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CHLORIDE CORROSION OF STEEL IN CONCRETE

Tonini / Dean, *editors*

AMERICAN SOCIETY FOR TESTING AND MATERIALS

CHLORIDE CORROSION OF STEEL IN CONCRETE

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presented at the
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D. E. Tonini, American Hot Dip Galvanizers Association, Inc.
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Foreword

The symposium on Chloride Corrosion of Steel in Concrete was presented at the Seventy-ninth Annual Meeting of the American Society for Testing and Materials held in Chicago, Ill., 27 June–2 July 1976. Committee G-1 on Corrosion of Metals sponsored the symposium. D. E. Tonini, American Hot Dip Galvanizers Association, Inc., and S. W. Dean, Jr., Air Products and Chemicals, Inc., presided as symposium cochairmen and also as editors of this publication.

Related
ASTM Publications

Galvanic and Pitting Corrosion—Field and Laboratory Studies, STP 576 (1976), $29.75 (04-576000-27)

Manual of Industrial Corrosion Standards and Control, STP 534 (1974), $16.75 (04-534000-27)

Stress Corrosion Cracking of Metals—A State of the Art, STP 518 (1972), $11.75 (04-518000-27)

A Note of Appreciation to Reviewers

This publication is made possible by the authors and, also, the unheralded efforts of the reviewers. This body of technical experts whose dedication, sacrifice of time and effort, and collective wisdom in reviewing the papers must be acknowledged. The quality level of ASTM publications is a direct function of their respected opinions. On behalf of ASTM we acknowledge with appreciation their contribution.

ASTM Committee on Publications

Editorial Staff

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Introduction

The corrosion of steel in concrete exposed to chlorides has become a major problem for bridge and concrete engineers. This problem is evidenced by large-scale premature failures of reinforced concrete structures in a fraction of their design life. These failures have been traced to corrosion of the reinforcing steel and are generally sufficiently severe to require refurbishing or replacement of the structure.

Once the corrosion phenomena underlying this problem were understood, several techniques and materials were put forward as potential solutions to the problem. A symposium was organized by ASTM Subcommittee G01.04 on Atmospheric Corrosion to provide a forum for bringing together the experience of both laboratory and field engineers who have dealt with the problem.

The papers selected for inclusion in this publication are representative of the state of the art with respect to both research and field practice in controlling corrosion of steel in concrete. As presented here, both the researcher and the engineer will have a convenient compilation of the experience and recommendations of those who have already worked to solve the problem in various ways. As a result, it is hoped that the knowledgeable worker as well as those with less experience will have a basic reference document to assist in the selection of a material or method to apply to a specific situation. Some methods are better for new construction, others for rehabilitation of existing structures. Some are uniquely suited to bridge decks; others may be applied to a wider range of structural designs. We hope that the reader will be aided in his understanding and his ability to select a control method suited to his particular application.

Most of the evaluations of materials and methods are based upon extrapolations of laboratory data to field service conditions. Much work remains in validating many of the assumptions inherent in accelerated laboratory tests and in assessing the long-term implications of field measurement techniques. ASTM Committee G-1 on Corrosion of Metals has responded to this need by moving to organize a subcommittee to standardize procedures for measuring corrosion rates of materials exposed to chloride in concrete. In the interim, this publication should provide a useful future reference point as refinements are made to the methodology for designing reinforced structures for service in chloride environments.

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P. D. Cady[1]

Corrosion of Reinforcing Steel in Concrete—A General Overview of the Problem

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REFERENCE: Cady, P. D., "**Corrosion of Reinforcing Steel in Concrete—A General Overview of the Problem,**" *Chloride Corrosion of Steel in Concrete, ASTM STP 629,* D. E. Tonini and S. W. Dean, Jr., Eds., American Society for Testing and Materials, 1977, pp. 3-11.

ABSTRACT: Corrosion of reinforcing steel in concrete has produced a problem of catastrophic proportions in the highway field. Bridge decks in the snow belt areas of the country, subject to heavy application of deicer salts in response to the public's demand for year-round bare pavements, frequently start to show signs of severe deterioration after only a few years of service. A wide variety of techniques for retarding or preventing corrosion of reinforcing steel and subsequent potholes in bridge decks have been tried, with varying degrees of success. It appears that the time is ripe for a synthesis and evaluation of these methods in order to properly assess the current status of ameliorative actions and to indicate those areas requiring further investigative effort.

Corrosion of reinforcing steel in concrete is not limited to bridge decks. Examples of reinforcement corrosion in a marine atmospheric environment are cited.

KEY WORDS: corrosion, concretes, reinforcing steels, bridge decks, deicers, marine atmospheres

In the absence of certain deleterious factors and agents, the environment provided reinforcing steel by the surrounding concrete is almost ideally suited for preventing corrosion of the steel. High-quality concrete, properly placed, consolidated, and cured, provides a highly alkaline, low-permeability covering for the steel. However, the entrance of moisture, electrolytes, and oxygen by diffusion or through hairline cracks in the concrete can destroy the passive environment, establish galvanic corrosion cells, and eventually destroy the concrete through the expansive forces created by the corroding steel. Among the electrolytes, chloride salts seem to be the most vigorous in promoting this kind of attack.

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The problem of corrosion of reinforcing steel in concrete exposed to chloride salts has been known and studied for a long time [*1*–*5*].[2] However, it has achieved national prominence in recent years because of its impact on the maintenance and construction budgets of the state highway agencies. Therefore, most of the discussion in this paper will center around the corrosion of reinforcing steel in concrete bridge decks.

The Bridge Deck Problem

The observation that "potholing" of concrete bridge decks presents a very serious maintenance problem will come as no surprise, even to the casually observant driver. Scenes similar to the one portrayed in Fig. 1 will likely confront a vehicle operator on numerous occasions during a short drive in any of the snow belt states. The type of deterioration shown here is potholing, sometimes called spalling, delamination, or fracture plane. It is the most serious, in terms of structural damage and potential danger to motorists (though not in extent of afflicted area), of the three forms of deterioration found on bridge decks other than cracks. The other two forms are scaling and wear.

FIG. 1—*Potholes (some patched) on a concrete bridge deck.*

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[2] The italic numbers in brackets refer to the list of references appended to this paper.

It has been hypothesized [*6*] that the genesis of a pothole is as follows. When the concrete was placed, the resistance to subsidence of the fresh concrete afforded by the top mat of reinforcing steel caused small tension cracks to occur over the reinforcing bars. These provided channels for access by water and deicing chemicals (sodium and calcium chlorides) to the reinforcement. Subsequent corrosion of the reinforcing bars generated forces of sufficient magnitude to produce horizontal fracture planes. The layers of concrete above these fracture planes were then removed by the impact of vehicular traffic. It should be evident from this model that the depth of clear cover of concrete over the steel should be an important factor due to its effect on the magnitude of tensile stresses developed during subsidence. This has been shown to be the case in field examinations [*7*] and has been verified in the laboratory [*8*]. The relationship between the location of the reinforcing bars and spalls is shown in Fig. 2. Figure 3 shows a core extracted from a fracture plane area. Notice the location of the fracture plane above the top reinforcement. Figure 4 is a photomicrograph showing the shattered concrete in the vicinity of a corroding reinforcing bar.

There has been a dramatic increase in the incidence and severity of bridge deck deterioration in the last decade, which accounts for the upsurge in interest in this topic. Two factors have been largely responsible

FIG. 2—*Fracture planes and spalls. Extent of the fracture planes and the locations of the reinforcing bars are indicated.*

FIG. 3—*Core through fracture plane.*

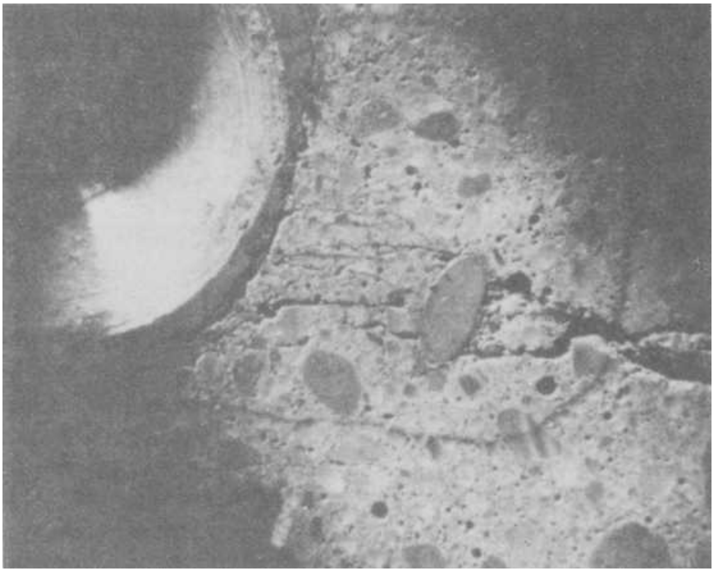

FIG. 4—*Shattered concrete in vicinity of corroded reinforcing bar (upper left-hand corner).*

for precipitating this situation. First, the large volume of highway construction associated with the building of the Interstate Highway System produced a significant increase in the number of highway structures. Second, the so-called "bare pavement" policy, stressing safe, all-weather driving conditions, was adopted by the various highway agencies in response to public pressure. The latter factor is, by a wide margin, the more important of the two in its impact on the pothole problem since its objective was achieved through a phenomenal increase in the use of chloride deicing chemicals. For example, less than one-half million tons of salt were used throughout the United States for deicing highways in 1947. By the winter of 1966–1967, this figure had risen to over 6 1/2 million tons, and this year it is expected to be between 11 and 12 million tons [*9*]. A typical highway pavement in one of the snow belt states will receive about a quarter of a pound of salt per square foot during the course of a winter. The effect of increased deicer use is illustrated dramatically by the fact that surface spalls were quite rare in California until the late 1960's when the use of chemical deicers was instituted in that state [*10*].

The problem of reinforcement corrosion and resultant potholing primarily affects the bridge decks because the roadway pavements contain less reinforcing steel and are not salted as heavily. There are approximately 560 000 bridges in this country, of which 230 000 are on federally funded highways. State highway agencies expect bridges to last 30 years or more, but many show signs of reinforcement corrosion and delamination in five years or less [*11*]. In 1973, it was reported that the Federal Highway Administration estimated the annual cost for bridge deck repairs in the United States at $70 million [*12*]. By 1975, that figure had increased to $200 million per year [*13*]. These figures are deceptively low as measures of actual damage being incurred when one considers that they represent only the amounts actually being spent for repairs. In other words, they represent the bounds of limited maintenance budgets. The average motorist needs no special powers of perception to observe that all of the potholes are not being repaired! Furthermore, we are just at the threshold of the period in which the large number of bridges built during the interstate construction program are starting to show signs of deterioration.

The fact that is frightening to most knowledgeable highway engineers is the early age at which deterioration begins to appear. In 1970, researchers at The Pennsylvania State University under contract to the Pennsylvania Department of Transportation and the Federal Highway Administration inspected 249 four-year-old bridge decks in the Commonwealth [*14*]. Nearly one fourth (22 percent) of the bridges had at least one pothole or fracture plane. The 55 affected decks had an average of 23 potholes or fracture planes covering 36 ft^2. Again, all of the 249 bridges were built in 1966 and had experienced only four winter seasons. In addition to the fact that nearly one fourth of the bridge decks displayed potholes or fracture planes, there was evidence that the worst was yet to come in that 71 percent of the decks exhibited transverse cracks, the initial stage in the genesis of a pothole. An average of 41 transverse cracks was observed on the decks so afflicted.

In the early 1960's, the Portland Cement Association and the Bureau of Public Roads (now the Federal Highway Administration), in cooperation with ten state highway agencies, carried out detailed inspections of a total of 70 bridge decks in 4 states and random surveys of an additional 100 to 150 bridges in each of 8 states [*10,15*]. In the detailed inspections, 40 percent of the decks exhibited surface spalls. In the random surveys, 12 percent of the bridges were afflicted, but the incidence of spalling varied considerably from one state to another, with 65 percent of the bridges in one state exhibiting spalling. Transverse cracks were observed on approximately 80 percent of the bridges inspected in both surveys, and their locations nearly always occurred over the top reinforcing bars.

The magnitude of the maintenance problems that we face now, and even more critically in the future, should, at this juncture, be coming into focus. Taking the data cited previously for the number of bridges in this country and the incidence of potholing (even recognizing that the problem is largely confined to the snow belt states) and combining this with deck repair costs (currently estimated at \$2.50 to \$10.00/ft^2, depending on repair method), figures rivaling the national debt can be envisioned.

Attempts to Solve the Problem

In the construction of new bridge decks, a variety of measures have been proposed to deter the problem. In the early 1960's, the New York State Department of Transportation built over 30 bridges with waterproof membranes topped with asphaltic concrete wearing courses. The membranes consisted of polyester resin or epoxy-modified coal tar. There is evidence to suggest that this procedure does retard the corrosion problem [*16*]. The New Jersey Turnpike Authority has used a similar system since 1963 and more recently (since 1973) has employed preformed membranes in new deck construction, all with reportedly good results. The Vermont Department of Highways has done a considerable amount of research in the use of waterproof membranes on bridge decks and favors the use of the preformed type [*17*]. The National Cooperative Highway Research Program also funded an extensive study to identify and prepare specifications for suitable membrane systems [*18*].

Latex-modified portland cement mortar overlays have been used with varying degrees of success on old, as well as new, bridge decks to retard the pothole problem. The Pennsylvania Department of Transportation has constructed 10 such decks since 1966 and has plans for another 134 [*11*].

