated. Under these circumstances it should be noted that the authors did not complete the program they originally planned, and further work is required.

Statements on porosity are highly speculative. No porosity or permeability was measured. Table 2 shows only a 10 percent difference in absorption between the 279 and 418-kg/m³ (5 and 7½-sack) concrete. A 279-kg/m³ (5-sack) concrete with 0.63 water-cement ratio can hardly be called porous, particularly in the absence of permeability data.

There are misleading speculations that galvanized steel can assume noble potentials in low salt concrete and cause accelerated corrosion in highly salt-contaminated concrete. The authors are discussing a chloride concentration cell in which either steel or galvanized steel in a low chloride concentration will be noble relative to steel or galvanized steel in concrete that is highly contaminated with salt.

The authors do not show any instance in which the zinc would not be sacrificial to steel in salt-contaminated

concrete. They also confuse open-circuit potentials with polarized potentials that would exist if current were flowing.

They speak of tests in tap water. They had 0.95 kg/m³ (1.6 lb/yd³) of chloride ion in the atmospherically exposed concrete. They note that the source of the chloride ion in the concrete was not determined. The concrete had 3.33 kg/m³ (5.6 lb/yd³) of chloride ion in sections immersed in tap water. It is not obvious how tap water with 20 to 40 ppm of chloride ion could contaminate concrete with so much salt. It is misleading to call this a tap water test.

Regarding concrete absorption, the 279-kg/m³ (5-sack) concrete is not "highly absorptive," nor are there any test data given that bear on the porosity per se.

In their results, specimens were partially immersed in Sacramento City tap water initially containing about 20 to 40 ppm of chloride. The authors should note that none of the steam-cured specimens cracked in the tap water in 1700 days of exposure. This is somewhat inconsistent with other data reported in which steam-cured specimens lasted only three-fourths as long as the moist-cured specimens before cracking.

The authors state in their results that there does not appear to be any indication that differences in concrete absorption affected any comparative results. This makes it particularly difficult to understand their emphasis on porosity.

In evaluating the relationship of zinc thickness to corrosion resistance, the authors present mean values and overlook the wide variation of values about the mean. The standard deviation indicates considerable overlap in days to concrete cracking for thick- and thin-coated specimens, and the present data may be of questionable statistical validity. If, however, further testing confirms the trends claimed by the authors, it is likely that there can be an optimum thickness of galvanizing that could provide more than twice the crack-free life of the black steel reinforcement.

In the discussion of the results, much information is highly speculative and not supported by test data. The authors confuse a concentration cell effect, low chloride versus high chloride concentrations, with "reversed polarity of zinc." They also confuse open-circuit potentials with polarized potentials.

The discussion of reversal of polarity of zinc is particularly erroneous and highly misleading since the authors themselves have referred to a publication by Hoxeng and Prutton (13). In aerated solutions, bicarbonates and nitrates promote cathodic zinc potentials, particularly in potable waters at temperatures above 60°C (140°F). This reversal in potential does not occur in chloride concentrations of more than about 30 ppm. Silicates also inhibit the reversal of potential. It is farfetched and misleading to liken the alkaline solution in salt-laden concrete to a potable water at elevated temperatures.

It should be noted that the corrosion rate of zinc is at a minimum when the pH is about 12.5.

The authors produce a control series for their tap water tests that is quite unconvincing. No chloride analyses are presented for the earlier tests, which lasted over a period of about 3 years compared to the present 1700-day test. Nor is there any explanation of why the steam-cured galvanized specimens failed to crack in 1700 days.

REFERENCES

30. D. L. Spellman and R. F. Stratfull. Laboratory Corrosion Test of Steel in Concrete. Materials and Research Department, California Division of Highways, Research Rept. M&R 635116-3, Sept. 1968.

Authors' Closure

In the closure below, the authors comment on specific statements made by the discussants. Our response follows their comments.

DISCUSSION BY TONINI

The lack of control specimens is unsatisfactory.

Control specimens were used.

The use of 10 batches of concrete raises serious doubts about the validity of the results.

When more than one batch is used, then any effect of a singular batch of concrete is eliminated.

The assumption that the data were normally distributed is open to serious question and must be justified in using the mean and standard deviation.

We observed that Tonini used the Student's t-test to evaluate the difference in our reported arithmetic means without justifying the type of distribution curve.

It is not obvious that zinc corrosion products can crack concrete. Cornet and Bressler (24) show that cracking during earlier stages of exposure can be unrelated to corrosion effects for either black or galvanized bars.

Cornet and Bressler stated: "Specimens stored in air showed no cracks or rust stains after 24 months' exposure."

It is assumed that absorption testing was made on samples prepared for that purpose and not on the rebar test specimens.

Correct! We overlooked to mention that 15 by 15-cm (6 by 6-in) concrete specimens were obtained from the test batches to measure absorption.

How can the authors seriously suggest that their major conclusion can even be remotely ascribable to cement factor and absorption when they have overlooked (or failed to control) the consolidation of their specimens?

We neither overlooked nor failed to control the consolidation of the concrete. All concrete was prepared in the laboratory and consolidated by means of vibration and checked by unit weight. Cement factor as a means to reduce water-cement ratio has been well established as a control on the time to corrosion of steel in concrete.

Stark and Perenchio (25) found no correlation between cement factor and galvanized rebar performance in any of the full-scale or simulated deck structures they investigated.

The simulated deck structures on which they reported had the same cement factor (6 sack), so no relationship could possibly be derived.

Stark and Perenchio (25) concluded that "...this investigation shows that galvanized steel clearly outperformed the untreated steel where a corrosive environment exists as defined by chloride ion content and condition of the steel."

Stark and Perenchio did not compare the corrosion be-

havior of untreated and galvanized steel in bridges.

In the Stark and Perenchio report (25), there are no differences in galvanized rebar performance that could be attributed to coating weight.

Stark and Perenchio did not report any attempt to relate zinc thickness to performance.

The test conditions were so poorly controlled that they produced extreme distortions of real-world conditions. A chloride level of 37.5 kg/m³ (63.3 lb/yd³) does not appear to be typical at bar level for either a full-scale structure or that found in other experiments of this type.

The chloride content was found in the submerged section of about a 6.4-cm-thick ($2\frac{1}{2}$-in) concrete sample. We reported (31) the chloride content of a bridge pile exposed to salt water after about 40 years to be 22 kg/m³ (38 lb/yd³).

DISCUSSION BY COOK

Core tests on a 21-year-old Bermuda bridge by the Portland Cement Association showed that galvanizing was still protecting the underlying steel even though the chloride level was above that considered aggressive to untreated steel.

The one concrete core obtained from this bridge showed 0.9 kg/m³ (1.68 lb/yd³) of chlorine at the level of the steel. No report was made on the performance of untreated steel in this bridge. Based on the low level of salt reported for the one core after 21 years of exposure, it may be that Bermuda is a relatively noncorrosive environment as compared to the environment of most bridge decks subjected to deicing salt.

A large variation in coating weight was reported, which indicates a lack of control of the galvanizing operation or inadequate control of the composition of the steel or both.

The coefficient of variation of zinc coating weight was about 7 percent, which we consider to be small. We have normally seen greater variations of weight on galvanized steel construction materials.

The authors ignore the evidence that the diameter of the bar, the depth of cover, and the time to cracking are related. It seems likely that the reason that thicker coatings cracked earlier was geometry rather than any special corrosion susceptibility of the zinc.

The only known difference between the bars was the average thickness or weight of the zinc. If diameter of the bar was of paramount importance, then the thinner zinc-free bars should have consistently performed better than galvanized steel.

It is the closed-circuit potential that is important to the course of corrosion and not the open-circuit potential.

The closed-circuit potential can have less meaning than the open-circuit potential because it is influenced by current flow and polarization; the open circuit does not contain these variables.

The authors suggest that zinc may reverse its polarity. This statement can only be characterized as irresponsible.

We presented data that showed the distinct possibility of polarity reversal in certain cases and cited literature references in which this was found to occur.

DISCUSSION BY CORNET

The mean time to cracking of 7½-sack moist-cured specimens in Table 3 does not agree with that shown in Figure 3.

We have made the necessary correction, which does not alter the conclusions.

For the purpose of this discussion, it is assumed that table values of mean and arithmetic mean are correct, and an arithmetic mean is intended in the figures.

The average, mean, and arithmetic mean are equivalent and all designate the same numerical value.

There is a clear indication that there is an optimum thickness of zinc coating that might give even greater improvement over black steel.

If Cornet is correct regarding an unknown optimum thickness of zinc and variation from optimum is significant, then it is obvious that those who use galvanized rebars in concrete may be endangering the longevity of their structures. Specifications contain no limits for a maximum thickness of zinc, and, as received, it may be too thick to provide maximum or optimum life.

The authors are to be congratulated on demonstrating a two to one improvement in corrosion resistance of the zinc in the 5-sack concrete.

The data did not demonstrate or imply that zinc showed an improvement in corrosion resistance over black steel. Concrete cracking resulting from corrosion of reinforcement is related to its strength and absorptive properties and depends on the contingencies of the formation of corrosion products. Corrosion is the electrochemical behavior of the metal. The data indicate that, in all the concretes studied, galvanized and black steel began to corrode at substantially the same time. Therefore, there is no implication in the data that there is a significant difference in corrosion resistance of zinc-coated steel as compared to that of black steel in a chloride-contaminated concrete. The embedment of zinc or steel does not affect the rate of salt penetration into the concrete.

Comparisons between galvanized and black steel require prediction of when the black steel reinforced specimens would have cracked, because tests were terminated before all of the black steel specimens had cracked.

Cornet's calculations of the predicted time to concrete cracking are incorrect because the equation used was not derived on the basis of the time to a potential of -0.35 V. The equation was derived on the basis of that point in time when the potential of the steel first shifted from the passive to the active. The half-cell potentials in this report are referenced to a saturated calomel half cell (SCE). An active potential of steel referenced to the latter half cell would be -0.27 V (8). Cornet also used the wrong value in his calculations to designate a corrosive potential for steel in concrete.

Five-sack concrete may be regarded as average construction practice.

We do not agree that 5-sack concrete should be regarded as an average construction practice. For California Department of Transportation structures, we require minimums of 6-sack concrete in noncorrosive and 7-sack in corrosive environments. We are aware, however, that much commercial building is done with concrete having a cement factor of about 5 sacks.

Galvanized steel in this test does better when compared to the published statistics for black steel in 8-sack concrete.

When Cornet used data from a previous report, he reached erroneous conclusions because he apparently overlooked half of the data. He states that, in the previous report (30), the average time to concrete cracking for an 8-sack concrete was 390.8 and 406.8 days for 5.1 and 10.2-cm (2 and 4-in) slump. However, this was for the vertically cast specimens. Given in the same table of that report is horizontally cast specimens (the same casting method used in this test), which did not crack in 798 days. In this latter case, Cornet is in error because the galvanized steel specimens had an average time to cracking of 549 days.

Concrete with a water-cement ratio of 0.63 can hardly be called porous.

Permeable or absorptive might have been a better term to use in this case. In the paper, the term porous was used in the context of the ability of the concrete to inhibit the penetration of corrosive chlorides. For piles, architectural concrete, pipes, and rails exposed to fresh water and mild temperatures, ACI Standard 613-54 recommends a maximum permissible water-cement ratio of about 0.49. Therefore, Cornet's ststement does not agree with normal concrete standards.

A concrete mix containing 2½ percent entrapped air falls in the range of normal nonporous concrete.

We have reviewed almost all of the references to publications by the American Concrete Institute and can find no one who has related the amount of entrapped air to concrete porosity. (One to $1\frac{1}{2}$ percent of entrapped air is probably more normal.)

A 6-sack mix with admixture No. 1 (a water-reducing agent) falls way off the curve in the previously reported data (30). This may indicate that something might have gotten into the 7½-sack concrete in this test.

Again, Cornet seems to have overlooked some of the data. The data that fell off the curve were the vertically cast concrete. The time to cracking of the horizontally cast concrete (30) fell within the limitation of the standard error of estimate. Nothing got into the concrete.

An arbitrarily chosen potential of -0.80 V SCE is indicative of the corrosion of galvanized steel.

We do not disagree that -0.80 V SCE as an indicator of corrosion of galvanizing is an arbitrary assumption. This is in view of the potentials of galvanizing measured in the tap water tests. In these latter tests, corrosion of zinc and underlying steel was observed even though the half-cell potential, within one standard deviation, did not attain a value of -0.80 V. It was for this reason that we stated in the paper that we did not observe a definitive half-cell potential that would clearly denote an active (corroding) or a passive (noncorroding) condition of the zinc. Therefore, we view Cornet's derived equation and plots also to be arbitrary because his derivations are based on a half-cell potential for zinc that is not verified by test data.

The authors are perhaps unduly negative and pessimistic about their findings.

Our finding that galvanizing in concrete was apparently not cost beneficial is supported by others. For example, Bird and Strauss (4) from South Africa, reporting on their experimental results with galvanized steel in concrete, state: "In the presence of 1 percent salt (by weight of cement), however, sacrificial attack in the vicinity of exposed steel increases by approximately 30

times, while the rate of self-corrosion is accelerated even more." Griffin (3), reporting on his sea salt spraying of concrete panels at the U.S. Naval Civil Engineering Laboratory, stated: "The air-entrainment provided more protection to the concrete than did the zinc coating on the steel. In no case did the zinc coating prevent the formation of red rust."

REFERENCE

31. D. L. Spellman and R. F. Stratfull. Chlorides and Bridge Deck Deterioration. HRB, Highway Research Record 328, 1970, pp. 38-49.

Cathodic Protection of Bridge Decks: A Study of Three Ontario Bridges

H. J. Fromm and G. P. Wilson, Ontario Ministry of Transportation and Communications, Downsview

Many concrete bridge decks are being damaged by surface spalling or internal delaminations caused by corrosion of the reinforcing steel. Cathodic protection can be applied to bridge decks to stop this type of damage. Cathodic protection was applied to three bridge decks in Ontario: two slab decks on AASHTO beams and a voided, posttensioned structure. Each deck was equipped with resistance probes, which showed that the corrosion of the reinforcing steel was stopped when cathodic protection was applied. The bridges were studied to determine the minimum potential required and the most advantageous electrode configuration and spacing for adequate protection. The technique for measuring the polarized potential on the steel was studied, and probes buried in the conductive layer were found to be more effective than half-cell measurements. Data for all three bridges are presented. The protection on the first two of these bridges has been operated successfully for 1 year, and that on the third bridge has been operated successfully for 9 months.

Some exposed concrete bridge decks in Ontario have begun to show signs of deterioration in the form of spalling of the concrete over or under some of the reinforcing bars. Closer examination of these decks showed that delaminations were also present within the deck. These problems were caused by corrosion of the reinforcing steel and the resultant buildup on the bar of corrosion products, which exert pressure on the concrete and cause it to rupture.

This corrosion is caused by deicing salt solutions entering the concrete and eventually reaching the steel bars. This may occur even with high-quality, high-strength concretes. A review of the literature on this subject is given by Stratfull (1). Another study concerning the corrosion of steel in bridges has been reported by Moore (2).

Reinforcing steel in concrete is normally in a noncorroding, passive condition. The pH of normal concrete ranges from 12.5 to 12.8 (3, 4). In this pH range, steel is essentially passive (4, 5, 6). When salt solutions, either sodium chloride or calcium chloride, enter the concrete and reach the level of steel, however, corro-

sion of the reinforcing steel usually results. Gouda (6) has shown that alkaline solutions in which steel is normally passive become corrosive when sodium chloride is present in certain concentrations. Salt solutions reduce the pH of the concrete to about 11.5 (4), at which level the steel is no longer passive. Distressed bridge decks that have been investigated by the ministry have always been found to contain considerable quantities of chloride at the level of the steel reinforcing bars (7). This effect has been found by others (8, 9). To stop the corrosion requires that either the chloride ion be removed or that the corrosion reaction be inhibited by some other means.

One method that has been available for years to prevent corrosion when a corrosive medium is present is cathodic protection (1, 10, 11, 12). This method was first used by Stratfull (13) when he applied it to the reinforced concrete beams of the San Mateo Bridge over San Francisco Bay. Stratfull has since applied this technique to part of the deck of the Sly Park Bridge near Sacramento (1). The apparent success of this first application warranted further investigation, so it was experimented with on some Ontario bridge decks.

CATHODIC PROTECTION

The corrosion of steel is an electrochemical reaction (14). When steel is in the state of active corrosion there are many small electrochemical cells on its surface. At the anodic areas an oxidation reaction takes place and the iron goes into solution as ions. At the cathodic areas a reduction reaction takes place and electrons are consumed. Of the several possible cathodic reactions, the one that occurs depends on the conditions existing at the cathode in question. The anodic reaction and one common cathodic reaction are symbolized below.

1. Anodic reaction: $Fe \rightarrow Fe^{++} + 2e$
2. Cathodic reaction: $\frac{1}{2}O_2 + H_2O + 2e \rightarrow 2(OH)^-$

The corrosion reaction can be stopped (or retarded) by preventing the access of oxygen or moisture to the cathodic areas. Another method of stopping the reaction is to lower the potential of the entire steel bar.

Galvanic corrosion is halted when all points on the bar have been polarized to a potential equal to or more than the open circuit potential of the most anodic point on the structure ($\underline{15}$). This method is known as cathodic protection and has been used for years to inhibit the corrosion of buried pipelines, concrete water tanks, and ships' hulls. In cases such as these, the application is straightforward. In the case of bridge decks, however, there is no surrounding conducting medium. The deck is suspended in air. Therefore, a conducting medium has to be supplied. Stratfull ($\underline{1}$) solved this problem by using a conductive layer consisting of a coke breeze and asphalt mixture on the deck. The coke mix was laid and energized by anodes at several points so that an even distribution of power was accomplished over the deck. The flow of current then was from the rectifier through the coke then down through the concrete bridge deck to the reinforcing steel and back to the rectifier. The coke breeze mix was covered with a wearing course of asphalt concrete to complete the system.

In this type of installation it is important that the coke mixture be insulated from any bare reinforcing steel, deck scuppers, expansion joints, and the like to prevent a direct short circuit to the bridge steel. The circuit diagram for such an installation is shown in Figure 1.

EXPERIMENTAL DESIGN

Initially two bridge decks were tested: a posttensioned, voided deck and a slab deck on AASHTO beams. Each of the decks showed signs of active corrosion. One part of each deck was protected, and the other part was left unprotected for comparison.

Sufficient electrodes were placed on each deck so that different electrode configurations could be studied. Both a regular high silicon-cast iron electrode ($\underline{16}$) and a regular graphite electrode were used. These were compared for efficiency at distributing the current over the bridge and for cost.

The circuit was designed so that varying amounts of current could be fed to each individual electrode and an even distribution of power could be obtained across the bridge deck.

The rectifier used was a constant current type to avoid the use of a standard cell buried in the deck. It was feared that the severe winters in Canada could damage a standard cell.

To determine whether corrosion had been halted, resistance probes were buried at several locations in each deck.

After cathodic protection had been installed on two bridges and operated for some time, improvements in construction and protective equipment were indicated. A third bridge, which was also showing active corrosion, was then selected and the improved methods of protection were applied to it.

BRIDGE DATA

The first bridge chosen was a posttensioned voided structure. It was one of the ramp bridges at a major interchange in the Toronto area. This 8-year-old bridge is 113 m (370 ft) long and 7.8 m (25.5 ft) wide and is a curved superelevated structure. The bridge was showing active corrosion in only one area near its western end. It was, however, the only bridge of its type available in which there was some active corrosion and for which it was possible to obtain a reasonable traffic control without causing a major traffic disruption. This bridge will be referred to as bridge 9.

The second bridge chosen has a concrete slab deck, 20 cm (8 in) thick on AASHTO beams. It was 7 years

old, 28 m (92 ft) long, and 11 m (36 ft) wide with 0.9-m (3-ft) sidewalks. This bridge was showing very active corrosion in several areas and has many spalled areas on its surface. It is the Duffins Creek bridge.

The surface of each bridge deck was surveyed to determine the extent of corrosion. This was determined by means of a Cu/CuSO half-cell (CSE) by using the technique described by Stratfull ($\underline{17}$, $\underline{18}$). The data were plotted to show lines of equal voltage on a diagram of the bridge deck. The data for Duffins Creek bridge (Figure 2) show that there were many areas of active corrosion where the voltage detected was greater than -0.35 V with reference to the CSE ($\underline{1}$).

All reinforcing steel, cable ducts, and guardrails were found to be continuous electrically on both bridges. There was, therefore, very little danger of any part of the steel being electrically isolated and thus in danger of stray current corrosion.

Each bridge deck was cored in several locations, and the cores were analyzed to determine the chloride content. The results of three cores selected from each bridge are shown in Table 1.