Impregnation of old and new concrete bridge decks with polymeric materials to prevent ingress of water and deicing salts has shown considerable promise in preventing reinforcement corrosion in the laboratory and small-scale trials, but it is very expensive and still lacks long-term field evaluation [*19*]. Research is currently under way by the Federal Highway Administration to develop an internally sealed concrete for bridge decks using wax particles which are subsequently melted to seal the pores [*20*].

Attempts to protect the reinforcing steel through the use of corrosion inhibitors [*21*], cathodic protection [*22*], and epoxy coatings and galvanizing have been tried. The Pennsylvania Department of Transportation, for example, has built 4 decks with epoxy-coated reinforcing bars and plans to build 77 more, and has constructed 11 with galvanized rebars, with plans for an additional 171 [*11*]. Research is even under way to attempt to neutralize or remove the chlorides from salt-contaminated bridge decks.

Other Areas of Concern

To this point, only the impact of corrosion of reinforcement in concrete highway bridge decks has been discussed since it is a very serious problem currently receiving considerable attention. However, concrete in other types of structures and in other environments can also be subject to deterioration due to reinforcement corrosion. The most important of these are reinforced concrete structures exposed to seawater [*23*] and marine atmospheric environments. It is easily recognized that the same destructive elements found in the bridge deck problem—chloride salts, water, and oxygen—are also present here. The fact that corrosion of reinforcement can occur under these considerations is illustrated in Fig. 5. This is a pipe support at a seacoast oil refinery located on an island in the Caribbean. At this particular location, problems of the type shown in Fig. 5 created constant maintenance problems. In some instances, complete structural failure of the concrete member resulted, as in the case of the column shown in Fig. 6.

Conclusions

In conclusion, it has been shown that the problems resulting from corrosion of reinforcing steel in concrete are widespread and very serious. This is especially true of the bridge decks on the highways of the snow belt states, where increased deicer salt use associated with the bare pavement policy has resulted in maintenance expenses that are currently costing the American taxpayers $200 million per year and are rapidly rising. Corrosion of reinforcing steel in concrete exposed to marine atmospheric environments was also shown to be a problem.

A considerable amount of testing and research has been carried out on methods and techniques to retard or prevent corrosion of reinforcing steel in bridge decks. It certainly appears that the time is ripe for a major, centralized effort to collect, evaluate, and synthesize that work. The author believes that such an effort would provide invaluable information

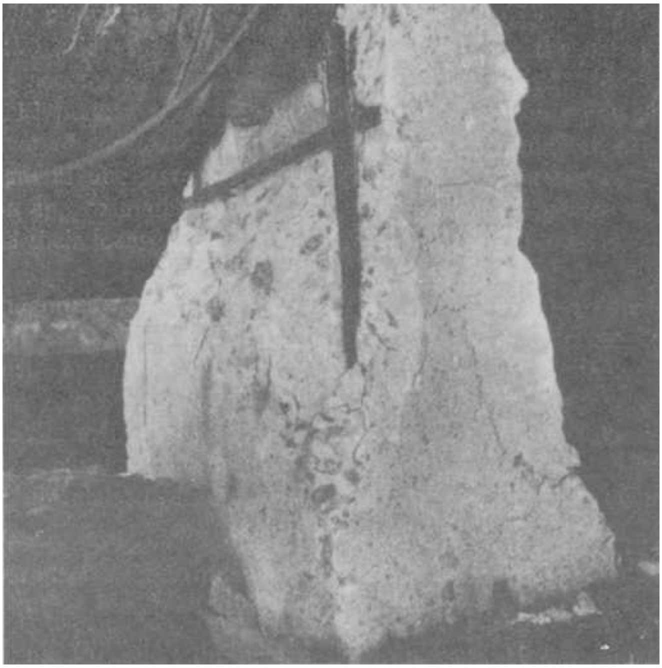

FIG. 5—*Corrosion of reinforcing steel in a tropical marine environment.*

to highway agencies in the fight to defuse the bridge deck problem, as well as to indicate those areas requiring further investigative effort.

References

[*1*] Brown, H. P., *Engineering News,* Vol. 65, 1911, p. 684.

[*2*] Wig, R. J., *Engineering News Record,* Vol. 79, 1917, pp. 689-693.

[*3*] Maggee, G. M., *Corrosion,* Vol. 5, No. 11, Nov. 1945, pp. 378-382.

[*4*] Stratfull, R. F., *Corrosion,* Vol. 13, No. 3, March 1957, pp. 173-178 t.

[*5*] Lewis, D. A. and Copenhagen, W. J., *Corrosion,* Vol. 15, No. 7, July 1959, pp. 382-388 t.

[*6*] "Concrete Bridge Deck Durability," *NCHRP Synthesis of Highway Practice—4,* Highway Research Board, National Cooperative Highway Research Program, 1970.

[*7*] Carrier, R. E. and Cady, P. D., "Factors Affecting the Durability of Concrete Bridge Decks," Special Publication SP-47, American Concrete Institute, 1975, pp. 121-168.

[*8*] Dakhil, F. H., Cady, P. D., and Carrier, R. E., *Journal,* American Concrete Institute, Vol. 72, No. 8, Aug. 1975, pp. 421-428.

[*9*] "Environmental Impact of Highway Deicing," *Water Pollution Control Research Series 11040 GKK 06/71,* Environmental Protection Agency, June 1971.

[*10*] "Durability of Concrete Bridge Decks," Final Report, Portland Cement Association and U.S. Bureau of Public Roads in Cooperation With Ten State Highway Agencies, 1970.

[*11*] Sandvig, L. D., *Highway Builder,* Nov. 1974, pp. 16-17, 42.

[*12*] Dallaire, G., *Civil Engineering,* Vol. 43, No. 8, Aug. 1973, pp. 43-48.

[*13*] Godfrey, K. A., Jr., *Civil Engineering,* Vol. 45, No. 8, Aug. 1975, pp. 60-65.

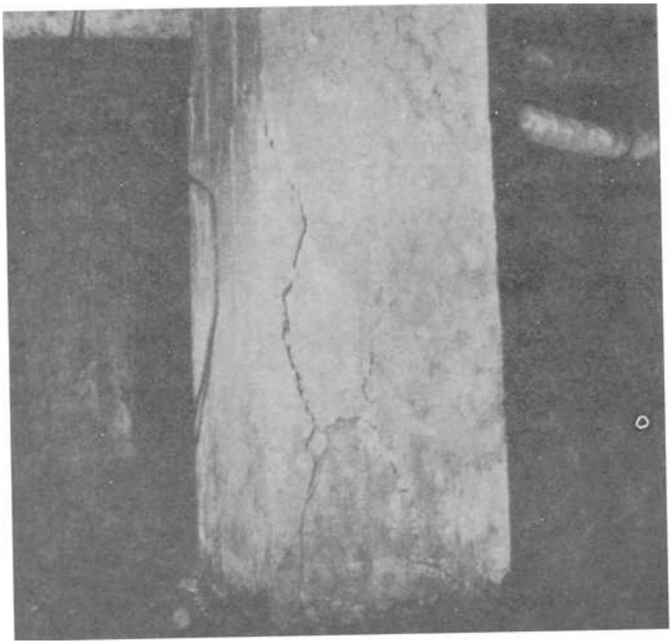

FIG. 6—*Failure of a column due to corrosion of reinforcing steel in a tropical marine environment.*

[*14*] Carrier, R. E. and Cady, P. D., "Deterioration of 249 Bridge Decks," Highway Research Record No. 423, Highway Research Board, 1973, pp. 46–55.

[*15*] "Durability of Concrete Bridge Decks," Report 5, Portland Cement Association and U.S. Bureau of Public Roads in Cooperation With Ten State Highway Agencies, 1969.

[*16*] Cady, P. D. and Renton, J. B., "Durability of Steel-Formed, Sealed Bridge Decks," Final Report on AISI Project 1201-405, American Iron and Steel Institute, 1 July 1975.

[*17*] Frascoia, R. I., "Experimental Bridge Deck Applications in Vermont," Report 74-4, Vermont Department of Highways, April 1974.

[*18*] VanTil, C. J., Carr, B. J., and Vallerga, B. A., "Waterproof Membranes for Protection of Concrete Bridge Decks," Final Report, NCHRP Project 12-11, Transportation Research Board, National Cooperative Highway Research Program, July 1973.

[*19*] Manson, J. A., Chen, W. F., Vanderhoff, J. W., Mehta, H. C., Cady, P. D., Kline, D. E., and Blankenhorn, P. R., "Use of Polymers in Highway Concrete," Final Report, NCHRP Project 18-2, Transportation Research Board, National Cooperative Highway Research Program, Sept. 1975.

[*20*] Clear, K. C. and Ormsby, W. C., "Concept of Internally Sealed Concrete," Report No. FHWA-RD-75-21, Federal Highway Administration, March 1975.

[*21*] Craig, R. J. and Wood, L. E., "Effectiveness of Corrosion Inhibitors and Their Influence on the Physical Properties of Portland Cement Mortars," *Highway Research Record No. 328,* Highway Research Board, 1970, pp. 77–88.

[*22*] Stratfull, R. F., "Experimental Cathodic Protection of a Bridge Deck," *Highway Research Record No. 500,* Highway Research Board, 1974, pp. 1–15.

[*23*] Beaton, J. L., Spellman, D. L., and Stratfull, R. F., "Corrosion of Steel in Continuously Submerged Reinforced Concrete Piling," *Highway Research Record No. 204,* Highway Research Board, 1967, pp. 11–21.

P. K. Mehta[1]

Effect of Cement Composition on Corrosion of Reinforcing Steel in Concrete

REFERENCE: Mehta, P. K., "**Effect of Cement Composition on Corrosion of Reinforcing Steel in Concrete,**" *Chloride Corrosion of Steel in Concrete, ASTM STP 629,* D. E. Tonini and S. W. Dean, Jr., Eds., American Society for Testing and Materials, 1977, pp. 12-19.

ABSTRACT: The tricalcium aluminate present in portland cement is known to be effective for chloride removal and can thus provide protection against steel corrosion. Contradictions are found in literature with regard to the minimum tricalcium aluminate content of a cement which is desirable to prevent corrosion of reinforcing steel in concretes exposed to chlorides. A review of both published experimental data and some theoretical considerations are presented to show that not only the amount of tricalcium aluminate present but also its crystallographic type and the source of chloride are necessary factors in predicting the corrosion behavior of steel in reinforced concrete.

KEY WORDS: reinforcing steels, concretes, corrosion, chlorides, cements, aluminates, calcium sulfates, hydrates

Corrosion of steel is associated with electrolytic cells which are formed due to inhomogeneities in steel or in the surrounding environment. As a result of atmospheric oxidation, a thin film of iron oxide is usually present at the surface of the metal. When stabilized under certain environmental conditions, this film can protect steel from corrosion.

With regard to corrosion of reinforcing steel in concrete, several investigators, including Steinour [*1*],[2] concluded that occurrence of any significant corrosion depends both on availability of oxygen and the impairment of the protective oxide film. Normally, the film would remain stable

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[2] The italic numbers in brackets refer to the list of references appended to this paper.

in the high pH environment produced by the hydration of portland cement. However, it may become impaired either due to lowering of the pH, such as caused by carbonation of concrete, or due to presence of sufficient concentration of chloride ions in solution. It is well known that portland cement plays an important role in preventing corrosion of the reinforcing steel by providing an alkaline environment under which the iron oxide film remains stable. What is not so well known is yet another protective influence due to portland cement, namely, the fact that certain portland cement compounds or their hydration products can reduce the deleterious effect of chloride by removing from the liquid phase a considerable proportion of the chloride.

The amount of solid calcium hydroxide which is produced by complete hydration of any of the five American Society for Testing and Materials (ASTM) standard portland cements varies from a minimum of 20 percent for the Type IV portland cement to a maximum of 30 percent for the Type III portland cement. This provides enough reserve basicity in concrete to maintain a high pH of the pore fluid which is important for stability of the protective film on steel. Thus, with regard to basicity of the environment, differences in the composition of portland cements should have little effect on steel corrosion unless the amount of solid calcium hydroxide produced by hydration has been drastically reduced, such as in autoclaved concrete products or in concretes containing large additions of reactive siliceous materials. However, there is evidence that the differences in composition of portland cements play an important role in steel corrosion associated with the chlorides. The author has reviewed the limited experimental data available and has presented some theoretical considerations in the hope of stimulating further research on this subject.

Sources of Chlorides in Concrete

The chlorides may be present in fresh concrete but they may also permeate into hardened concrete from environments containing chlorides. One source of chloride in fresh concrete is calcium chloride which is frequently used as an accelerator for strength. For this purpose, generally about 2 percent calcium chloride ($CaCl_2 \cdot 2H_2O$) (or 1.4 percent anhydrous calcium chloride) by weight of cement is added to the concrete mixing water. Another source of chloride in fresh concrete is the use of seawater as concrete mixing water in certain coastal areas where fresh water may not be readily available. Concrete structures exposed either to marine environments or to deicer chloride salts represent the typical cases where chloride ions may not initially be present in fresh concrete but subsequently would find their way into hardened concrete by diffusion of the chloride solution.

Effect of Cement Composition on Chloride Removal

It is known that tricalcium aluminate (C_3A)[3] of portland cement can react with a chloride solution and take much of the chloride out of the solution by forming an insoluble compound, $C_3A \cdot CaCl_2 \cdot 10H_2O$. Roberts [*2*] obtained experimental evidence from laboratory tests in which steel wires were embedded in portland cement concretes both with and without calcium chloride added to the mixing water used for making concrete prisms. A very slight corrosion was recorded for the Type I portland cement (9 percent C_3A) when 1.4 percent $CaCl_2$ by weight of cement was present, while considerable corrosion occurred for the Type V portland cement (1 percent C_3A) containing the same amount of $CaCl_2$. No corrosion of the reinforcement was reported for either the low C_3A cement concrete or the high C_3A concrete when no calcium chloride was present. These conclusions were drawn from normally cured concrete prisms stored in air for periods up to two years.