The other cores showed similar salt contents. These data show that there was sufficient chloride present at the level of the reinforcing steel to cause corrosion. It has been reported that the threshold level to cause the onset of corrosion is 0.59 kg of sodium chloride/m^3 of concrete (1 lb/yd^3) ($\underline{1}$). The upper level of the steel in both of these bridges was within 2.5 to 3.5 cm (1 to $1\frac{1}{2}$ in) of the surface.

CONDUCTIVE LAYER TESTS

A coke breeze-asphalt cement mix similar to that used by Stratfull ($\underline{1}$) was used for the conductive layer. Samples of coke breeze were obtained from a local steel company. This material had the following gradation:

Sieve Size	Percentage Passing	Sieve Size	Percentage Passing
No. 4	100	No. 50	18
No. 8	68	No. 100	12
No. 16	47	No. 200	7
No. 30	32		

The coke breeze was blended with 85 to 100 penetration grade asphalt cement to make a series of blends with increasing asphalt cement concrete. The mixes were made into beams by using a steel mold and were compacted with a kneading compactor followed by a leveling load applied by a compression testing machine. The density and resistivity of each beam were determined (Table 2).

From the data the blend containing 20 percent by weight of asphalt cement was chosen for this project. The resistivity of all blends was quite low. It was thought that the extra asphalt cement would give the blend chosen greater resistance to water action (stripping) since some water was almost certain to collect in this porous coke layer.

The work done by Stratfull ($\underline{1}$) showed that the coke breeze-asphalt mixture appeared to have sufficient strength to stand up under traffic; hence, no further testing was done in this direction.

ELECTRICAL CIRCUIT

The electrical circuit used is shown in Figure 3. This circuitry was installed in a panel box mounted beside the current rectifier on an abutment under the bridge. A switching arrangement made it possible to switch the panel ammeter into the circuit so that the current flow
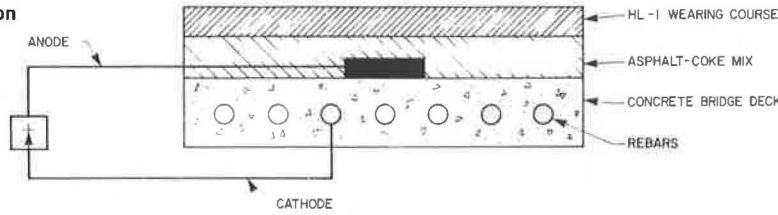
Figure 1. Cathodic protection circuit.

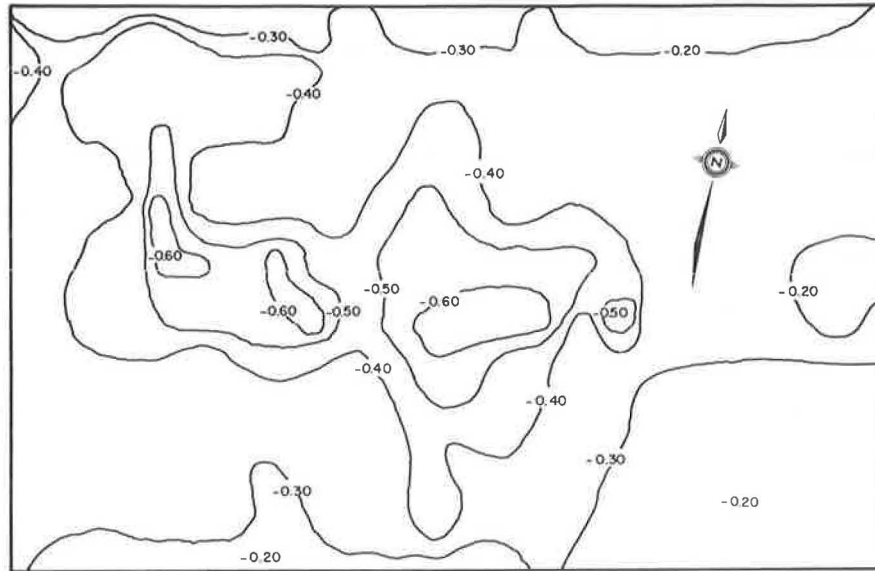Figure 2. CSE potentials on Duffins Creek bridge.

Table 1. Chloride content of bridge deck cores.

Bridge	Core	Depth (mm)	NaCl (kg/m³)	Bridge	Core	Depth (mm)	NaCl (kg/m³)
9	1	6.35	9.3	Duffins	1	12.7	11.4
		12.7	3.3	Creek		25.4	6.5
		25.4	2.2			38.1	3.4
		50.8	2.2			50.8	2.2
	2	12.7	12.0		2	6.35	17.8
		25.4	6.5			12.7	9.1
		38.1	3.1			25.4	1.1
		50.8	2.6			50.8	0.5
		76.2	2.4		3	6.35	9.9
	3	6.35	9.9			12.7	5.4
		12.7	5.5			25.4	0.8
		25.4	2.9			50.8	0.4
		50.8	2.8				

Note: 1 mm = 0.039 in; 1 kg/m³ = 1.7 lb/yd³.

Figure 3. Cathodic protection circuit.

A₁ = ANODES
R₁ = POWER RHEOSTATS
⊕ = FEED FROM CURRENT CONTROLED RECTIFIER
⊖ = REINFORCING STEEL GROUND BUS
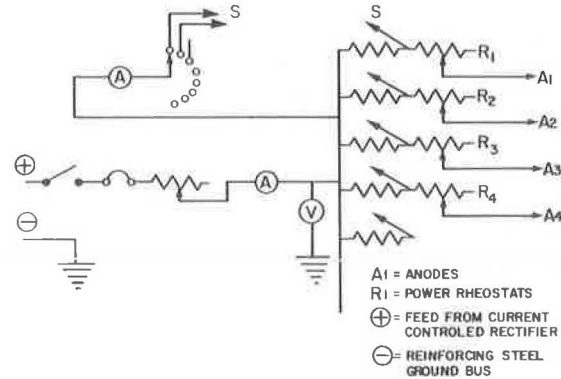
Table 2. Conductive mix properties.

Asphalt Grade	Percentage of Asphalt by Weight	Mix Density (g/cm³)	Resistivity (Ω·m)
85/100	10	0.99	0.0125
85/100	13	1.02	0.0125
85/100	15	1.06	0.0124
150/200	15	1.08	0.0133
85/100	17	1.14	0.0128
85/100	20	1.17	0.0143

Note: 1 g/cm³ = 62.4 lb/ft³.

to each individual anode could be measured. The power rheostats in each anode circuit made it possible to vary the anode resistance if necessary so that equal current flowed to each anode.

The majority of anodes used were made from high silicon-iron alloy (Duriron). This alloy is very resistant to corrosion, and its weight loss in such service is stated to be about 0.18 kg (0.4 lb) per ampere-year (16). Some experimental graphite anodes were also used on each deck. All of these anodes were approximately 30.5 cm (12 in) in diameter and 3.8 cm (1.5 in) thick. To keep the circuit resistance low despite the long runs of wire, No. 6 gauge wire was used. Wire with Canadian Standards Association specification TWU insulation was used. This is a heavy insulation for underground use and was used as the best available to resist the high temperatures (149° C or 300° F) of the coke-asphalt mixture.

The resistance probes used to determine whether and when corrosion was stopped were designed for underground service.

INSTALLATION ON BRIDGE DECK

Before the circuitry was installed on the deck, all delaminations were repaired by an epoxy cement injection technique developed by Crumpton (19). All spalls were repaired with concrete, and any exposed iron on the deck was covered with epoxy cement to insulate it from the coke mix.

The anodes were placed on each deck in three rows, each anode within a row separated from its adjoining anode by 3.6 m (12 ft). The plan of the two first decks is shown in Figures 4 and 5. Each anode was cemented to the deck with epoxy cement to prevent it from moving when the coke mixture was compacted.

Ground connections to the reinforcing steel were made at five randomly chosen locations on each deck. A resistance probe was placed in the hole and grounded to the bar. The hole was then refilled with concrete containing sodium chloride to initiate corrosion on the probe. The grounding cable was run to the curb to join the common grounding bus. Then all cables were run along the curb to a hole in the deck and down through the hole to the control boxes under the bridge.

After all the electrical equipment was installed on the surface, the coke layer was placed. This was spread by hand and compacted to a thickness of 5 cm (2 in). The coke was hand spread to prevent any injury to the cables and anodes. The following day a 3.8-cm (1.5-in) layer of wearing course was spread by a regular paver, and the compaction was done in the normal manner.

On bridge 9 only the western 33.5 m (110 ft) of the bridge was protected. On the Duffins Creek bridge the eastbound lane of the bridge was protected cathodically while the westbound lane was left unprotected.

EXPERIMENTAL DATA FROM BRIDGES

After all the electrical equipment had been installed on the bridge decks and they had been paved, they were left in that condition without any power being applied for 4 weeks. The purpose was to permit the probes to begin to corrode to such an extent that the trend was clearly indicated. Then the effectiveness of cathodic protection could be determined by the behavior of the probes.

Anode Resistance

The resistance of each anode circuit in both bridge decks was measured. Inasmuch as a potential difference of about 0.2 V normally exists between the concrete and the reinforcing steel, a normal ohmmeter could not be readily used. A meter for measuring soil or ground bed resistance was used. This was a battery-operated instrument that applied a high ac voltage, used a dc blocking capacitor, and could be read accurately to 0.05 Ω. Resistance was measured at the control panel between the wire leading to each anode and the common ground connection.

These values ranged from 0.9 to 2.8 Ω at bridge 9 and from 1.2 to 3.7 Ω at the Duffins Creek bridge. The resistance of the graphite anodes was generally lower than that of the silicon-iron anodes.

Voltage Measurement in Coke Bed

When the power was first applied to bridge 9, anodes 1, 2, 6, 7, 11, 12, 16, 17, 21, and 22 were connected, and at the Duffins Creek bridge anodes 101, 105, 106, 110, 111, 115, 116, and 120 were connected. Figures 4 and 5 show the location of these anodes on the bridge decks.

Initially the rectifiers (current controlled) were set to deliver 1 A to each bridge deck. Voltage readings were taken on the anode connections at the control panel and these varied from 1.6 to 1.8 V.

As soon as the power was applied to these bridges, the readings of one of the two active probes at Duffins Creek stopped rising. The other probes had still not become active. This cessation of corrosion is shown in Figure 6 for the probes at the Duffins Creek bridge. The curve in this figure for probe D showed an immediate halt in corrosion on the twenty-seventh day when the power was applied. Probe E, however, continued to rise. Some further testing showed that the connection between probe E and the reinforcing steel had been broken. On the fifty-sixth day this probe was reconnected to the steel. Immediately the probe values stopped rising. Since these probes were connected directly to the bridge steel it was a reasonable assumption that the bridge steel had also stopped corroding due to the application of the protective cathodic polarizing voltage.

Stratfull (1) measured the voltage drops in the coke by means of a CSE placed on a wet sponge on the asphalt surface by using a high-impedance solid-state voltmeter and grounded to the rebars. The charts for the Sly Park deck (1) showed considerable voltage variation across the deck. It was felt here that this variation could possibly be due to varying resistance in the asphalt concrete surfacing through which the readings had to be made. The coke mix itself has low resistivity and should not lead to such relatively large voltage drops. When this technique was tried on the decks of bridge 9 and the Duffins Creek bridge, even wider potential variations were measured. In some areas of the deck virtually no readings could be obtained. Inasmuch as 10 electrodes were in use on bridge 9, and 8 on the Duffins Creek bridge, a very even voltage distribution should have been present. The surfacing mixes used in Ontario are denser and have fewer voids than those used in California, so it appeared to be a problem of conductivity.

A series of holes was then drilled through the asphalt concrete surfacing to reach the coke layer, and No. 6 gauge insulated wires were driven into the holes to contact the coke mix. When the CSE was placed on these wires, the expected readings of 1.6 to 1.8 V were obtained. When the voltmeter probe was placed directly on the wire the same readings were obtained. It was obvious that the CSE was not needed in this instance to read the voltages. It was acting only as a liquid voltmeter probe and not as a half-cell. (The CSE or other half-cell is necessary, however, when a deck is surveyed to detect the presence of corrosion.) Because fewer than half of the anodes on each of these decks were being used to distribute the power, the remaining anodes were available for use as voltage probes by connecting the voltmeter to the anode wires at the control panel. In this way the entire bridge surface could be surveyed for potential drops by working from the control panel.

It was found that the voltage distribution across both decks was very even, within ±0.1 V. This suggested that a considerable reduction could be made in the number of electrodes used.

Polarization of Bridge Decks

When power was first applied to the bridge decks the voltage of the supply was relatively low because the method being used was a current control system and there was little or no back EMF in the bridge deck. After a short time the steel began to polarize and the back EMF began to build up in the decks. Thus, the effective resistance of the decks increased and the applied voltage rose and maintained the set current strength. This polarized (or
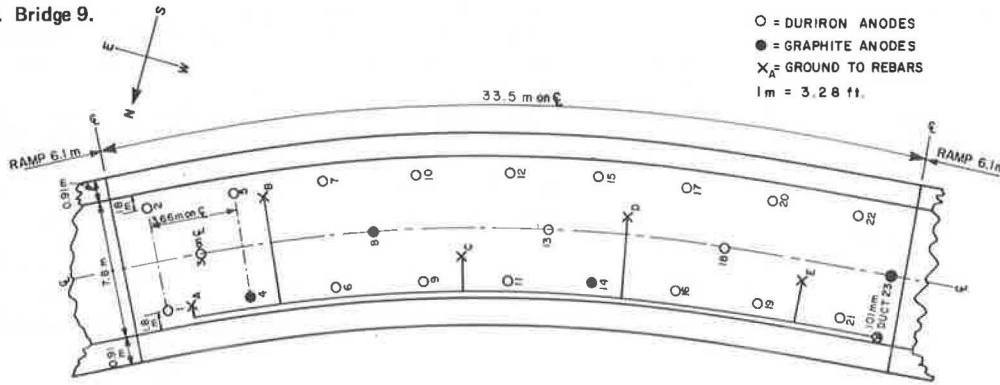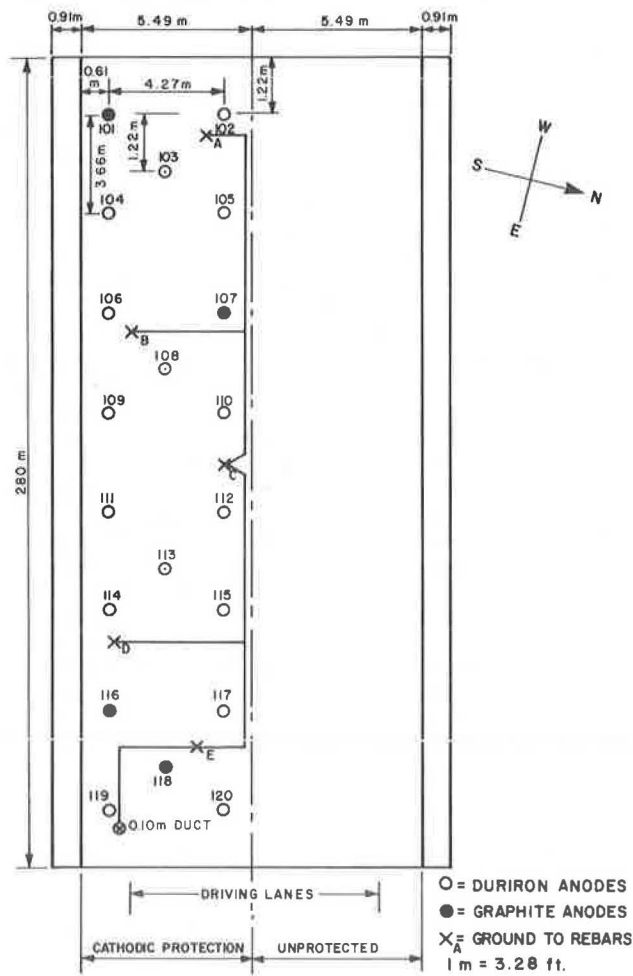
Figure 4. Bridge 9.

O = DURIRON ANODES
● = GRAPHITE ANODES
X_A = GROUND TO REBARS
1 m = 3.28 ft.

Figure 5. Duffins Creek bridge.

O = DURIRON ANODES
● = GRAPHITE ANODES
X_A = GROUND TO REBARS
1 m = 3.28 ft.

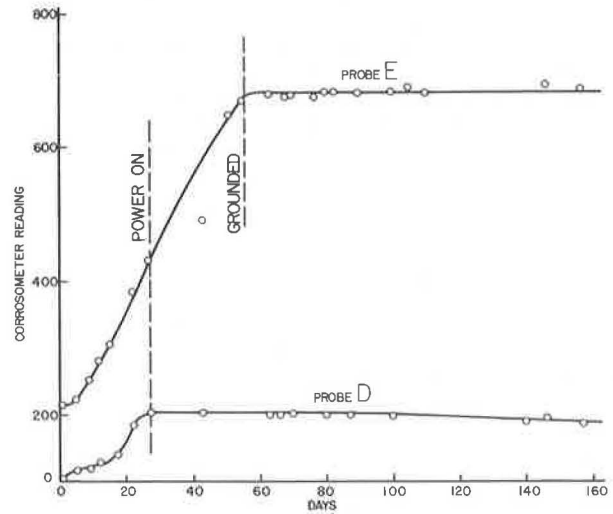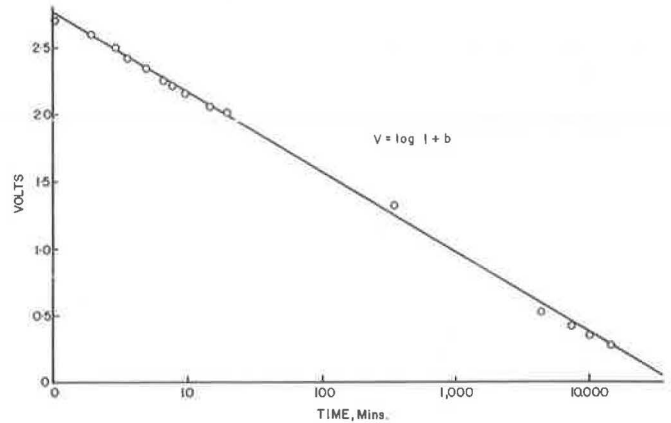Figure 6. Probe values on Duffins Creek bridge.

Figure 7. Residual voltage decline with time.

$$V = \log I + b$$

residual) voltage was measured between the anodes and the ground after the rectifier was switched off. After the bridges had attained electrical equilibrium the voltages were measured:

Item	Bridge 9	Duffins Creek
Current, A	1.0	1.0
Average applied voltage	1.85	1.6
Average residual voltage	1.40	1.0

It is common practice to polarize structures such as pipelines or water tanks in the range of -0.85 to -1.1 V with respect to the CSE in order to obtain protection (12, 20, 21, 22). The upper limit should be set at -1.1 V to prevent weakening of the bond between the reinforcing steel and the concrete (12, 22). A somewhat lower minimum potential for protection had been suggested by both Scott and Hausman. This value was -0.71 V (12, 22).

Electrode Configuration Trials

To determine the best configuration of electrodes for the

distribution of power throughout the bridge decks several different electrode combinations were tested. These tests were all run on bridge 9. After each test, the power was turned off so that the residual voltage on the deck could drop to a potential of -0.5 V or less. Before this study on the bridges a series of experiments was run on reinforced concrete slabs in the laboratory. The rate of decline of voltage with time for one of these slabs is shown in Figure 7. It was found that this curve matched very closely the rate of decline for both bridges. These bridges, however, were never polarized to such a high residual voltage as shown in the curve for the experimental slab.

Several different anode configurations were studied. When electrodes 3, 8, 13, 18, and 23 down the center of the bridge were used, a very even distribution of voltage again was obtained.

When electrodes 3, 13, and 23 were used at a current strength of 0.8 A, a good power distribution was obtained over the bridge surface once equilibrium had been established. This effect is shown in Figure 8. Curve A shows the drops in the applied voltage down the length of the deck when the power was first applied. Before this application of power the bridge had been left without power until the residual voltage on the reinforcing steel had subsided to -0.2 V. After 3 days the bridge had attained electrical equilibrium and the voltage drop across the surface was as shown by curve B in Figure 8. It can be seen that these three electrodes separated by 15.2 m (50 ft) from each other produced a very regular power distribution across the deck surface.

The next trial was run by using just two anodes, 5 and 19, again at a current strength of 0.8 A. The data obtained are shown in Figure 9. From the coincidence of curves D and E it can be concluded that equilibrium was obtained in approximately 48 h. The voltage drops here, however, were greater than when three anodes were used. Here a maximum difference between the applied voltage and the lowest point on the deck was 0.42 V.