In other tests by Roberts [*2*], both the Type I and Type V portland cements were separately treated with $CaCl_2$ solutions of varying concentration, and the filtrate was analyzed for chloride at regular intervals. The data in Fig. 1 show the changes in the chloride concentration of the solution phase when the amount of mixing water used corresponded to 1.0 water/cement ratio (w/c), and the water contained 1.4 percent $CaCl_2$ by weight of cement.

It can be seen from the data that the chloride concentration in solution became constant after about three days. In the case of low C_3A cement (Type V), the chloride concentration registered a drop from about 9 to 6 g/litre, whereas in the case of high C_3A cement (Type I), the chloride

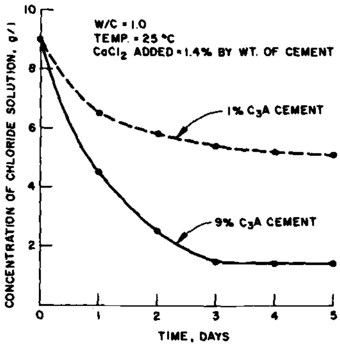

FIG. 1—*Effect of cement composition on the chloride concentration of the aqueous phase* [2].

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[3] Abbreviations used in cement chemistry are: C = calcium oxide, S = silica, A = aluminum oxide, F = ferric oxide, $\bar{S}$ = sulfur trioxide, H = water.

concentration dropped from about 9 to 2 g/litre, thus proving that the high C_3A cement was able to remove about 7 g/litre chloride from the solution phase as compared with about 3 g/litre for the low C_3A cement. For the high C_3A cement, the small quantity of the residual chloride remaining in solution under equilibrium conditions was not found to be significant in Roberts' corrosion tests.

The data in Fig. 2 confirm the results shown in Fig. 1, the differences being due to the higher w/c and the higher proportions of $CaCl_2$ added to the mixing water. With increasing proportions of $CaCl_2$, the residual chloride concentration in the solution phase increased proportionately for both the cements, but, in all cases, the chloride concentration was much higher for the low C_3A cement than for the high C_3A cement.

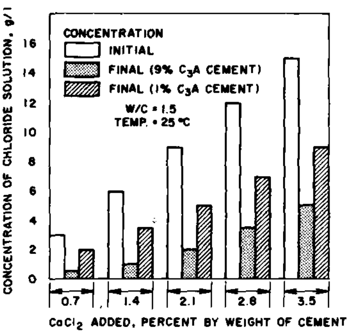

FIG. 2—*Effect of cement composition on the chloride concentration of the aqueous phase* [2].

Since there was 19 percent C_4AF in the Type V cement and 10 percent C_4AF in Type I cement, it can also be concluded from Roberts' corrosion data that C_4AF had little or no effect on chloride removal.

Verbeck [*3*] investigated the effect of long-term exposure to seawater at St. Augustine, Florida, of reinforced concrete piles made with 22 portland cements of varying composition. The data on effect of C_3A content of cement on concrete cracking due to corrosion of steel having 1 1/2-in. (38-mm)-thick concrete cover is shown in Fig. 3. From the data, it can be concluded that the average linear cracking was 36 ft (10.97 m) for the 2 to 5 percent C_3A cements, 30 ft (9.14 m) for the 5 to 8 percent C_3A cements, and 13 ft (3.96 m) for the 8 to 11 percent C_3A cements. Thus, the threefold reduction in cracking associated with corrosion of steel due to permeation of chloride containing seawater into hardened concrete occurred only when the C_3A content of the portland cement was above 8 percent.

Shalon and Raphael [*4*] investigated corrosion of steel reinforcement embedded in mortar bars made with a 12 percent C_3A portland cement

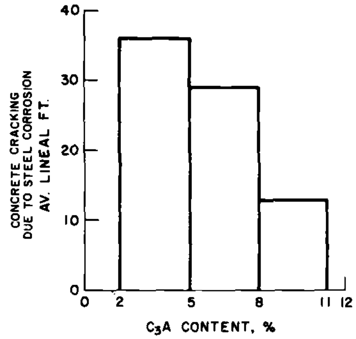

FIG. 3—*Effect of C_3A content on concrete cracking due to steel corrosion* [3].

and varying quantities, 0.5 to 1.1 w/c, of Mediterranean seawater which contained 2.7 percent sodium chloride (NaCl) and 0.4 percent magnesium chloride ($MgCl_2$). The mortar bars which were stored in moist air for periods up to four years showed considerable corrosion of the reinforcement. This showed that, from the standpoint of steel corrosion under some conditions, even high C_3A portland cements may not be effective chlorided-removing agents. In this context, therefore, it is interesting to note that structural designers continue to recommend 5 to 6 percent C_3A cements as adequate for durability of reinforced concrete for seawater exposure [5]. The apparent contradiction in the conclusion drawn by several researchers on the effect of C_3A content of portland cement on the chloride-related corrosion of steel in concrete proves the need for additional theoretical and experimental work on this subject.

Chlorides Present in Mixing Water

Stoichiometric calculations can be made to show that if 2 percent flake $CaCl_2 \cdot 2H_2O$ (corresponding to 1.4 percent $CaCl_2$) by weight of cement is added to concrete mixing water, it will require 3.4 percent C_3A in cement for complete removal of the chloride as $C_3A \cdot CaCl_2 \cdot 10H_2O$. The situation is complicated by the fact that there is demand for the aluminate by the sulfates which are invariably present in the system. Since sulfate corresponding to about 5 percent gypsum ($CaSO_4 \cdot 2H_2O$) is usually added to all portland cements for regulation of time of set, there is competition for the available aluminate present between the chloride and the sulfate ions.

Schwiete et al [6] investigated the order of combination of aluminates in pastes containing calcium hydroxide, gypsum, and calcium chloride. The combination of the sulfate and chloride occurred in the following order: formation of the trisulfoaluminate hydrate until the sulfate was consumed, formation of the chloroaluminate hydrate until the chloride was consumed, and formation of the monosulfoaluminate hydrate from the trisulfoaluminate and excess aluminate or aluminoferrites present.

On stoichiometric considerations, 2.7 percent C_3A will be needed for combination with the 5 percent $CaSO_4 \cdot 2H_2O$ to form the trisulfoaluminate hydrate and, subsequently, 3.4 percent C_3A will be needed for removal of 1.4 percent $CaCl_2$ present in mixing water. Therefore, from the standpoint of prevention of steel corrosion due to chloride, more than 6 percent C_3A in cement will be needed for effective chloride removal. Evidently, Type V portland cements, or even most Type II portland cements, would not meet this criterion. Only Type I or Type III portland cements having more than 6 percent actual C_3A would be effective in chloride removal and would provide adequate protection against corrosion of reinforcement.

Considering the case where seawater was used as mixing water [*4*], in addition to 2.7 percent C_3A required for combination with the gypsum present in cement, about 7 percent C_3A was needed for chloride removal, assuming that NaCl could react with $Ca(OH)_2$ produced by the hydrating portland cement to yield an equivalent amount of $CaCl_2$. This amounts to 9.7 percent C_3A requirement as compared with 12 percent potential C_3A in cement reported by Shalon and Raphael [*4*].

Two additional factors must be considered. The compound composition shown for cements is generally the potential compound composition and not the actual compound composition as determined by X-ray diffraction or microscopic analyses. It should be noted that, in high C_3A portland cements, the actual C_3A present may be about 2 percent less than the potential C_3A which is usually reported [*7*]. Also, it should be noted that, in addition to the gypsum present in cement, seawater contains small quantities of magnesium sulfate ($MgSO_4$) and $CaSO_4$. Theoretically, when portland cement is hydrated in the presence of sulfates and chlorides, the sulfates enter preferentially into combination with C_3A. Hence, for a given quantity of chloride in mixing water, more chloride will remain free in solution in the presence of sulfates than in the absence of sulfates. Perhaps this accounts for the fact that even 12 percent potential C_3A was not adequate for chloride removal when seawater was used as mixing water in the studies reported by Shalon and Raphael [*4*].

Permeation of Chloride Solutions into Hardened Concrete

Diffusion of solution into portland cement concrete from outside is a slow process. Depending upon the thickness of a structural member and its permeability, diffusion of a chloride-bearing solution into the interior of concrete may take several months or even years. Meanwhile, the aluminates present in cement would have hydrated to calcium aluminate hydrates or calcium sulfoaluminate hydrates.

In a hydrated cement paste containing aluminate, calcium, and sulfate ions, Lerch et al [*8*] showed the presence of two calcium sulfoaluminate hydrates, the trisulfoaluminate hydrate ($C_3A \cdot 3CaSO_4 \cdot aq$), and the monosulfoaluminate hydrate ($C_3A \cdot CaSO_4 \cdot aq$). The authors also reported that both the sulfoaluminate hydrates remained unchanged in the presence of chloride solutions. If this is so, it means that C_3A, once converted to the sulfoaluminate hydrates, would no longer be available for chloride removal. Alternatively, only the C_3A which has hydrated to calcium aluminate hydrates would be available for chloride removal.

Due to relatively high solubility of the calcium sulfates which are present in portland cements in the form of gypsum, hemihydrate, or soluble anhydrite, the trisulfoaluminate hydrate forms quickly in a freshly hydrated portland cement paste. In Type I, II, and III portland cements, where considerably more than 2.7 percent C_3A is normally present, the trisulfoaluminate hydrate converts over a period of a few days to several weeks into the monosulfoaluminate hydrate. Stoichiometric calculations show that 8 percent C_3A will be needed for complete combination of 5 percent gypsum into the monosulfoaluminate hydrate. Since Type II portland cements are limited to a maximum of 8 percent C_3A, it is obvious that hardened concretes made with Type I and Type III portland cements having more than 8 percent C_3A will alone have enough aluminate available in the form of calcium aluminate hydrates which is effective in chloride removal from a permeating solution. It should be noted that this type of theoretical approach is able to explain the results of Verbeck [*3*], where significant reduction in concrete cracking due to corrosion of reinforcing steel was not observed until the C_3A content of the portland cements was more than 8 percent.

Recently, Regourd [*9*] reported that the formation and the stability of the chloroaluminate hydrate in portland cement pastes exposed to seawater may be related to the crystallographic form of C_3A present in cement. It is now known that C_3A can exist in commercial portland cements in three crystallographic forms, cubic, orthorhombic, or tetragonal. In hydrated cement pastes exposed to seawater, Regourd discovered that ettringite rather than the chloroaluminate hydrate was the preferred phase in portland cements containing cubic C_3A, but significant proportions of the chloroaluminate hydrate coexisted with ettringite in cements containing orthorhombic or tetragonal C_3A. Obviously, such findings should have great significance for any efforts which are made to relate cement composition with corrosion of steel in presence of chlorides.

Conclusions

The chloride-related corrosion of reinforcing steel in concrete can be influenced by the C_3A content of a cement. It is possible to predict the amount of C_3A desired in cement which is necessary for effective chloride removal. This amount depends on the sulfate content of the cement and the source of chlorides in concrete.

Assuming that sulfate equivalent to 5 percent $CaSO_4 \cdot 2H_2O$ by weight of cement is typically present in portland cements, stoichiometric calculations are made to determine the C_3A required for removal of both the sulfate and the chloride present. When the source of chloride is in concrete mixing water, the calculations assume that ettringite formation takes precedent over removal of chloride as the calcium chloroaluminate hydrate. However, in case the availability of chloride is delayed, such as when chloride-bearing water has to diffuse into hardened concrete, it is assumed that C_3A for chloride removal is available only after the C_3A corresponding to the formation of calcium monosulfoaluminate hydrate has been used up. These calculations show that if 2 percent flake calcium chloride weight of cement is present in fresh concrete, portland cement containing at least 6 percent C_3A will be desired for effective chloride removal. Evidently, Type V portland cement and many Type II portland cements would not be able to meet this requirement. In situations when chloride is available through solutions permeating into hardened concrete, such as concrete exposed to seawater, portland cement containing at least 8 percent C_3A will be needed for effective chloride removal. Here, only Type I portland cements containing more than 8 percent C_3A would be applicable.

References

[*1*] Steinour, H. H., "Influence of the Cement on the Corrosion Behavior of Steel in Concrete," Bulletin 168, Portland Cement Association, 1964, p. 14.

[*2*] Roberts, M. H., *Magazine of Concrete Research,* Vol. 14, No. 42, 1962, pp. 143-154.

[*3*] Verbeck, G. J., *Performance of Concrete,* E. G. Swenson, Ed., University of Toronto Press, Toronto, 1968, pp. 113-124.

[*4*] Shalon, R. and Raphael, M., *American Concrete Institute Journal, Proceedings,* Vol. 55, No. 6, 1959, pp. 1251-1268.

[*5*] Gerwick, B. C., "Durability of Concrete," American Concrete Institute, SP 47, 1974, pp. 317-324.

[*6*] Schwiete, H. E., Ludwig, U., and Albeck, J., "Combination of Calcium Chloride and Calcium Sulfate in Hydration of Aluminate-Ferritic Clinker Constituents, "Zement-Kalk-Gips, No. 5, 1969, pp. 225-234.

[*7*] Moore, A. E., "Comparison of the Results Obtained for the Compound Composition of Portland Cements by XRD, Microscopy, and Wet Chemical Methods," Symposium on Analysis of Calcareous Materials, Society of Chemical Industry, Monograph No. 18, 1965, pp. 372-390.