Two tests were made by using one anode. The first was made by using anode 13, which was situated near the center of the bridge deck. In this test also the current strength was 0.8 A. The voltage drops along the deck are shown in Figure 10 for the applied voltage. Even after 48 h, the voltage drops between the applied voltage and the lowest points on the deck were 0.8 V, which were much too wide for satisfactory operation. The second test applied power to anode 23 at the extreme end of the deck. For this test the current strength was reduced. To force 0.8 A through the deck from one anode, as shown in the previous test, much too high a voltage (2.5 V) was required. For this test the rectifier was set to deliver 0.2 A. The voltage drops in the deck are shown in Figure 11. Curve A shows the applied voltage at the start of the test and curve A_1 the resulting residual or polarized voltage. Curves B and B_1 showed the situation after 24 h. The deck voltages were again measured after 5 days, and the curve obtained was slightly higher but virtually identical with curve B. The bridge under these conditions was used for comparing voltage measurement techniques and this is described in the next section.

The above tests suggested that the best anode configuration was the one in which three anodes were used and were spaced down the center of the deck. Anodes 3, 13, and 23 were again connected, and the current was set to deliver 0.4 A. When the bridge had attained equilibrium after 48 h, the voltage drops in the bridge deck were as shown in Figure 12. The voltage applied was 1.05 V, and the voltage at the lowest point in the deck was 0.85 V. This resulted in a residual or polarized voltage varying from -0.88 to -0.82 V, which was satisfactory for the

protection of the reinforcing steel.

At the Duffins Creek bridge when power was first applied, eight electrodes were used. This gave a very uniform power distribution. The anodes were then reduced to four situated in the center of the protected lane. A current strength of 0.5 A was used, and the resulting voltages in the coke varied between 0.96 and 1.14 and the polarized voltage varied between -0.93 and -0.99. Thus the steel in this deck was satisfactorily protected.

Thus to protect bridge 9 only 0.5 W of power was required (1.13 V, 0.4 A on three anodes) and to protect the Duffins Creek bridge only 0.6 W of power (1.2 V, 0.5 A on four electrodes) was required.

Comparison of Voltage Measurements

Some doubt was expressed regarding the accuracy of the measurements of the applied and polarized voltages using probes in the coke bed to determine the actual voltage in the concrete slab. It was felt that the highly conductive coke mix could possibly even out local potential differences that might exist. Because the anodes being used as voltage probes were also quite large and were insulated from the deck directly below them, these would also tend to average out small differences in potential. To determine whether such was the situation a series of tests was conducted on both bridge 9 and the Duffins Creek bridge.

Bridge 9 for this test was powered only by anode 23 at one end of the deck. The current strength was 0.2 A. The voltage drop and the polarized voltages along the deck are shown in Figure 11.

Holes 5.4 cm (2½ in) in diameter were drilled through the asphaltic concrete surfacing and the coke mix to expose the concrete. The holes were drilled 1.2 m (4 ft) from the curb and close to anodes 4, 6, 9, 11, 14, 16, 19, and 21.

At each hole the CSE was placed in contact with the exposed concrete surface, and on and off voltage readings were obtained. A regular steel voltmeter probe was then thrust into the coke at the side of the hole, and on and off voltage readings were obtained on the probe and on each anode connection. These results are given in Table 3.

The data show that there is little difference between the readings taken in the hole with either the CSE or the voltmeter probe. The slight difference of +0.05 V higher for those taken with the probe could be due to higher resistance in the CSE or higher contact resistance. There is, however, a consistent difference of +0.16 V on the average higher for off readings taken on the anodes and for those taken by the CSE when used as a probe. This might be explained by the lower surface resistance between the anodes and the coke; there was a much greater contact surface in this case than there was when the CSE or voltmeter probe was used.

A somewhat similar test was run at the Duffins Creek bridge. Here eight holes were bored through the surfacing and the coke mixes in the protected side of the bridge to expose the concrete. The anodes were not so close to these holes as they were on bridge 9, so a close comparison between anode and CSE probe voltages could not be obtained. The anode readings did seem to be 0.1 to 0.2 V higher than the CSE probe readings similar to those obtained on bridge 9. When the voltage readings obtained with the CSE on the concrete surface in the hole were compared with the voltmeter probe readings in the coke at the sides of the hole, the same relationship held as at bridge 9: The probe in the coke readings was a little higher than the CSE on the concrete readings. In addition to the above tests, voltage readings were taken in each hole by simply placing the voltmeter steel probe

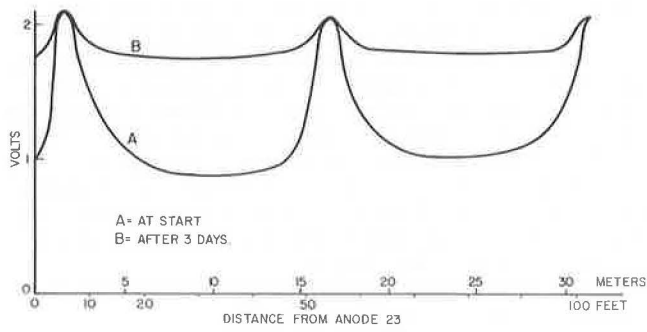**Figure 8. Voltage drop along deck of bridge 9 (0.8 A to anodes 3, 13, and 23).**

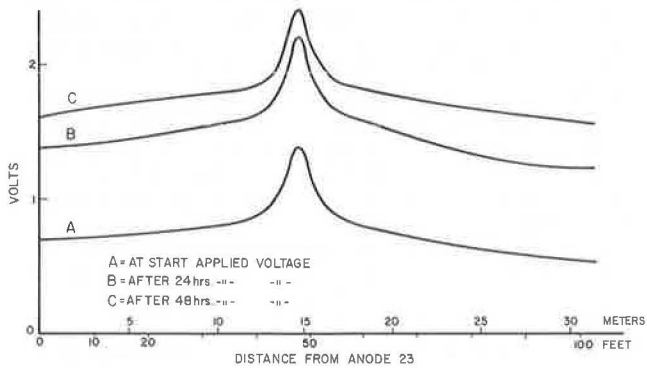**Figure 9. Voltage drop along deck of bridge 9 (0.8 A to anodes 4 and 19).**

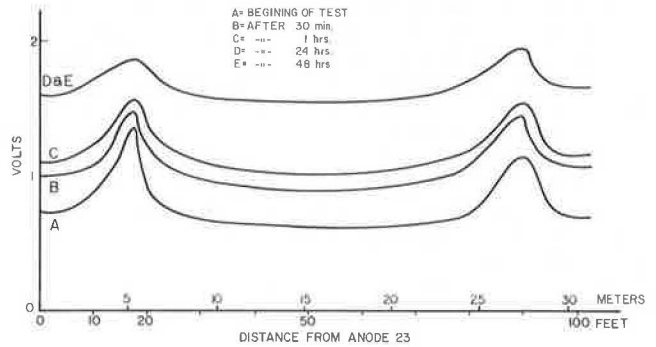**Figure 10. Voltage drop along deck of bridge 9 (0.8 A to anode 13).**

Figure 11. Voltage drop along deck of bridge 9 (0.2 A to anode 23).
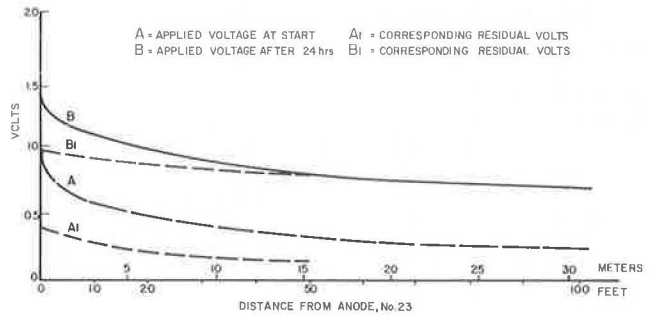

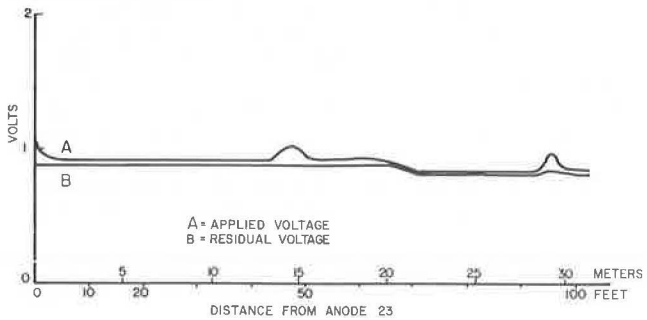**Figure 12. Voltage drop along deck of bridge 9 at equilibrium (0.4 A to anodes 3, 13, and 23).**

Table 3. Comparison of voltage measurement methods on bridge 9.

Anode	Anode On	Voltage Off	CSE on Concrete Surface		Voltmeter On	Probe in Coke Off
			On	Off		
1	0.68	0.67	0.56	0.56	0.58	0.57
4	0.70	0.70	0.52	0.52	0.55	0.55
6	0.72	0.71	0.57	0.56	0.60	0.59
9	0.76	0.75	0.60	0.59	0.62	0.62
11	0.78	0.77	0.58	0.58	0.62	0.62
14	0.80	0.79	0.64	0.63	0.67	0.66
16	0.83	0.82	0.67	0.65	0.70	0.68
19	0.87	0.85	0.72	0.69	0.83	0.80
21	0.93	0.86	0.82	0.73	0.86	0.76

Figure 13. Anode and probe placement on Medway Creek bridge.
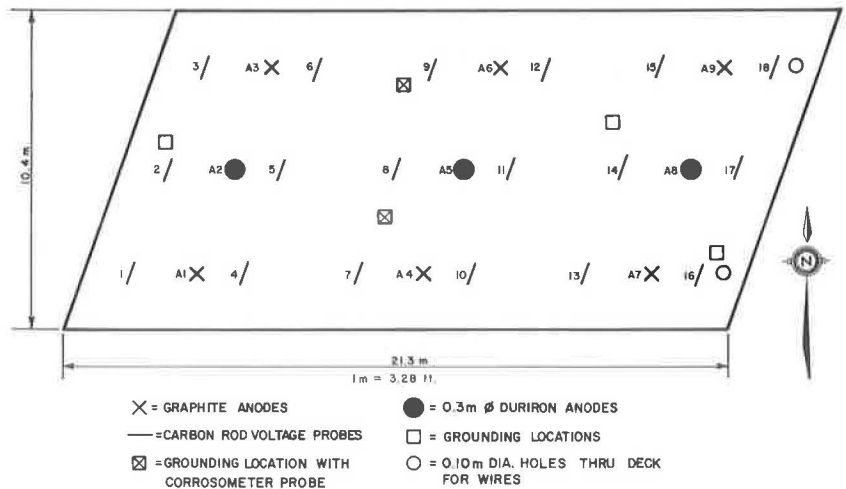
directly on the concrete surface. A series of good readings was obtained; these were a little higher than the CSE readings but a little lower than the probe in the coke readings.

These tests showed that it was perfectly feasible and accurate to monitor the polarized voltages on the reinforcing steel by means of probes buried in the coke layer.

MEDWAY CREEK BRIDGE

The data obtained from bridge 9 and the Duffins Creek bridge showed that the circuit could be simplified, the number of anodes could be reduced, and voltage probes properly spaced in the deck could be used to monitor the bridge. To try these ideas out it was decided to apply protection to another medium-sized bridge and to protect the entire bridge deck and not just a portion of it.

The bridge chosen was the 8-year-old Medway Creek bridge, and its deck was in a state of active corrosion. The surface of this deck was spalling, and some delaminations were present within the deck. A CSE half-cell survey was made of the deck. This survey showed that there were several areas in a state of active corrosion with CSE potentials greater than -0.35 V.

Graphite anodes were used for this bridge. Graphite rods 3.2 cm (1.3 in) in diameter were obtained, and the anodes were fabricated locally from this material. The rod was cut into pieces 41 cm (16 in) long, and four lengths were connected together so that the anode could be laid out on the deck in the form of a four-pointed star with a central connection. Some silicon-iron anodes were also used in case problems arose with the use of the graphite anodes. The data obtained from bridge 9 indicated that two anodes should be sufficient to provide protection for this bridge. Because this was still an experimental installation, extra anodes were used on the deck so that different anode configurations could be used if this became necessary.

Voltage probes were placed on the deck and buried in the coke mix so that it would not be necessary to use the anodes as voltage probes. These probes consisted of small carbon rods, 2.5 cm (1 in) in diameter and 15.2 cm (6 in) long, to which a No. 10 wire would be attached with a mechanical seal.

The layout of the deck is shown in Figure 13. The six graphite anodes were evenly spaced, three on each side of the deck. The three silicon-iron anodes were placed down the centerline. The voltage probes were laid out on 3.6-m (12-ft) centers, and five ground connections were made to the reinforcing steel.

The control panel was made smaller and simpler. Each of the nine anodes had a power rheostat in the circuit in case adjustments had to be made. The connection to each voltage probe was through a banana plug jack on the panel. There were three meters, one for the applied volts, one for the total current, and one for the current to each individual anode, which was selected through a switching system.

Construction Method

The experience gained at the two test bridges suggested that the electrical equipment and both the coke and surfacing mixes could be laid and compacted in 1 day for each half of the deck with a minimum of inconvenience to traffic. This involved closing half of the bridge each day for construction and using the remaining half for two-way traffic, which was controlled by flagmen.

After the electrical equipment and wire had been placed on the deck it was covered by 6 cm (2 in) of coke mix similar to that used previously. This was then protected by a 3.8-cm (1½-in) wearing course of asphaltic concrete.

Experimental Data

After the electrical installation was completed the anode resistances were measured. These values, measured between the anode connection at the control panel and the connection to the reinforcing steel, were as follows:

Anode	Resistance (Ω)	Anode	Resistance (Ω)
1G	1.5	6G	2.1
2	2.3	7G	1.8
3G	1.9	8	2.6
4G	1.8	9G	2.6
5	2.6		

Again, the silicon-iron anodes had generally higher resistance than the graphite anodes.

The bridge was allowed to remain without power being applied for 4 weeks. During this period the probes began to corrode because salt had been added to the concrete covering them. After the trend was clearly established power was applied to the circuit. Immediately the reading of each probe ceased rising, showing that corrosion had been effectively halted. This is shown in Figure 14.

Current was applied to the bridge at a strength of 0.9 A, and anodes 3 and 7 were used to distribute the current. After the bridge had achieved electrical equilibrium the voltage drops within the deck were as shown in Figure 15. The residual or polarized voltage in the deck with the current off was measured at an average value of 1.03 V.

The electrical characteristics of the installation of this bridge were studied during the winter and spring of 1974 and 1975. Unlike the other two bridges, this bridge was more sensitive to the weather, particularly to the amount of precipitation and to the use of deicing chemicals. After heavy rains and during the winter, the resistance of the deck decreased with a resulting drop in potential since this deck was also under current control. This required that the rectifier be reset to deliver a larger amount of current to maintain the required residual potential on the steel.

After one heavy rainstorm the polarized potential dropped to a range of 0.6 to 0.7 V, i.e., the probes remained steady and did not show the onset of corrosion. It seemed that rainwater had seeped into the porous coke layer and from it was entering into the concrete, thus lowering its resistance and thereby requiring a larger amount of current to keep the voltage in the coke at a sufficiently high level (about 1.2 V) to induce a polarized potential on the steel of 0.9 to 1.0 V. To compensate for this the current strength was raised to 1.2 A.

When the current strength at the bridge was 1.0 A, the bridge required 1.4 W for protection; the rise in current to 1.2 A raised the power required to 1.7 W for complete protection.

Resistance measurements had been made at the start on each voltage probe. The resistance was measured between each probe and the reinforcing steel, and it was also measured between probe 1 and each of the other probes to obtain the interprobe resistance. These values were checked again after the deck potential began dropping because of what was assumed to be increasing moisture and salt in the concrete. Some of these data are given below.

Probe	November 8, 1974	June 24, 1975
1-4	3.0	2.5
1-7	3.6	2.3
1-10	4.0	2.2
1-13	4.2	16.0
1-16	4.2	2.1

The high resistance between probes 1 and 13 on June 24 was caused by development of a poor contact between probe and wire.

An examination of the data brought out two points. The probes are all 3.6 m (12 ft) apart; the resistance does not decrease linearly with distance. There is little increase in resistance after probe 10, 11 m (36 ft) from probe 1. This suggests that from this point on most of the current had passed down to the concrete and was flowing along the reinforcing steel then back up to the probe connected to probe 1. The data for June follow the same pattern only to a greater extent. Here the resistance was the same all down the deck. This suggested that the concrete had absorbed a lot of brine during the winter and the rain which had recently fallen decreased the resistivity of the concrete to a level where a much larger current was required to achieve the polarizing voltage sufficient to protect the steel.

DISCUSSION OF RESULTS

During the anode configuration experiments the polarized voltage on the bridge steel was allowed to subside on several occasions in a series of steps. During these tests it was found that the probes did not show the onset of corrosion until the polarized voltage had dropped below -0.55 (CSE) V. The same effect was seen at the Duffins Creek and Medway Creek bridges when power

failures occurred. There may have been a lag between the voltage falling to a certain point and the onset of corrosion. The results do point out, however, that several days' protection is built into the deck in case of a power failure if the polarized voltage on the steel has been kept at a level of -0.85 V or higher.

The experiments showed that an anode separation of 15.2 m (50 ft) was suitable on all bridges to keep the voltage drop in the coke layer at a reasonable level. At Medway the anode separation was less than 15.2 m (50 ft).

The amount of power required to provide adequate protection to the three bridge decks is given in Table 4. The table shows the amount of power in watts required for the entire bridge deck. Also shown is the current flow per square meter of deck. These figures show that the amount of power required is negligible and could quite conceivably be supplied in remote locations by solar cells in conjunction with storage batteries.

The principal of using voltage probes in the coke layer to determine the voltage and the polarized voltage has been proved to work at the Medway bridge. The experiments on all these bridges have shown that either graphite or silicon-iron anodes are suitable for supplying power and protection. Because graphite anodes are not sacrificial, the anode reaction must be some chemical reaction other than the oxidation of the metal to form ions. It could be the oxidation of chloride ions to either the gaseous state or to some higher valence state.

The use of a constant current type of rectifier has advantages and disadvantages. This type of rectifier did not require a standard cell in the bridge deck, which could have been damaged by the low temperatures in this country. At bridge 9 and at Duffins Creek this type of rectifier worked very well. There were voltage swings at these bridges caused by weather conditions. Weather conditions caused changes in the resistivity of the deck, and this change in resistance caused the voltage to fluctuate to maintain a constant current. The applied voltage remained, however, within the limits required to provide adequate protection, the protection being judged by the polarized voltage. There were wider voltage

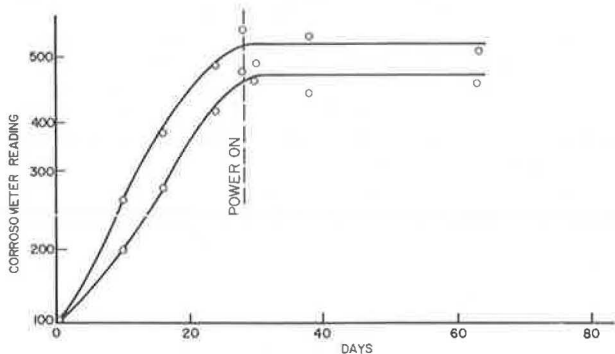Figure 14. Probe values on Medway Creek bridge.

Table 4. Power requirements for bridge decks.

Bridge	Current (A)	Deck Area (m²)	Power (W)	Current Density (mA/m²)
Bridge 9	0.4	277.12	0.5	1.44
Duffins Creek	0.5	153.84	0.6	3.2
Medway Creek	1.0	221.1	1.4	4.5
	1.2	221.1	1.7	5.4

Note: $1\ m^2 = 10.76\ ft^2$; $1\ mA/m^2 = 10.76\ mA/ft^2$.

Figure 15. Voltage distribution on Medway Creek bridge (1.8 V, 0.9 A on anodes 3 and 7).

swings at Medway Creek bridge. This deck appeared to be more open, and in dry weather the supply voltage would swing high and then drop to lower levels in wet weather. This type of bridge could have benefited from potential control.

When the coke mix was being laid, it was very absorbent and could contain considerable quantities of water. This was disadvantageous because it could hold water in contact with the bridge surface, which could result in increased freeze-thaw damage to the decks. It was feared that this water might cause stripping of the asphalt from the coke surface and then cause the mix to lose strength. Samples of the coke mix were removed from the Medway Creek deck after 6 months' service and were examined. The mix seemed to have retained all its strength, and no sign of stripping was detected. It could be an advantage to have a more impervious mix that would keep the water away from the bridge deck and yet have sufficient voids to permit any gas formed at the interface to escape. Such a mix is being developed in these laboratories.