[*8*] Lerch, W., Ashton, F. W., and Bogue, R. H., *National Bureau of Standards, Journal of Research,* Vol. 2, RP 54, 1929, pp. 715-731.

[*9*] Regourd, M., Annales de l' Institute Technique du Batiment et des Travaux Publiques, No. 329, 1975, pp. 86-102.

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Influence of Chloride in Reinforced Concrete

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ABSTRACT: The present state of knowledge regarding the action of chloride, principally calcium chloride, in reinforced concrete is reviewed, with special emphasis on its effect on the reinforcing steel. The reactions through which calcium chloride accelerates the set and strength of portland cement are explained, as well as the significance these reactions with cement have with regard to the potential corrosion of steel reinforcement in concrete. Calcium chloride's beneficial effects in ordinary reinforced concrete are enumerated, along with the precautions regarding its use in prestressed concrete and special situations. The deterioration of bridge decks and the significance of the "chloride content corrosion threshold" that has been suggested for bridge decks are discussed. It is pointed out that the limit for chloride that has been suggested as a guide to determine the extent of repair to existing bridge decks that have been subject to repeated application of deicer salts is not a requirement that should be applied to limit the chloride content of ordinary reinforced concrete. When the amount of commercial calcium chloride is limited to two percent by weight of cement, the degree of corrosion in ordinary reinforced concrete is insignificant if the concrete is reasonably dense and there is adequate cover over the steel.

KEY WORDS: reinforcing steels, concretes, corrosion, chlorides, prestressed concrete, admixtures, calcium chlorides, portland cement, pH

The objective of this paper is to review briefly the state of knowledge regarding the action of calcium chloride in concrete, with special emphasis on its effect on the reinforcing steel and then to discuss and evaluate the chloride limitations that are being required in various codes and specifications.

Although the beneficial effects of calcium chloride have been known since 1885 when W. Millar and C. F. Nichols obtained an English patent

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[1][2] covering its use to accelerate the setting and hardening of cements, it has only been during the past 50 years or so that the use of calcium chloride in concrete has been the subject of considerable interest, numerous studies, investigations, and reports. One of the first cooperative series of tests was conducted by the American Society for Testing and Materials (ASTM) Committee C0901. The published results in 1923 [*2*] concluded that calcium chloride markedly accelerated the hydration of cement during the first few days, but the resulting gain in strength was less at later ages. That same year, three papers [*3–5*] were published on the general subject of the effect of calcium chloride on the corrosion of steel in concrete, and, in each case, the authors concluded that the use of chloride did not have a detrimental action on the steel. Since 1923, there have been hundreds of papers published on the subject of calcium chloride in concrete. In 1952, the Highway Research Board published an annotated bibliography [*6*] comprising 186 references on the subject.

There appears to be an increasing interest in the role that chlorides play as a potential corrosion hazard in reinforced concrete. In discussing this subject, it is well to separate it into three areas: (*a*) prestressed concrete, (*b*) concrete bridge decks, and (*c*) plain reinforced concrete.

It should be pointed out over 90 percent of reinforced concrete can probably be classified as plain reinforced concrete, but by far the majority of the recent papers, discussions, and publications regarding the potential corrosion hazard of calcium chloride have been primarily concerned with bridge decks and prestressed concrete.

It is generally agreed that calcium chloride and admixtures containing added amounts of chloride present a corrosion hazard when used in prestressed concrete, and a restriction against their use is now being included in numerous specifications, codes, and standards.

Concrete Bridge Decks

The deterioration of concrete bridge decks caused by the corrosion of reinforcing steel induced by the use of chloride deicers is one of the most severe problems currently facing the highway industry. A recent report by Clear [*7*] suggests a chloride content corrosion threshold limit which is defined as the minimum quantity of total chloride required to initiate reinforcing bar (rebar) corrosion in portland cement concrete when sufficient moisture, oxygen, and other necessary factors are present. To understand the background for this threshold limit of chloride, it should be noted that the bridge deck deteriorations problem is principally a delamination of the concrete near the level of the top mat of reinforcing steel and subsequent spalling of the surface concrete. Clear and Hay [*8*] re-

[2] The italic numbers in brackets refer to the list of references appended to this paper.

ported that research has shown the most prevalent cause of this type of distress to be corrosion of the reinforcing steel due to the intrusion of chlorides into the concrete from repeated deicer applications for snow and ice removal. A large research effort has been undertaken to provide solutions to this problem. Waterproof membranes, epoxy-coated reinforcing steel, galvanized steel, latex-modified concrete, and polymer-impregnated concrete are some of the potential solutions in various stages of development. Cathodic protection, neutralization of chlorides, and polymer impregnation are among the techniques being studied for application to existing structures. However, at the present time, concrete bridge decks continue to deteriorate, and they must be repaired. Certain patching materials and overlay systems are available for use in reconstruction, but it is still necessary to determine the portion of the concrete that must be removed prior to repair. The purpose of the corrosion threshold limit for chloride suggested by Clear *[9]* is to determine by the analysis of concrete cores from bridge decks whether or not that portion of concrete represented by the cores should be removed previous to the permanent bridge deck repair operation. The threshold limits suggested on the basis of Clear's evaluation of the problem are:

Less than 1.0 lbs Cl^-/yd^3—leave concrete intact.

Greater than 2.0 lbs Cl^-/yd^3—remove concrete below top mat of rebars or replace the entire deck.

1.0 to 2.0 lbs Cl^-/yd^3—questionable area. The decision as to whether or not to remove concrete with chloride contents in this range will depend on the engineer's willingness to accept the risk and cost of future corrosion problems.

Recently, there have been instances where specification writers for construction involving regular reinforced concrete structures have written a needlessly restrictive specification with regard to chloride content, that is, where the amount of chloride has been limited to the "chloride content corrosion threshold" suggested for bridge decks. This is an unreasonable restriction because most of the chloride determined in the bridge deck survey was derived from repeated application of deicer salts applied to the surface of the concrete after it was placed in service, and most of the chloride penetrating into the concrete would be available for a corrosion reaction. This differs from the addition of small amounts of calcium chloride to the plastic concrete, in which case, a large portion of the chloride is taken out of solution as a result of reactions during the hydration of portland cement. Further, the chloride content corrosion threshold, as previously mentioned, is defined as the minimum quantity of total chloride required to initiate rebar corrosion in portland cement concrete when sufficient moisture, oxygen, and other necessary factors are present. The presence and availability of these factors are closely related to the cracking and spalling of concrete bridge decks which are caused, to a large extent, by the application of deicer salts and are entirely different, for example, from the service conditions of a reinforced concrete building.

One of the most comprehensive series of tests on the use of calcium chloride in concrete was conducted in the laboratories of the U.S. Bureau of Reclamation, and the results were reported in a paper by Shideler [*10*] which confirmed similar results obtained by numerous other investigators. Many of their findings are well known, and, for the purpose of this paper, it does not appear necessary to reproduce the data obtained; however, it is appropriate to provide some general background information regarding the effect of calcium chloride on the various properties of concrete. The following brief resume of the findings reported in the Shideler [*10*] paper provides a summary of this information:

> Bureau of Reclamation tests show that one or two percent calcium chloride accelerates early strength development and this increased strength may persist for several years. Calcium chloride may be especially beneficial for concrete exposed to low or freezing temperatures at early ages. It increases the early heat development, accelerates the set and reduces the initial settlement of concrete. Where this settlement is important, as in backfilling horizontal holes, or under horizontal surfaces, the use of calcium chloride in the concrete can be very beneficial. The data obtained in abrasion and cavitation tests strongly indicate that concretes containing 2 percent calcium chloride are most resistive to erosive forces than plain concrete under either moist curing or drying conditions. At all ages to 90 days the cavitation resistance of the admixed concrete was about 100 percent greater than the plain concrete, the abrasive resistance was from 10 to 25 percent greater. In tests at Grand Coulee with three types of cement and three cement contents, concrete without calcium chloride exhibited about two-thirds the expansion of concrete with calcium chloride in 5 years of storage in water. However, none of the expansions exceeded 0.026 percent and are not considered detrimental. Two percent calcium chloride increased the drying shrinkage about 10 percent for concrete dried for 550 days at 70F and 50 percent relative humidity. Although drying shrinkage is increased, moisture loss is reduced one-sixth by the addition of calcium chloride. Calcium chloride increases the expansion caused by the alkali-aggregate reaction. However, when this reaction is effectively controlled by the use of both low-alkali cement and pozzolan, the effect of the calcium chloride appears to be unimportant. Calcium chloride lowers the resistance of concrete to sulfate attack particularly with type II cement and leaner mixes. The use of air-entrainment counteracted to a large extent the detrimental effect of calcium chloride. Calcium chloride is not detrimental to embedded steel.

Chemical Aspects

Rosenberg [*11*] concluded, after a study of the mechanism through which calcium chloride accelerates the set and strength of portland cement, that the chloride reacts chemically with tricalcium aluminate[3] (C_3A) and tetracalcium aluminoferrite (C_4AF), but this reaction does not appear to be the cause of acceleration. The sulfate reacts first with the aluminates, and then the chloride reacts with the remaining aluminates together with the aluminate ions produced from C_4AF. He also concluded that calcium chloride does not react with tricalcium silicate (C_3S) even though the

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[3] Abbreviations used in cement chemistry are: C = calcium oxide, s = silica, and A = aluminum oxide.

compressive strength at the end of 24 h is greatly increased when 2 percent of calcium chloride by weight of cement is added. Electron microscope studies were made of the hydration of portland cement with and without calcium chloride and a fibrous growth seemed to be evident when calcium chloride was present which could be interpreted as evidence for a rapid crystallization of the resulting silicate hydration products. In the case of the calcium chloride, a larger amount of hydration products are formed at a faster rate or the high ionic character of the calcium chloride solution at the time when silicates are hydrating causes the silicate to crystallize out rapidly in small crystals.

These findings by Rosenberg [*11*] confirm the previously reported conclusion by Rapp [*12*] that the effect of calcium chloride in increasing early strength is largely an effect on the calcium silicates. In their paper, Skalny and Maycock [*13*] have included a review of the foregoing but also have reported on studies of the effect of calcium chloride upon beta-dicalcium silicate (C_2S) hydration. They reported that the hydration of beta-C_2S, when followed by calorimetric techniques, is similar in form to that observed for C_3S with the differences being in the amounts of heat evolved and the times at which heat peaks are evolved. They also reported on the results of studies of the effect of calcium chloride upon beta-C_2S hydration by pH measurements, transmission electron microscopy and dynamic differential calorimetry which showed (*a*) no calcium oxychlorides are formed; (*b*) the degree of hydration is a strong function of the calcium chloride content; and (*c*) more than 2 percent calcium chloride results in a strength decrease.

Kalousek, Jumper, and Tregoning [*14*] reported that the chloroluminates do not form until after the exhaustion of the sulfate, and, in accordance with this, they found little removal of chloride from solution during the first 2 h, however, the sulfo-aluminate reaction appeared to be greatly accelerated, as indicated by the amount of sulfur trioxide (SO_3) in solution.

As reported previously, it has been known for many years that portland cement reacts chemically with a chloride solution and takes much of the chloride out of solution; however, a small amount does remain in solution even after 28 days, as indicated by the data in Table 1 which were reported by Monfore and Verbeck [*15*]. These data are the results of tests on water-cement systems containing various quantities of calcium chloride. The slurries, which consisted of three parts water, one part Type I cement (12 percent calculated C_3A), and various percentages of calcium chloride by weight of cement, were agitated continuously for various periods up to and including 28 days. At the end of each period, the quantities of calcium chloride remaining in solution were determined by chemical analysis. Examination of Table 1 will show that the amount of calcium chloride by weight of cement remaining in solution after 28 days

TABLE 1—*Chlorides in water-cement slurries (percent $CaCl_2$ by weight of cement, in solution after the following number of days).*

0[a]	1	2	3	7	28[b]
0.25	0.12	0.08	0.04	0.03	0.03
0.50	0.26	0.16	0.12	0.08	0.07
1.00	0.51	0.34	0.25	0.17	0.15
2.00	1.13	0.77	0.61	0.41	0.37

$CaCl_2$, Percent Added	$CaCl_2$, Percent at 28 Days	Equivalent Cl^-, lb/yd^3	
		658 lb Cement	564 lb Cement
0.25	0.03	0.13	0.11
0.50	0.07	0.29	0.25
1.00	0.15	0.63	0.54
2.00	0.37	1.56	1.33

NOTE—Data of top part of table from Monfore and Verbeck [*15*].

[a] Amounts initially added at 0 days.

[b] See below for calculations converting percent $CaCl_2$ by weight of cement to pounds Cl^- per cubic yard of concrete in connection with the suggested corrosion threshold limits quoted on p. 4 from Clear [*9*].

varied from 0.03 percent when 0.25 percent was added to 0.37 percent when 2.00 percent was added.

Iron does not exist in nature as a free metal, but, in the oxidized form and after it is refined to the metallic state, it has a strong tendency to rust or reoxidize; however, it is well known that, in normal reinforced concrete, this does not usually occur to any significant extent. The reason for this, as explained by Steinour [*16*], is that the cement matrix of the concrete provides excellent protection from rusting because the free water in it is highly basic or, in other words, has a high pH of about 13. This is due to the calcium hydroxide produced by the reaction of cement with water and to the alkalies of the cement which become hydroxides in solution. This basic solution also contains oxygen dissolved from the air. In such a solution, a thin, tightly adherent protective film forms over the surface of iron or steel. Unless it is impaired in some way, this film prevents corrosion and is said to render the metal passive. The thin film is normally invisible, and its exact composition has been difficult to establish, but it seems definite that it represents chemical combination with oxygen. The more basic the solution, the more highly insoluble the oxide film, which explains the significance of the high pH.