The area to which the cathodic protection extends in a deck is being studied. It was mentioned in discussion with other researchers that perhaps the upper layer of steel intercepted most of the current and there was little remaining current to provide protection to the lower layers. Investigations in progress in these laboratories indicate that the protection goes much deeper, especially in the case of slab decks. Heuze (24) suggested that the deck may behave like a capacitor. These bridge decks have the property of storing large quantities of current, and to do this they must act either as a capacitor or as a battery. The experiments currently in progress tend to confirm the capacitor action.

CONCLUSIONS

1. Cathodic protection of bridge decks is feasible and has been demonstrated on three bridge decks.

2. A conductive coke-asphalt mix with anodes spaced 15.2 m (50 ft) apart in the mix is suitable for providing power to the bridge deck.

3. Voltage probes buried in the coke mix have proved acceptable for monitoring the applied voltage and the polarized voltage.

4. Both slab and posttensioned voided decks can be protected by this method.

5. The power required for protection is very small and varies from 0.014 to 0.04 mA/m^2 of deck surface.

6. The cost of epoxy injection concrete repairs and of applying the cathodic protection is small compared to the cost of replacing the deck.

REFERENCES

1. R. F. Stratfull. Experimental Cathodic Protection of a Bridge Deck. TRB, Transportation Research Record 500, 1974, pp. 1-5.
2. D. G. Moore, D. T. Klopt, and R. J. Hensen. Protection of Steel in Prestressed Concrete Bridges. NCHRP, Rept. 90, 1970.
3. B. Tremper, J. L. Beaton, and R. F. Stratfull. Causes and Repair of Deterioration to a California Bridge Due to Corrosion of Reinforcing Steel in a Marine Environment—Part II: Fundamental Factors Causing Corrosion. HRB, Bulletin 182, 1958, pp. 18-41.
4. R. Shalon and M. Raphael. Influence of Seawater on Corrosion of Reinforcement. Proc., American Concrete Institute, Vol. 55, No. 12, 1959, p. 1251.
5. G. W. Whitman, R. R. Russell, and V. J. Altieri. Effect of Hydrogen-Ion Concentration on the Sub-merged Corrosion of Steel. Industrial Engineering Chemistry, Vol. 16, 1924, p. 665.
6. V. K. Gouda. Corrosion and Corrosion Inhibition of Reinforcing Steel: Part I—Immersed in Alkaline Solution. British Corrosion Journal, Vol. 5, No. 198, 1970.
7. J. Ryell and B. S. Richardson. Cracks in Concrete Bridge Decks and Their Contribution to Corrosion of Reinforcing Steel and Prestressing Steel Cables. Ontario Ministry of Transportation and Communications, Downsview, Rept. IR-51, 1972.
8. Durability of Concrete Bridge Decks, Report 1. State Highway Commission of Kansas, Bureau of Public Roads, and Portland Cement Association, 1965.
9. Durability of Concrete Bridge Decks, Report 2. Michigan State Highway Department, Bureau of Public Roads, and Portland Cement Association, 1965.
10. H. H. Uhlig. The Corrosion Handbook. Wiley, New York, 1955.
11. B. Heuze. Cathodic Protection of Steel in Prestressed Concrete. Materials Protection, Vol. 4, Nov. 1965, p. 57.
12. D. A. Hausmann. Criteria for Cathodic Protection of Steel in Concrete Structures. Materials Protection, Vol. 8, No. 10, 1969.
13. R. F. Stratfull. Inhibiting the Corrosion of Steel in a Reinforced Concrete Bridge. Corrosion, Vol. 15, No. 6, 1959, p. 331.
14. D. A. Jones. The Measurement and Prevention of Corrosion by Electrochemical Methods. Industrial Engineering Chemistry, Vol. 11, No. 1, 1972, p. 12.
15. R. H. Brown and R. B. Mears. A Theory of Cathodic Protection. Trans., Electrochemical Society, Vol. 74, 1938.
16. The Use of High Silicon Cast Iron for Anodes. Corrosion, Vol. 13, No. 2, 1957, p. 33.
17. D. L. Spellman and R. F. Stratfull. Chlorides and Bridge Deck Deterioration. HRB, Highway Research Record 328, 1970, pp. 38-49.
18. R. F. Stratfull. Corrosion Autopsy of a Structurally Unsound Bridge Deck. HRB, Highway Research Record 433, 1973, pp. 1-11.
19. C. F. Crumpton and others. Epoxy Injection Repairs Delaminated Bridge Decks. Civil Engineering Journal, Proc., ASCE, Vol. 44, Nov. 1974, p. 55.
20. R. J. Kuln. Proc., API, Vol. 14, Sec. 4, 1933, p. 153.
21. Control of External Corrosion on Underground Piping Systems. National Association of Corrosion, Standard RP-01-69.
22. G. N. Scott. Corrosion Protection Properties of Portland Cement. Journal of American Water Works Association, Vol. 57, No. 8, 1965.
23. B. Heuze. Corrosion and Cathodic Protection of Steel in Prestressed Concrete Structures, Part 2, Construction, Vol. 21, April 1966.

Use of Characteristic Curves in the Design of Elastomeric Pavement Seals

Ravindra K. Vyas, University of Utah

This paper describes and illustrates a relatively simple technique for predicting performance of and designing certain elastomeric seal sections. The approach relies on characteristic curves. The emphasis is on practical applications. The basic advantage of the approach is that the highway engineer can deal with a problem of nonlinear structural analysis without performing the analysis. The procedures suggested do not require anything more than a slide rule or a pocket computer.

Elastomeric seals, because of their high flexibility, seem to be well suited for sealing expansion joints in highway pavements. This paper illustrates how the results of some earlier research (1, 2, 3) on elastomeric seals may be used by the highway engineer for practical predictions and design.

The suitability of an elastomeric seal is assessed from a set of standard laboratory tests. An important test in this set is the load-deflection experiment. The purpose of the experiment is to determine whether the product exerts the required forces at the specified minimum and maximum compressions. The characteristic curves and the approach discussed in this paper make it possible to arrive at these values without conducting an experiment. In addition, the characteristic curves may be used in the design of seals that will meet the specified requirements.

The discussion in this paper is restricted to seal sections whose geometry consists of identical, symmetric, V-shaped web members with vertical sidewalls (Figure 1). The section may also have a central vertical diaphragm passing through the apex of each V-shaped segment. The stress-strain curve of the material is assumed to be reasonably linear. It has been shown (1, 2, 3) that with these assumptions one can easily construct an analytical load-deflection curve for a given sample that conforms with the assumed geometry. The analytical technique has yielded results that compare satisfactorily with experimental results.

The theoretical problem is that of nonlinear structural analysis because of the large deformations involved (as much as 40 or 50 percent compression). The highway engineer, however, can bypass this process of nonlinear analysis and use instead the characteristic curves and the expressions given in this paper.

Let us consider a single web member (Figure 2). When the member is loaded by a pair of forces P, it will undergo a compression, say Δ. Next, if we assume that the angle α at the sidewall does not change, a pair of moments M_0 must be exerted by the sidewalls on the member. The relationship among P, Δ, and M_0 depends on the angle α, the modulus of elasticity E, the thickness t of the web member, and the undeformed width 2b from sidewall to sidewall. A study of the relationship of P and Δ will require an independent analysis for each given seal, which in a nonlinear problem would be rather time-consuming. However, we can simplify our task by reducing the specific problem to a characteristic problem by introducing dimensionless force quantities and deflections defined as follows:

$$u^2 = P\ell^2/EI \qquad (1)$$

$$v = M_0\ell/EI \qquad (2)$$

$$\delta = \Delta/2b \qquad (3)$$

Equations 1, 2, and 3 stand for dimensionless force, dimensionless end moment, and dimensionless compression. If we know the values of the dimensionless force u^2 and the dimensionless end moment v for any given compression δ, the corresponding values of P and M_0 can be easily computed from equations 1 and 2. For a given compression δ, the dimensionless force quantities u^2 and v depend only on the characteristic angle α. Then we can cover a wide spectrum of load-deflection responses by constructing curves of u^2 to δ and v to δ for properly selected values of α. These curves are referred to as characteristic curves. The characteristic curves for α varying from 5 to 60 deg in steps of 5 deg are given in Figures 3 and 4. The analytical details have already been given (1, 2, 3) and are therefore not repeated here.

In addition to the characteristic curves, we will need,
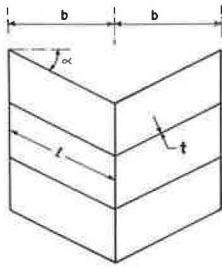
Figure 1. Characteristic section geometry.

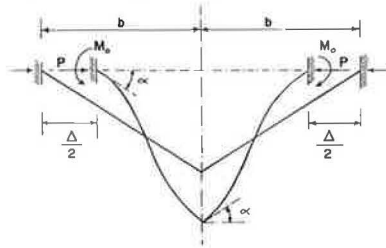**Figure 2. Single V-shaped web member.**

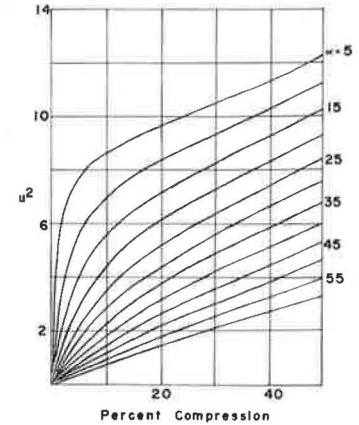**Figure 3. Characteristic curves for the dimensionless force u².**

Percent Compression

Figure 4. Characteristic curves for the dimensionless moment v.
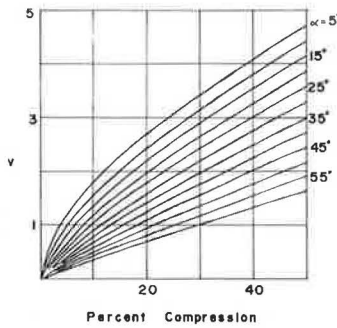

Percent Compression

Table 1. Values of u² and v for checking the product performance at 20 and 50 percent compression.