A recent paper by Verbeck [*17*] confirms the significance of this oxide film. He points out that, even though a potentiality for electrochemical corrosion might exist because of nonuniformity of the steel in the concrete, the corrosion is normally prevented by a "passivating" iron oxide film which rapidly forms on the steel surface in the presence of moisture, oxygen, and the water-soluble alkaline products formed during the hydration of the cement. The principal product involved is calcium hydroxide, and he explains that the initial alkalinity of the concrete is at least that of saturated lime water which is a pH of about 12.4, depending upon the temperature. There are two general mechanisms by which this highly alkaline environment and accompanying passivating effect may be destroyed: (*a*) reduction of the alkalinity by leaching of alkaline substances or partial neutralization by reaction with carbon dioxide and (*b*) electrochemical action involving chloride ions in the presence of oxygen.

Mozer, Bianchini, and Kesler [*18*] pointed out that the influence of chloride ions is not as readily predictable as the influence of pH, but it is well to know that the presence of salts may be expected to increase the corrosion rate if only because it is conducive to ionic activity. However, the presence of salts provide two opposing effects: (*a*) it increases the conductivity of the electrolyte, thus raising the corrosion rate, and (*b*) at high concentrations, it diminishes the solubility of oxygen and thereby may lower the corrosion rate. Thus the influence of salts depends chiefly on their concentrations.

Corrosive Factors

The passivating film mentioned previously is protective enough to prevent any significant corrosion from ocurring. Even with an imperfect film, significant corrosion will not occur unless oxygen has access to the reinforcing steel. Concrete has access to oxygen from the air but it reaches the reinforcement only through the covering layer of concrete. As already stated, corrosion can occur only through a deficiency in the protective film in addition to oxygen. The high basicity of the hydrated cement matrix is what assures a good film so a reduction of pH can result in deterioration or destruction of the film. Under normal conditions, concrete is not exposed to acid attack as such, but it is exposed to carbon dioxide in the air which, with moisture, forms carbonic acid, a weak acid, but still capable of reacting with calcium hydroxide to form calcium carbonate which has a tendency to lower the basicity or pH. This reaction which is usually referred to as carbonation does not penetrate very far into portland cement concrete. According to studies made by Gille [*9*], good reinforced concrete of normal water-cement ratio is unlikely to show significant carbonation beyond a depth of ½ in. even though exposed to the weather for many years. Hamada [*19*] reported on the results of a comprehensive series of laboratory tests and on extensive field studies. His conclusions with regard to depth of carbonation were generally the same as those reported by Gille [*9*]. However, excessive water-cement ratio, significant cracking, thin cover of concrete over steel, or poor consolidation will facilitate access of air and can result in reduction of basicity at places along the reinforcement, and this, in turn, can lead to corrosion.

Plain Reinforced Concrete

The action of calcium chloride in plain reinforced concrete has been discussed, and a rather detailed summary of the findings of the extensive test program conducted by the U.S. Bureau of Reclamation has been reviewed. Their findings are in agreement with those obtained and reported by many other investigators. It is generally agreed (Uhlig [*20*], Halstead [*21*], Blenkinsop [*22*], Hill [*23*], Tomek and Vavrin [*24*], and the Reunion Internationale des Laboratoires d' Essais et de Recherches sur les Materiaux et les Constructions 12-Corrosion of Reinforced Concrete Committee [*25*]) that, when the amount of commercial calcium chloride is limited to 2 percent by weight of cement, the degree of corrosion in ordinary reinforced concrete is insignificant if the concrete is reasonably dense and there is adequate cover over the steel.

Information has been presented regarding the reaction of calcium chloride with cement, the potential reaction with steel, the benefits that can be obtained, and the precautions that should be taken when it is used in reinforced concrete. The following information has been abstracted from the indicated American Concrete Institute (ACI) documents and represents the consensus of members of the Institute regarding the use of calcium chloride as an accelerator in concrete.

ACI Committee 212 on Admixtures has published a guide [*26*] for use of admixtures in concrete. Paragraph 4.7 states, "Usually the recommended maximum dosage of calcium chloride is 2% by weight of portland cement. Paragraph 4.3.9 states, "The use of calcium chloride in recommended amounts does not cause progressive corrosion of conventional steel reinforcement in typical reinforced concrete under normal conditions where the bars have sufficient concrete cover."

A more complete discussion of the use of calcium chloride is contained in the ACI Standard Recommended Practice for Cold Weather Concreting (ACI 306-66). In this document, the use of calcium chloride is recommended in connection with the placement of concrete under cold weather conditions except for the following special situations:

1. Prestressed concrete.
2. Moist concrete which may be exposed to stray dc currents.
3. Concrete in direct contact with galvanized steel used as permanent forms for roof decks.
4. Concrete which contains two or more embedded items such as steel reinforcing and aluminum conduit in direct contact and exposed to moisture conditions.

5. Where the concrete is exposed to moist conditions and potential attack by sulfates.

Particular note should be taken of Section 6.1 of ACI 306-66 which is quoted in part as follows: "If proper precautions are followed, accelerating admixtures, high early strength cement (Type III), or additional cement, may be used to develop the level of strength required in a shorter period. This often results in a savings due to shorter duration of protection, faster reuse of forms, easier removal of shores, or less labor in finishing flatwork."

It might also be added that such use will reduce the time the concrete must be heated in subfreezing temperatures which is highly desirable from the standpoints of both economics and energy conservation.

In Paragraph 3.6.1 of the ACI Building Code Requirements for Reinforced Concrete (ACI 318-71) the statement is made that: "Admixtures containing chloride ions shall not be used in prestressed concrete or concrete containing aluminum embedments if their use will produce a deleterious concentration of chloride ion in the mixing water." This precaution is also noted in Paragraph 3.6.1 of the Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-71) where it states that: "Attention is called to the possible adverse effects of excessive chloride ions in the presence of aluminum, and in prestressed concrete. Admixtures containing any chloride other than that which may be contributed as impurities from admixture ingredients should not be used in prestressed concrete or in concrete which will have aluminum embedments."

Because of this justifiable concern regarding the use of admixtures containing chloride in prestressed concrete, there have been instances which have resulted in an impossible specification. For example, Paragraph CB-2224.1 of the ACI-American Society of Mechanical Engineers (ASME) C 359-74 "Code for Concrete Reactor Vessels and Containments" contained the statement: "Admixtures containing chloride ions shall not be used." This specification is impossible to meet because almost all commercial grade chemical products contain traces of chloride ion, and even municipal water may contain as much as 280 ppm of chloride ion [27]. Consequently, it is infeasible from the standpoint of practical industrial technology to produce an admixture that does not contain a trace of chloride ion. This was recognized by both ACI and ASME. The *Winter 1975 Addenda* to ACI-ASME 359-74 contains an "Errata" which changes the previously quoted statement to read: "Admixtures containing more than 1% chloride ions shall not be used."

As a final comment, it would appear appropriate to again make the point that problems with chloride in bridge decks and prestressed concrete do not justify highly restrictive specifications and requirements regarding its use in ordinary reinforced concrete.

References

[*1*] Millar, W. and Nichols, C. F., "Improvements in Means of Accelerating the Setting and Hardening of Cements," London, England, Patent 2886, 4 March 1885.

[*2*] *Proceedings,* American Society for Testing and Materials, 1923, Vol. 23, Part 1, p. 223.

[*3*] Mattimore, H. S., *Proceedings,* American Society for Testing and Materials, Vol. 23, 1923, p. 328.

[*4*] Pearson, J. C., *Proceedings,* American Society for Testing and Materials, Vol. 23, 1923, p. 328.

[*5*] Cottringer and Kendall, *Concrete Magazine,* Vol. 4, 1923, pp. 150–55.

[*6*] "Calcium Chloride in Concrete," Highway Research Board Bibliography 13, annotated.

[*7*] Clear, K. C., "Evaluation of Portland Cement Concrete for Permanent Bridge Deck Repair," Report No. FHWA-RD-74-5, Federal Highway Administration, 1974.

[*8*] Clear, K. C. and Hay, R. F., "Time-To-Corrosion of Reinforcing Steel in Concrete Slabs," Report No. FHWA-RD-73-32, Vol. 1, Federal Highway Administration, 1973.

[*9*] Shideler, J. J., *Journal of American Concrete Institute,* March 1952, p. 537.

[*10*] Rosenberg, A. M., *Journal of American Concrete Institute,* Oct. 1964, pp. 1261-1268.

[*11*] Rapp, P., *Proceedings,* Highway Research Board, Vol. 14, 1934, pp. 341-381.

[*12*] Skalny, J. and Maycock, J. N., *Journal of Testing and Evaluation,* American Society for Testing and Materials, Vol. 3, No. 4, 1975, pp. 303-311.

[*13*] Kalousek, G. L., Jumper, C. H., and Tregoning, J. J., *Journal Research National Bureau Standards,* Vol. 30, 1943, pp. 215-255.

[*14*] Monfore, G. E. and Verbeck, G. J., *Journal of American Concrete Institute,* Nov. 1960, pp. 491-515.

[*15*] Steinour, H. H., "Influence of the Cement on Corrosion Behavior of Steel in Concrete," Research Dept. Bulletin 168, Portland Cement Association, 1964.

[*16*] Verbeck, G. J., Publication SP-49, American Concrete Institute, 1975, pp. 21-38.

[*17*] Mozer, J. D., Bianchini, A. C., and Kesler, C. E., *Journal of American Concrete Institute,* Aug. 1965, pp. 909-930.

[*18*] Gille, F., *Beton Herstellung-Verwendung,* Vol. 10, 1960, pp. 328-330.

[*19*] Hamada, M., *Proceedings,* Fifth International Symposium on the Chemistry of Cement, Vol. III, 1968, pp. 343-368.

[*20*] Uhlig, H. H., *Corrosion Handbook,* Wiley, 1948, p. 125.

[*21*] Halstead, P. E., "The Corrosion of Metals in Contact with Concrete," *Chemistry and Industry,* London, Aug. 24, 1957.

[*22*] Blenkinsop, J. D., *Magazine of Concrete Research,* Vol. 15, March 1963, pp. 33-38.

[*23*] Hill, A. W., "Corrosion of Steel in Precast Concrete Containing Calcium Chloride Admixtures," *4th International Congress of the Precast Concrete Industry,* Paris, 1963, reprint.

[*24*] Tamek, J. and Vavrin, F., *Zement-Kolk-Gips,* Vol. 14, 1961, pp. 108-112.

[*25*] "Corrosion of Reinforcement and Prestressing Tendons," a state-of-the-art report, *Materiaux Et Constructions,* Vol. 9, No. 51, May-June 1976, pp. 187-206.

[*26*] *Guide for Use of Admixtures in Concrete,* Title No. 68-56, American Concrete Institute Committee 212, 1971.

[*27*] McCoy, W. J. in *Symposium on Significance of Tests of Concrete,* 2nd ed., *ASTM STP 169 A,* American Society for Testing and Materials, 1966, p. 517.

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Marine Corrosion Behavior of Bare and Metallic-Coated Steel Reinforcing Rods in Concrete

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ABSTRACT: Concrete failure can develop from many causes, for example, thermal gradients, fatigue, insufficient cover, admixtures, rebar material, and exposure location. Cracking of concrete in itself was not found to be detrimental but could lead to spalling and structural deterioration. Metallic-coated rebars are shown to prevent concrete spalling.

KEY WORDS: corrosion, concretes, reinforcing steels, cracking, spalling, marine atmospheres, nickel coatings, zinc coatings, low alloy steels, high strength steels, electrochemistry

The corrosion behavior and material performance of several reinforcing rod (rebar) materials have been evaluated in an eleven-year exposure program in marine environments at International Nickel's Francis L. LaQue Corrosion Laboratory (FLLCL).

The purpose of the investigation was to determine if low-alloy or nickel-coated steel had any advantage over bare or zinc-coated steel when used for concrete reinforcement in marine environments.

Concrete failure can develop from many causes, for example, thermal gradients, fatigue, insufficient cover, admixtures, rebar material, and exposure location. Cracking of concrete in itself was not found to be detrimental but could lead to spalling and structural deterioration. Nickel- and zinc-coated rebars are shown to prevent concrete spalling.

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The rebar materials tested were ASTM Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement (A 615-76) steel, a high-strength, low-alloy Ni-Cu-Cr steel, AISI 4340 steel, nickel-coated ASTM A 615 steel, and zinc-coated ASTM A 615 steel. Each rebar (No. 6 deformed rod) was cast in concrete at depths of ½ in. and 1½ in. from the surface. The reinforced concrete specimens were exposed to three different marine environments: (*a*) seawater tidal immersion where specimens were subjected to alternating seawater and air exposure, (*b*) seawater splash and spray which was comparable to a bridge deck or other structures above high tide, and (*c*) marine atmosphere 80 ft from the ocean.

The results describe and compare the corrosion performance of bare and coated rebars and their influence on concrete failure. Failure is associated with either cracking or spalling of the concrete which is related to the degree of corrosion on the rebar. Several factors which influenced the degree of corrosion were reinforcing rod material, exposure location, concrete cover, concrete mixture, and type and grade of cement.

Reinforcing rods in concrete are often attacked due to intrusion through the concrete of corrosive elements from the environment. Once this occurred, it was found that the concrete with metallic-coated rebars performed substantially better than concrete with bare steel rebars.

Background

For many years, there has been an awareness of the problem of bridge deck deterioration involving reinforcing rod corrosion and cracking and spalling of the concrete. In the majority of cases, steel embedded in concrete remains free of corrosion for a very long time, but there are numerous cases where corrosion of reinforcement has been rapid and caused extensive damage. Two such widely studied highway failures are the San Mateo-Hayward Bridge and the Passiac River Bridge.

The San Mateo-Hayward Bridge across San Francisco Bay opened in 1929, and, after seven years, widespread cracking had developed. The structure was exposed to a severe marine environment which penetrated the concrete and caused corrosion product buildup on the reinforcing rods. After 37 years, 47 percent (8 out of 17) of the piling inspected displayed heavy corrosion of steel reinforcing rod, and broad pits 0.017 to 0.26 in. deep (average: 0.114 in.) were found [*1*].[3] This report stated that factors contributing to the distress included water-cement ratios as high as 7.5 gal/sack and the resultant absorption of substantial quantities of chlorides.

[3] The italic numbers in brackets refer to the list of references appended to this paper.

The Passiac River Bridge at the northern end of the New Jersey Turnpike (five miles from New York City) was constructed in 1952 and showed signs of deterioration in 1955. Deterioration was caused by penetration of deicing salts that caused rust to generate on the reinforcing steel, resulting in cracked and spalled concrete. These earliest failures occurred at points where the rebars were found to have been covered by only ½ in. of concrete [*2*].

There have also been recent indications that bridges in the interstate highway system designed to last 40 years are requiring major repair after only 10 years of use, and that quite expensive solutions are now under consideration by federal and state authorities. The following background information was released at the first meeting of the Department of Transportation Materials Study Group in 1973 at which the staff of the Fairbank Highway Research Station of the Federal Highway Administration presented an outline of the needs for materials research to solve highway problems:

1. Bridge deck deterioration is one of the biggest problems in the highway field.
2. It is costing the taxpayer $70 million per year, and the Federal Government is becoming extremely concerned about cost of maintenance as well as first cost of the federal interstate and intrastate systems. Overall maintenance costs are now equal annually to new construction costs.
3. Deterioration is caused by penetration of deicing salts that cause rust to generate on the reinforcing steel, and the consequent pressure spalls the concrete.

It is well known that the corrosion product of steel occupies 2.2 times as much space as the metal and may develop mechanical pressures as high as 4700 psi which is many times greater than the tensile strength of some concrete [*3*].

These were some of the primary reasons that stimulated The International Nickel Company, Inc. to investigate the behavior of coated and bare steel as reinforcing rod material.

Materials and Procedure

This program was designed to evaluate the performance of reinforcement bars in concrete when exposed in marine environments. A large number of concrete castings were prepared and then exposed in marine environments at the FLLCL of The International Nickel Company, Inc. The reinforcement materials were carbon steel, two low-alloy, high-strength steels, and steel coated with either nickel or zinc. Table 1 identifies the composition of the alloys used and the coating thicknesses applied.

TABLE 1—*Composition of rebar steels.*

Steel	Chemical Composition, percent by weight								
	C	Mn	P	S	Si	Cu	Ni	Cr	Mo
ASTM A-615[a]	0.15 to 0.25	0.30 to 0.60	0.040	0.04	0.20 to 1.00	. . .	. . .	. . .	. . .
Low-alloy, high-strength	0.092	0.44	0.088	0.025	0.52	0.30	0.28	0.85	<0.005
AISI 4340	0.39	0.79	0.010	0.017	0.23	0.07	1.66	0.73	0.25

NOTES—1. All reinforcing rods were ¾ in. diameter, No. 6 deformed.

2. ASTM A-615 intermediate grade specification steel was used as the substrate material for preparation of the nickel and zinc coated bars.

3. Rolled on nickel-coated steel; coating thicknesses were 0.025 mm (1 mil) and 0.076 mm (3 mils).

4. Zinc-coated steel; coating thickness was 0.2 mm (8 mils).

[a] Nominal composition.

Preparation of Nickel-Coated Reinforcing Bars

The nickel-coated rebars were produced from commercially nickel-plated ASTM A 615, 4 by 4-in. steel billets. Sulfur-free pure nickel was electrodeposited from a Watts-type nickel bath onto the descaled and cleaned billets. At a commercial rebar mill, the nickel-plated billets were heated to 1900 to 2000°F in an oxidizing atmosphere in a pusher type furnace, fueled with a low-sulfur #2 fuel oil, and hot rolled to size.

Preparation of Zinc-Coated Reinforcing Bars

Produced from the same type ASTM A 615 carbon steel billets, commercial hot-rolled No. 6 rebars were provided to a commercial galvanizing shop. The rebar was hot dip galvanized to a zinc thickness of approximately 8 mils, about twice the coating thickness specified.

Concrete and Coverage

The cement used in the concrete castings was either Type I or Type II and both met ASTM standard specifications for portland cement. Type I cement is used in general concrete construction when the special properties specified for the other cement types are not required. Type II cement is specified for use in general concrete construction exposed to moderate sulfate action or where moderate heat of hydration is required. At the time of the program's initiation, the concrete used was classified as either "good" (7.4 gal of water per sack) or "poor" (8.0 gal of water per sack) quality. Most of the available literature indicates that the water-cement ratio of bridge deck concrete should be reduced as close to 0.40 by weight (4.5 gal/sack) as feasible. By these current standards, both of the concrete formulations used in this program would have to be classified as "poor." The cement content was 5.3 sacks/yd^3. The effect of water-cement ratio on water permeability of concrete has been widely documented [*4*]. This ratio appears to be the primary determinant of the ability of cement concrete to resist chloride intrusion. It has been shown that portland cement concrete with a water-cement ratio of 0.50, cement factor of 7.0 bags/yd^3, and 45 volume percent sand in the total aggregate, had a time to corrosion with 1-in. rebar cover of only 1 week [*5*]. The electrical half-cell potential of the reinforcing steel was monitored versus time after the test slabs were subjected to ponding with a 3 percent sodium chloride solution. Criteria used to evaluate the chloride migration were potentials of the rebar consistently greater than 0.35-V copper sulfate electrode (CSE), which indicated high probability of rebar corrosion. Concrete castings were also made with or without admixtures for fluidity or air entrainment.

The rebar materials were cast into concrete blocks 6 by 6 by 24 in. with either ½ or 1 ½ in. of concrete cover over the rebar. The ½ in. coverage was chosen to represent an incorrectly placed or shifted bar. The 1 ½ in. coverage was considered to be the typical concrete coverage used in the construction industry during the planning stages of this study. Four rebars were cast into each block. A series of 14-ft-long castings were made with several of the rebar materials and were used as the supports for the exposure of the shorter castings. This caused the 14 ft by 6 in. by 7 in. beams to have a dead weight load applied for the duration of the 11-year test program. Each of the 14-ft beams had one reinforcing rod located with 4 ½ in. top coverage and 1 ½ in. bottom concrete coverage. All specimens were prepared by the Civil Engineering Department of North Carolina State University.

Exposure Locations

The three marine environment exposure locations used to evaluate the performance of these rebar materials were as follows:

Splash-and-Spray Zone Deck—A special static test deck was constructed approximately 5 ft above mean high tide. This marine splash-and-spray exposure approximates that of a bridge deck in a marine environment.

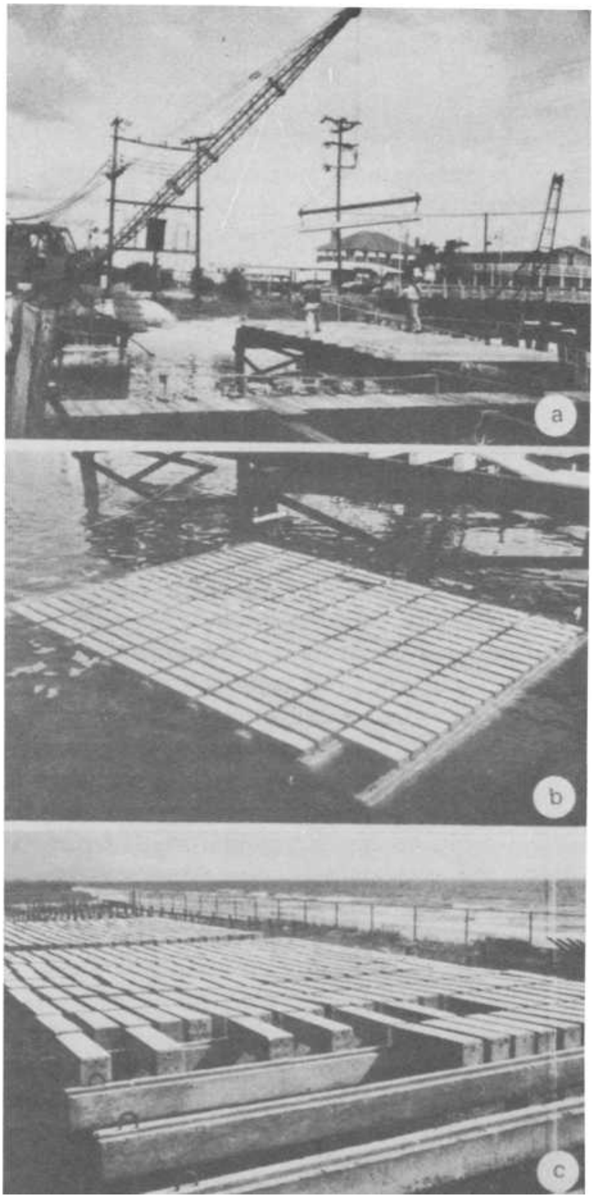
Seawater Tidal Zone—This exposure location allowed the test specimens to be alternately wet and dry twice each day with the tide changes. Results from this location could be applied to reinforced piling or structures in seawater.

80-ft Marine Atmosphere Lot—A series of specimens was exposed on an elevated platform approximately 80 ft from mean high tide. Results from this location could be applied to reinforced structures near shoreline or in severe marine atmospheres.

Specimens were exposed in the various marine environments in August 1961 (Fig. 1).

Evaluation Techniques

Before any discussion of results, it is necessary to define the criteria used in the evaluation of the results of this program. The first criterion of concrete failure used was the onset of cracking. Cracking was defined as visible fissures occurring in the concrete with varying degrees of severity from very slight to severe. Cracking in concrete bridge decks may be caused by several different factors. It has generally been observed that transverse cracks occur directly over top transverse reinforcing steel. While cracking is often associated with the more serious type of deterioration known as spalling, it does not appear to affect the incidence of spal-



(*a*) Exposure operation, FLLCL splash-and-spray deck.
(*b*) Exposure, FLLCL seawater tidal immersion.
(*c*) Exposure, Kure Beach, 80-ft lot.

FIG. 1—*Reinforced concrete marine exposure.*

ling. If chlorides can be detected at the rebar surface after only one week of exposure to the chloride-laden environment, it is apparent that the presence or absence of a crack would have little relative effect on the incidence of corrosion. Photographic standards were formulated for use as guides for cracking evaluations. These standards are shown in Fig. 2.

It is surmised that microcracking eventually led to spalling of the concrete which, in its worst condition, actually exposes the rebar directly to the environment. Severe spalling can be characterized by the inevitable pothole frequently encountered on our bridges and highways. The photographic standards used to depict degrees of spalling are shown in Fig. 3.

Initial observations were made as to the type of concrete damage and severity, and, subsequently, selected castings were broken open and the rebar removed. The rebar was then examined for severity of corrosion, and, if present, its relationship to the cracking or spalling initially noted was recorded. As each rebar was examined, estimates were made as to the amount of corrosion product present. Figure 4 illustrates the visual standard used to determine the percent rust coverage of the rebar examined. Equal numbers of concrete specimens, each containing four rebars, were taken from each exposure site. The specimens examined contained all planned combinations of variables for comparisons of rebar performance.

Results

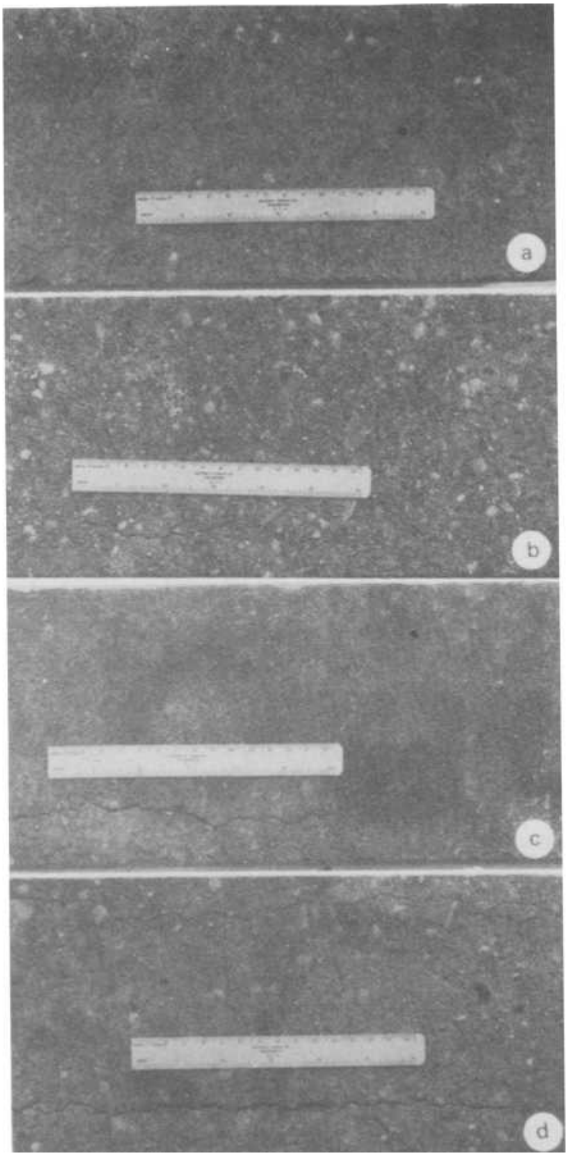
Tables 2, 3, and 4 summarize the performance of the reinforcing bars in concrete after eleven years of exposure in the various marine environments. The effects of steel type on concrete failure are shown in Fig. 5.