α (deg)	u² 20% Compression	50% Compression	v, 50% Compression
5	9.6	12.2	4.70
10	8.4	11.2	4.40
15	7.15	10.2	4.12
20	6.1	9.3	3.82
25	5.2	8.4	3.56
30	4.4	7.55	3.27
35	3.72	6.75	2.98
40	3.18	5.95	2.68
45	2.62	5.25	2.40
50	2.2	4.60	2.13
55	1.8	3.9	1.88
60	1.42	3.3	1.60

Figure 5. Structural weight versus the characteristic angle α.

α (Degrees)
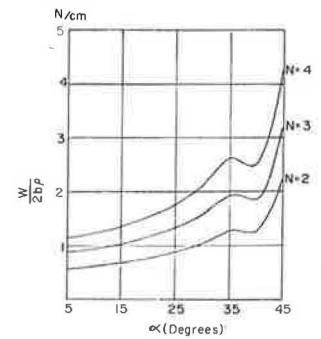
Table 2. Summary of data for seal design.

α (deg)	N = 2 t_1 (mm)	t_2 (mm)	t_3 (mm)	W/2bρ (N/cm)	N = 3 t_1 (mm)	t_2 (mm)	t_3 (mm)	W/2bρ (N/cm)	N = 4 t_1 (mm)	t_2 (mm)	t_3 (mm)	W/2bρ (N/cm)
5	2.46	4.14	4.02	0.569	2.16	3.60	4.02	0.883	1.96	3.28	4.02	1.12
10	2.59	4.29	4.32	0.608	2.26	3.76	4.32	0.912	2.06	3.40	4.32	1.22
15	2.77	4.49	4.10	0.667	2.44	3.91	4.70	1.01	2.21	3.56	4.70	1.34
20	3.00	4.72	5.00	0.765	2.60	4.12	5.20	1.15	2.36	3.73	5.20	1.53
25	3.22	5.00	5.79	0.873	2.32	4.37	5.79	1.32	2.57	3.96	5.79	1.76
30	3.50	5.33	6.57	1.05	3.08	4.67	6.57	1.57	2.79	4.29	6.57	2.10
35	3.86	5.74	7.62	1.28	3.38	5.03	7.62	1.92	3.07	4.57	7.62	2.61
40	4.29	6.27	6.90	1.24	3.71	5.49	6.90	1.87	3.38	4.98	6.90	2.49
45	4.77	6.91	10.90	2.23	4.19	6.02	10.90	3.20	3.78	5.49	10.90	4.26

for performance prediction and design, the expressions given below.

$$P = Cu^2 \qquad (4)$$

where C is a constant given by

$$C = [NE_T \beta^2 t^3 / 3\ell^2 (1 + \beta)^2] \qquad (5)$$

and

$$\beta = \sqrt{\frac{E_c}{E_T}} \qquad (6)$$

where

P = total force per unit length of the seal,
E_c = modulus of elasticity in compression,
E_T = modulus of elasticity in tension,
t = thickness of the web member,
ℓ = inclined length (Figure 1), and
N = number of identical web members.

The above expressions are of practical value in (a) predicting the performance of a sample that conforms with the assumed geometry and (b) designing a section that will meet a set of specified load requirements. Another important item that should be verified is the value of the maximum compressive stress σ at maximum compression:

$$\sigma = Av + Bu^2 \qquad (7)$$

where the constants A and B are given by

$$A = [t\beta^2 E_T/\ell(1 + \beta)] \qquad B = [t^2\beta^2 E_T \cos\alpha/3\ell^2(1 + \beta)^2] \qquad (8)$$

APPLICATIONS IN PERFORMANCE PREDICTION

Some useful applications of the material discussed above will be discussed here with the help of illustrative examples. Let us consider a seal section 1.75 cm wide consisting of four identical web members (N = 4), each 1.78 mm thick with characteristic angle α of 10 deg. Furthermore, for the properties of material of the seal we will assume $E_T = 559$ N/cm^2, $E_c = 363$ N/cm^2. [These values are equivalent to those used in illustrative examples in earlier publications (1, 2, 3).] Then from equation 6 we evaluate $\beta = 0.81$. For $\alpha = 10$ deg and width = 1.75 cm we find that $\ell = 0.89$ cm. Further, by using the above numerical values in equations 5 and 8, we find that A = 40.7 N/cm^2, B = 1.47 N/cm^2, and C = 1.08 N/cm.

With these data we can quickly generate the load-deflection curve for the seal. All we need to do is use the characteristic curve for $\alpha = 10$ deg in Figure 3 and the multiplier C = 1.08 already evaluated. We need to read off the values of u^2 for different values of percentage compression and scale these values of u^2 by the multiplier C. The resulting numbers will be the values of force in kilograms per centimeter of seal.

The characteristic curves can also be usefully applied to quickly verify whether a given seal will meet certain specifications. For example, a typical requirement is that the seal at least exert a force P_1 per unit length at 20 percent compression and a force P_2 per unit length at 50 percent compression. The Utah State Department of Highways recommends these values to be 3.51 N/cm and 21.1 N/cm for a 1.75-cm-wide seal. For this check we can use the characteristic curves of Figure 3 or the summarized values given in Table 1. The seal we have been considering here will exert a force $P_1 = 8.4C = 8.9$ N/cm at 20 percent compression and $P_2 = 11.2C = 12.1$ N/cm at 50 percent compression. It will therefore meet the specified requirement at 20 percent compression but not at 50 percent compression. The theoretical predictions have been found to be fairly close to experimental results. Hence, a check like the one suggested here can save a substantial amount of experimental work.

Another quantity of interest is the maximum compressive stress at 50 percent compression. This stress can be evaluated by the photoelastic experiments suggested by Cook (4) or by using equations 7 and 8 and the 50 percent compression values given in Table 1. For example, if we use the latter approach, for the seal under consideration,

$$\sigma = Av + Bu^2 = 40.7 \times 4.4 + 1.47 \times 11.2 = 195.5 \text{ N/cm}^2 \qquad (9)$$

APPLICATIONS IN DESIGN

So far we have considered the application of characteristic curves in predicting the product performance. These curves can also be used in designing a seal section that will meet specific requirements. To illustrate the process let us consider the following three requirements. The total force P should be P_1 at 20 percent compression and P_2 at 50 percent compression. Further, the maximum compressive stress at 50 percent compression should not exceed an allowable value σ_a. Next we designate t_1, t_2, and t_3 as the thicknesses that correspond with the above three requirements. Then, from equations 4 through 8 we obtain

$$t_1 = [3P_1 b^2(1 + \beta)^2/NE_T\beta^2 u_1^2 \cos^2\alpha]^{1/3} \qquad (10)$$

$$t_2 = [3P_2 b^2(1 + \beta)^2/NE_T\beta^2 u_2^2 \cos^2\alpha]^{1/3} \qquad (11)$$

$$t_3 = [3b(1 + \beta)/2u_2^2 \cos^2\alpha]\left[\sqrt{v_2^2 + {}^4/_3\,(\sigma_a/E_T)(u_2^2/\beta^2)\cos\alpha} - v_2\right] \qquad (12)$$

where

b = half width of the seal (Figure 1),
u_1^2 = value of u^2 at 20 percent compression (Table 1),
u_2^2 = value of u^2 at 50 percent compression (Table 1), and
v_2 = value of v at 50 percent compression (Table 1).

The values t_1 and t_2 depend, as would be expected, on the mechanical properties of the material, the geometry of the seal section, and the number N of identical web members. The value of t_3 depends on the allowable stress σ_a but does not depend on N. For any given configuration, the correct design value of t is the largest of the three values t_1, t_2, and t_3.

Let us consider the design of the seal 1.75 cm wide, i.e., 2b = 1.75 cm. We shall assume the material properties E, β, and so on and the values of P_1 and P_2 to be the same as those in the previous example. In addition, let $\sigma_a = 242$ N/cm^2, the value of the allowable compressive stress. Then the only two parameters left are α and N. We can choose a specific configuration angle α and evaluate t_1, t_2, and t_3 from equations 10, 11, and 12 for a practical range of values of N (say, from 2 to 4) and in each case choose the largest value of t as the design value. Then we repeat the entire process for another configuration angle α. By repeating this procedure several times we obtain a set of design values. The values given in Table 2 were obtained by following this procedure. The computations involved are quite simple and straightforward and can be performed with the aid of a slide rule or a pocket computer.

In Table 2, in addition to the thickness, an extra column is added to each category to record numbers that are proportional to the total weight of the web members for the correct design choice. The total weight of the web members per unit length is given by W = $(2b\rho Nt)/\cos\alpha$, where ρ is the weight of the material per unit volume. The results of the weight analysis are shown in Figure 5. It is clear from Table 2 as well as Figure 5 that, in the present case, the lightest satisfactory design corresponds to the lower values of α and the lowest practical value of N. In this case the lowest practical value of N is 2. We also observe from Table 2 that, for $\alpha = 5$ deg and N = 2, the value of the thickness is governed by the value P_2. In all other cases the design value of t is dictated by the allowable stress. It must, however, be borne in mind that these remarks are valid for the current example. The design values are very much dependent on material properties. Hence, for a different material the indications of Table 2 may be altered.

CONCLUSION

It has been demonstrated in this paper that characteristic curves may be used effectively in making laboratory predictions as well as in designing section geometry. The paper has been confined, for convenience, to the simplest geometry with a high degree of symmetry. However, the procedures described can be extended to more complicated combinations where the web members are not identical in thickness. In such cases, the characteristic curves of Figures 3 and 4 can be used without change provided the seal section does not have a central vertical diaphragm. In such cases the expressions given in equations 4 and 5 should be replaced by the appropriate expression from the report by Vyas (2). In the design procedure illustrated here one can include additional constraints such as the maximum allowable tensile stress

and the maximum allowable shear stress. It will require extension of Table 2, but the basic procedure will remain the same. The important advantage of the approach suggested here is the relative ease with which one can handle a problem of nonlinear structural analysis without performing the analysis.

REFERENCES

1. R. K. Vyas. Analysis of Elastomeric Pavement Seals. HRB, Highway Research Record 357, 1971, pp. 72-79.
2. R. K. Vyas. Preformed Elastomeric Seals: Kinematics of Deformation and Some Basic Seal Properties. Univ. of Utah, Technical Rept. UTEC CE 70-213, June 1971.
3. G. M. Jones, D. E. Peterson, and R. K. Vyas. Evaluation of Preformed Elastomeric Sealing Systems and Practices. Materials and Tests Division, Utah State Highway Department, Final Rept., June 1971.
4. J. P. Cook. The Photoelastic Stress Analysis of a Preformed Compression Seal. HRB, Highway Research Record 320, 1970, pp. 1-8.