The results show that metallic-coated rebars are definitely beneficial compared to carbon or low-alloy steels with regard to the overall performance of reinforced structures in marine environments. They also show that a 1-mil nickel coating is sufficient to achieve improved reinforced concrete performance.

This program did not reveal any particular advantage for the use of a low-alloy steel rebar over carbon steel reinforcing bars.

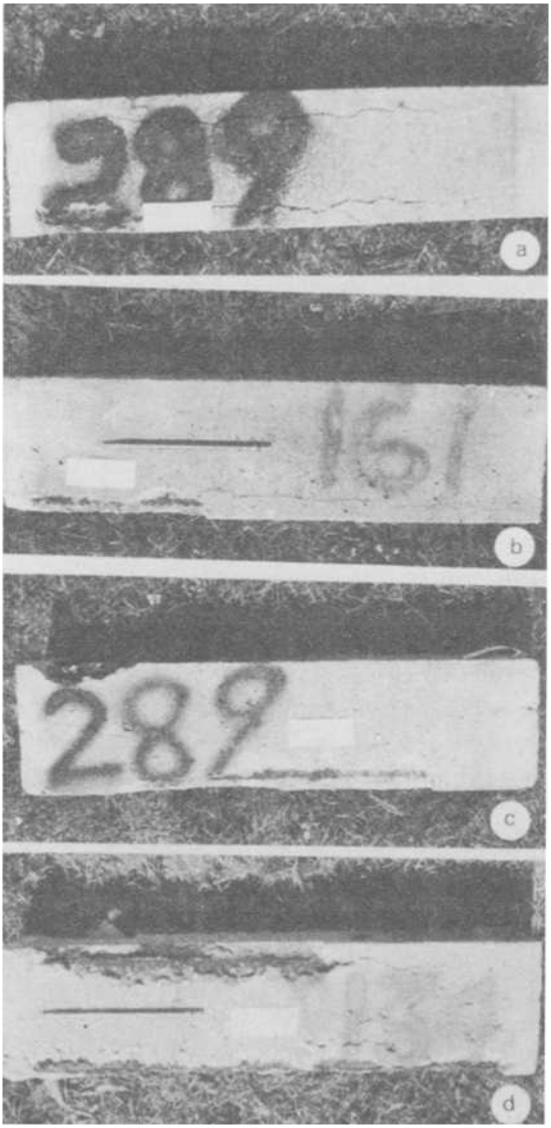
Discussion

Periodically throughout the eleven-year exposure program, *in situ* examinations were made of the castings for initiation of cracking and resultant spalling. The majority of the concrete failures which occurred were observed during the first four years of exposure. These mainly involved specimens with ½ in. concrete coverage over the reinforcing rods. The most extensive examination, which culminated in the destruction of the castings and removal of the test rebars, was conducted at the end of eleven years of exposure. From these examinations, the performance of



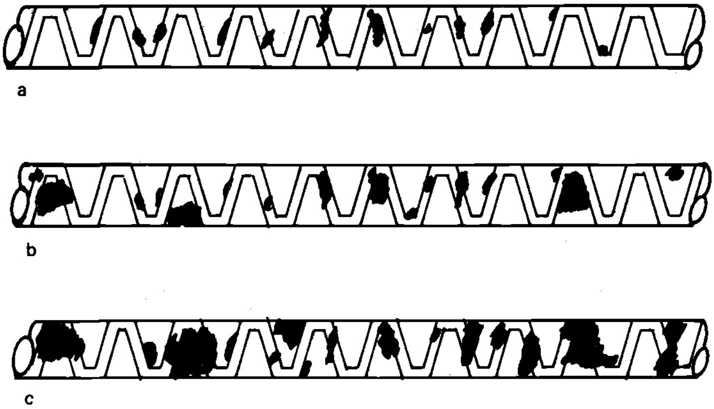
(*a*) Very slight cracking.
(*b*) Slight cracking.
(*c*) Moderate cracking.
(*d*) Severe cracking.

FIG. 2—*Photographic standards used to evaluate degree of concrete cracking.*



(*a*) Very slight spalling.
(*b*) Slight spalling.
(*c*) Moderate spalling.
(*d*) Severe spalling.

FIG. 3 —*Photographic standards used to evaluate degree of concrete spalling.*



(*a*) 5 percent rust coverage.
(*b*) 15 percent rust coverage.
(*c*) 25 percent rust coverage.

FIG. 4—*Estimating rust coverage on reinforcing rods.*

the various reinforcing rod material was determined with regard to the various exposure variables.

For example, Table 2 shows that severe spalling occurred in ASTM A 615 where the rebar rust coverage averaged greater than 50 percent, and, in five instances, ranged from 95 to 100 percent. The degree of rusting or corrosion noted on the coated rebars had a lesser effect on concrete failure. Based on the specimen population, 40 percent of the ASTM A 615 steel concrete castings suffered some degree of cracking which was related to an average rebar rust coverage of greater than 50 percent. The concrete castings with 1-mil nickel-coated rebars suffered only 18 percent cracking which was related to less than 10 percent visible corrosion on the nickel-coated rebars.

About 80 percent of the nickel-coated (1 or 3-mil) and zinc-coated (8-mil) steel reinforcing rods produced no cracks in the concrete with 10 percent or less rust coverage on the rods. In some instances, slight cracks in the concrete were related to rust coverages from 10 to 75 percent, and severe cracks in the concrete were related to 25 to 95 percent rust coverage. There was no instance where the base steel corrosion product volume was enough to cause spalling of the concrete in these systems.

The performance of the bare carbon steel rods was comparable to the low-alloy steel rods; both produced more numerous and more severe concrete failures than the metallic-coated steel reinforcing rod materials. Twenty to fifty percent of the rods were less than 10 percent covered by rust; 60 percent of these rods exhibited no rust related concrete failure. The balance of the rods ranged from 10 to 100 percent coverage by rust,

TABLE 2—*Performance of reinforcing steel in concrete exposed in marine splash-and-spray zone.*

(Concrete Coverage)	Materials											
	3 mil Ni		1 mil Ni		8 mil Zn		ASTM A 615		Ni-Cu-Cr		4340	
Concrete	1/2 in.	1 1/2 in.	1/2 in.	1 1/2 in.	1/2 in.	1 1/2 in.	1/2 in.	1 1/2 in.	1/2 in.	1 1/2 in.	1/2 in.	1 1/2 in.
Type 1 cement, 7.4 gal, with admixture	slight cracks (10 to 25)	no cracks (0)	...	no cracks (0)	...	...	severe spalling (9 to 26)	no cracks (0 to 8)	severe spalling (45 to 60)	no cracks (0 to 2)	severe spalling (40 to 60)	...
Type 1 cement, 8.0 gal, with admixture	slight cracks (20 to 75)	no cracks (1 to 2)	...	very slight cracks (0)	...	...	severe spalling (9 to 95)	slight cracks (2 to 17)	moderate spalling (50 to 100)	no cracks (2 to 40)	severe spalling (60 to 100)	...
Type 1 cement, 7.4 gal, no admixture	moderate cracks (2 to 7)	no cracks (0 to 1)	severe cracks (3 to 25)	very slight (0 to 1)	moderate cracks (0 to 1)	no cracks (0)	severe cracks (10 to 40)	no cracks (0 to 5)	...	very slight cracks (0)	...	...
Type 1 cement, 8.0 gal, no admixture	severe cracks (10 to 25)	...	moderate cracks (2 to 8)	no cracks (0 to 3)	severe cracks (0 to 10)	very slight cracks (0 to 1)	severe spalling (12 to 90)	slight cracks (12 to 50)	severe spalling (5 to 75)	severe cracks (2 to 12)	severe spalling (50 to 80)	...
Type II cement, 7.4 gal, with admixture	severe cracks (40 to 50)	slight cracks (3 to 12)	...	...	...	...	severe spalling (55 to 100)	no cracks (5 to 30)	...	...	...	...
Type II cement, 8.0 gal, with admixture	severe cracks (15 to 95)	...	...	no cracks (1 to 15)	...	...	severe spalling (100)	severe cracks (90 to 100)	...	...	...	...

NOTES—1. The cracking or spalling observation refers to the appearance of the concrete integrity.
2. The numbers in parentheses indicate the range of percent rust coverage on the four rebars removed from the concrete block.

TABLE 3—*Performance of reinforcing steel in concrete exposed in seawater tidal immersion zone.*

(Concrete Coverage)	Materials											
	3 Mil Ni		1 Mil Ni		8 Mil Zn		ASTM A 615		Ni-Cu-Cr		4340	
Concrete	1/2 in.	1 1/2 in.	1/2 in.	1 1/2 in.	1/2 in.	1 1/2 in.	1/2 in.	1 1/2 in.	1/2 in.	1 1/2 in.	1/2 in.	1 1/2 in.
Type 1 cement, 7.4 gal, with admixture	no cracks (1 to 2)	no cracks (1 to 3)	no cracks (2 to 8)	no cracks (1)	. . .	. . .	. . .	no cracks (9 to 14)	no cracks (1 to 2)	no cracks (4 to 10)	. . .	. . .
Type 1 cement, 8.0 gal, with admixture	no cracks (1 to 2)	no cracks (1 to 4)	no cracks (4 to 6)	no cracks (1 to 2)	. . .	. . .	no cracks (1 to 4)	no cracks (1 to 2)	no cracks (4 to 8)	no cracks (3 to 4)	. . .	. . .
Type 1 cement, 7.4 gal, with admixture	no cracks (1 to 3)	no cracks (1 to 4)	no cracks (1 to 13)	no cracks (3 to 6)	no cracks (0)	no cracks (0)	no cracks (1 to 5)	no cracks (6 to 18)	. . .	. . .	. . .	. . .
Type I cement, 8.0 gal, with admixture	no cracks (1)	no cracks (0 to 1)	no cracks (0 to 10)	no cracks (1)	no cracks (0)	no cracks (0)	no cracks (12 to 16)	no cracks (15 to 20)	no cracks (10 to 15)	no cracks (10 to 14)	. . .	. . .
Type II cement, 7.4 gal, with admixture	no cracks (0 to 1)	no cracks (1)	. . .	. . .	. . .	. . .	no cracks (1 to 2)	no cracks (3 to 5)	. . .	. . .	. . .	. . .
Type II cement, 8.0 gal, with admixture	no cracks (2 to 6)	no cracks (0 to 2)	. . .	no cracks (1 to 2)	. . .	. . .	no cracks (5 to 10)	no cracks (2 to 5)	. . .	. . .	. . .	. . .

NOTES—1. The cracking or spalling observation refers to the appearance of the concrete integrity.

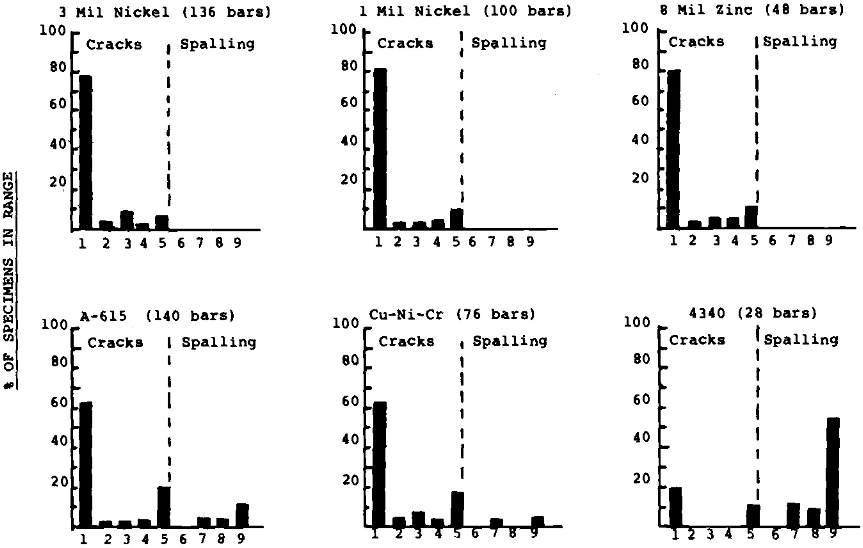
2. The numbers in parentheses indicate the range of percent rust coverage on the four rebars removed from the concrete block.

TABLE 4—*Performance of reinforcing steel in concrete exposed in marine atmosphere, 80 ft from the ocean.*

(Concrete Coverage)	Materials											
	3 mil Ni		1 mil Ni		8 mil Zn		ASTM A 615		Ni-Cu-Cr		4340	
Concrete	1/2 in.	1 1/2 in.	1/2 in.	1 1/2 in.	1/2 in.	1 1/2 in.	1/2 in.	1 1/2 in.	1/2 in.	1 1/2 in.	1/2 in.	1 1/2 in.
Type 1 cement, 7.4 gal, with admixture	severe cracks (6 to 40)	no cracks (1)	severe cracks (25 to 45)	no cracks (0 to 9)	... ...	... ...	moderate spalling (25 to 60)	no cracks (1 to 25)	slight spalling (50 to 75)	no cracks (0 to 2)	severe spalling (10 to 50)	...
Type 1 cement, 8.0 gal, with admixture	severe cracks (40 to 60)	no cracks (0 to 25)	severe cracks (15 to 65)	no cracks (3 to 6)	...	...	severe spalling (75 to 95)	no cracks (0)	severe spalling (85 to 95)	no cracks (5 to 35)	severe spalling (75 to 95)	no cracks (0 to 5)
Type 1 cement, 7.4 gal, no admixture	very slight cracks	no cracks (0 to 13)	severe cracks (10 to 20)	no cracks (0)	severe cracks (1)	no cracks (0)	severe cracks (5 to 25)	no cracks (2 to 25)		no cracks (1 to 10)		
Type 1 cement, 8.0 gal, no admixture	slight cracks (15 to 30)		severe cracks (10 to 25)	no cracks (0 to 3)	severe cracks (1 to 5)	no cracks (0)	severe spalling (60 to 75)	no cracks (0 to 10)	slight spalling (55 to 70)		severe spalling (30 to 80)	
Type II cement, 7.4 gal, with admixture	very slight cracks (5 to 35)	no cracks (0)	...	...	...	...	severe spalling (30 to 90)	no cracks (1 to 10)	...	...	...	...
Type II cement, 8.0 gal, with admixture	moderate cracks (40)	no cracks (0 to 1)	...	no cracks (1 to 4)	...	...	slight spalling (95)	no cracks (30 to 50)	...	...	...	...

NOTES—1. The cracking or spalling observation refers to the appearance of the concrete integrity.

2. The numbers in parentheses indicate the range of percent rust coverage on the four rebars removed from the concrete block.



1—No failure.
2—Very slight crack.
3—Slight crack.
4—Moderate crack.
5—Severe crack.
6—Very slight spalling.
7—Slight spalling.
8—Moderate spalling.
9—Severe spalling.

FIG. 5—*Effect of steel type on concrete failure (concrete failures observed before destructive examination).*

and the voluminous rust corrosion product cracked and spalled the concrete.

From the data in Table 3, the least degree of cracking and spalling is observed to have occurred in the seawater immersion zone. Although no cracking and spalling is evident, it will be noted that the percent range of rust coverage for the ASTM A 615 and Ni-Cu-Cr steel was slightly greater than for the metallic-coated rebars. It is obvious, on the basis of the percent rust coverage, that the rebar corrosion had not advanced significantly enough to have any deleterious effect on the specimens. One possible explanation for this phenomenon will be discussed in the section on thermal gradients.

From Table 4, it is apparent that the ASTM A 615 and low-alloy steel rebar materials caused the greatest degree of concrete cracking or spalling when compared with the specimens containing the metallic-coated rebars. Comparing the overall performance of the specimens exposed in the marine atmosphere 80 ft from the ocean with the other two environments, it is apparent that specimens exposed here had a higher incidence of failure than those in the seawater immersion zone but less than those exposed on the splash-and-spray deck.

Electrochemical Reactions

In normal marine electrochemical reactions, nickel is cathodic to steel while zinc is anodic. This would lead one to conclude that, if a pinhole or perforation occurred in the nickel-coated steel system, the exposed area of steel would be a small anode in relation to the large area of cathodic nickel. Normally, this condition would cause accelerated corrosion of the anode or the steel. There were no voluminous steel corrosion products observed in those instances where the steel substrate was exposed in the nickel-coated steel system. A study conducted by Boyd and Tripler [*6*] concluded that, under the alkaline conditions prevalent in sound concrete, a break in a nickel coating would not be a serious problem. In their studies, nickel-coated reinforcing rods, having file cuts in the nickel coating so as to expose the underlying steel, were exposed in an alkaline sodium chloride solution. In this alkaline solution, little or no corrosion was observed after two months.

In the zinc-coated system, the zinc would be anodic to any exposed steel and would corrode preferentially to the steel. This would cause corrosion of the zinc coating to protect the exposed steel. The Boyd and Tripler study also showed that, in alkaline sodium chloride solution, the zinc-coated specimens suffered rapid corrosion of the zinc coating. According to Pourbaix [*7*], zinc is thermodynamically unstable in the presence of water and tends to dissolve with the evolution of hydrogen in very alkaline solutions.

While work by Griffin [*8*] was inconclusive, he did state that the results of his work combined with that of Woods [*9*] tend to indicate the combined forces of expansive corrosion products and pressure of hydrogen gas developed in zinc-coated systems were sufficient to crack the concrete used in his work. Based on his experiments, he concluded that "at best, galvanized steel reinforcement is no better than non-galvanized steel reinforcement; further, it probably is not as good insofar as corrosion is concerned."

Thermal Gradient

Longitudinal cracks that paralleled the reinforcing rods were more numerous on the splash-and-spray deck platform than on the atmospheric platform 80 ft from the ocean. The skyward side of the concrete block specimens was more prone to failure than the groundward side. Failures were almost nonexistent on the lower platform where the specimens were exposed in the seawater tidal range. In the three marine exposure sites, three different ambient thermal conditions prevailed. Analysis of the tabular data (see Tables 3, 4) shows that the absence of cracking and spalling in the specimens from the seawater tidal zone could have been a result of the smaller thermal gradient in this environment. Unpublished work from Kure Beach indicates that the surface temperature of specimens can be as high as 52 °C in the summer months and as low as − 11 °C in the winter. These same temperatures could be expected on specimens exposed in the seawater splash-and-spray zone. Seawater temperatures at FLLCL range from a high of 27 °C in the summer to a low of 4 °C in the winter. The ranking of thermal gradients for the exposure sites were (*a*) the lowest gradient existed in the seawater exposure which had a moderation effect, reducing the thermal expansion and contraction that occurred, (*b*) the atmospheric site specimens experienced some slight thermal gradient effects, and (*c*) the splash-and-spray deck specimens, which suffered the most incidence of failure, suffered the largest thermal gradient.

It is possible that thermal expansion has as great an effect on the performance of reinforced concrete as rebar corrosion has in some environments.

14-ft Support Beams

The 14-ft-long support beams containing a single nickel-coated steel, zinc-coated steel, ASTM A 615 steel, or a low-alloy, high-strength reinforcing rod were examined after eleven years' exposure under the applied stress of the smaller castings. All castings were made with Type I concrete with 7.4 gal of water per sack. The concrete coverage was 1 ½ in. minimum.

Circumferential cracks were observed in all beams from all exposure locations, at first extending across the bottom side and later extending up the sides and across the top of the beams (Fig. 6). Areas with circumferential cracks near the center of the beams were selected for destructive examination. In most cases, the single reinforcing rod displayed localized corrosion at the cracks. This cracking occurred in beams used as support for smaller castings as well as those exposed without any applied load.

One beam containing the bare ASTM A 615 reinforcing bar was found to have an 8-ft longitudinal crack following the rebar. There was no evidence of longitudinal cracking in any of the 14-ft beams reinforced with the coated bars.

Metallographic Examination of Selected Coated Rebars

Several coated rebars were selected for metallographic examination to determine the loss of coating thickness. Photomicrographs of typical longitudinal cross sections of nickel- and zinc-coated rebars are shown in

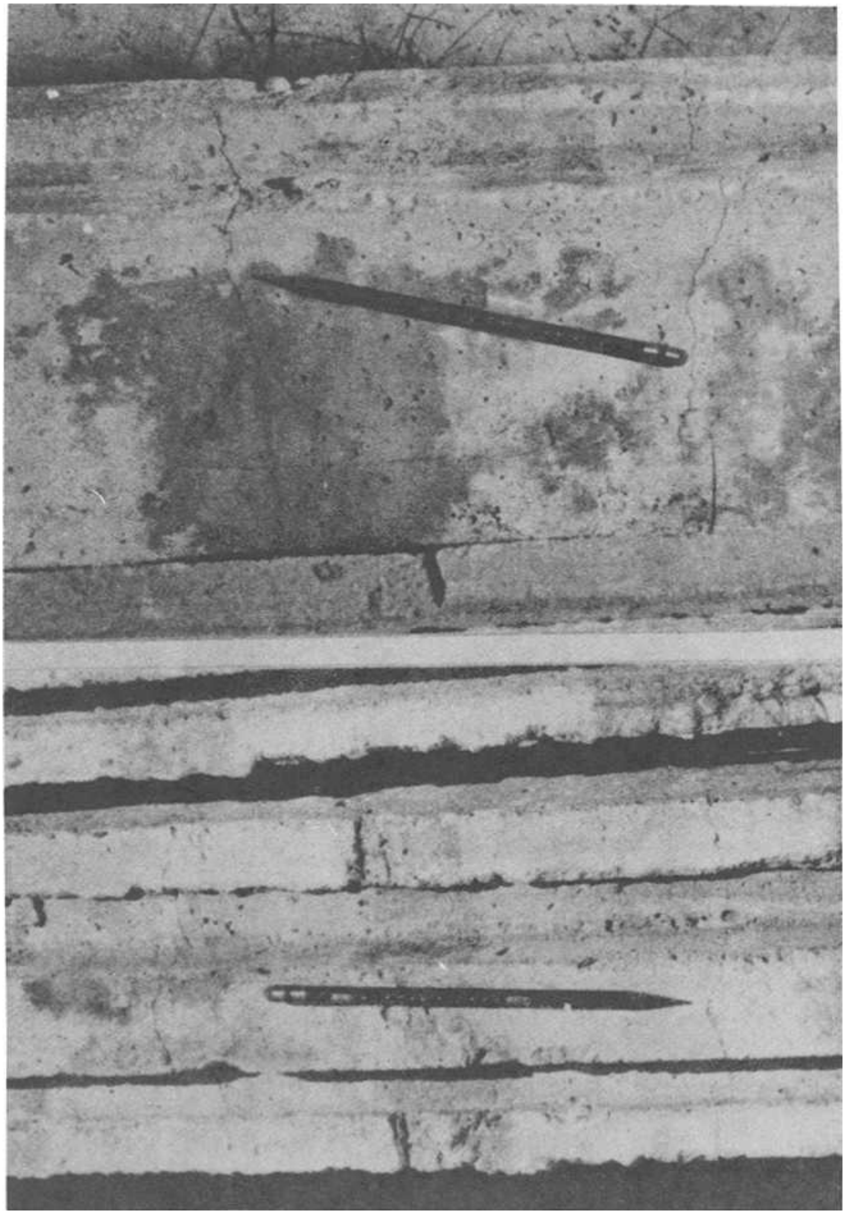
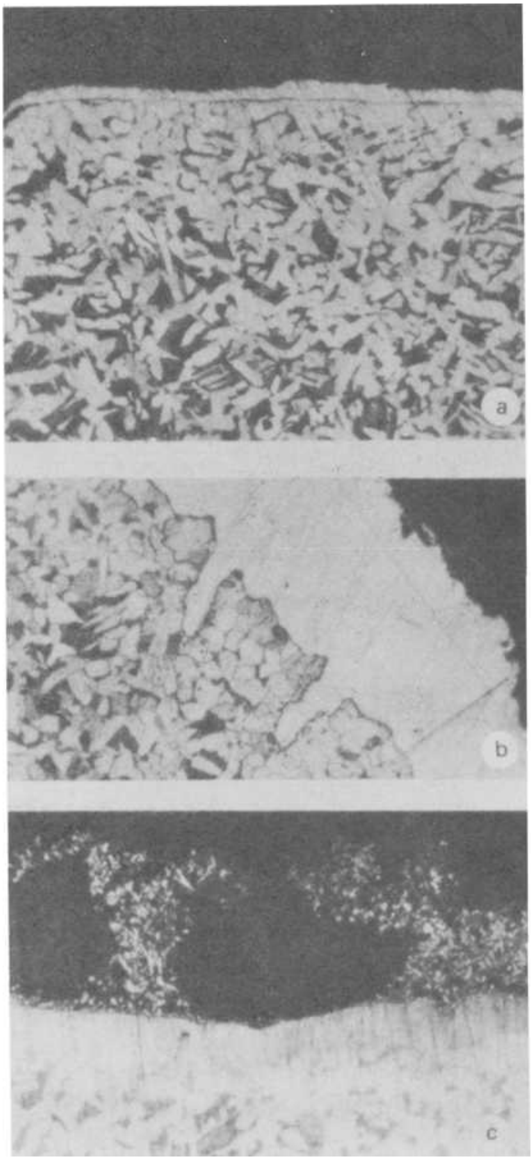


FIG. 6—*Typical circumferential cracks in 14-ft reinforced support beams from the splash-and-spray zone deck.*

Fig. 7. Figure 7*a* illustrates a typical cross section from a 1-mil nickel-coated rebar. It shows the coating to be intact, with no sign of visible deterioration, corrosion, or thinning. Photomicrographs of the 3-mil



(*a*) 1 mil nickel.
(*b*) 3 mil nickel.
(*c*) Zinc coating.

FIG. 7—*Typical cross sections of coated rebar materials* (×~153).

nickel coating also illustrate the excellent condition of the coating after eleven years of exposure. It is apparent from the results of this study that a nickel coating of 1 mil thickness is sufficient to improve the performance of reinforced concrete.

The zinc coating depicted in Fig. 7*c* shows the coating thickness to be approximately 5 to 6 mils (a loss of approximately 2 to 3 mils of the original thickness).

Metallographic examinations revealed that the bare steel rebars do suffer severe corrosion due to penetration of the chloride ion to the rebar surface. The nickel coating resisted corrosion of the nickel during the course of the evaluation program. The zinc coating suffered corrosion, as shown by the average 2 to 3 mil loss in thickness of the original zinc layer. In the current ASTM Specification for Zinc (Hot-Galvanized) Coatings on Products Fabricated from Rolled, Pressed, and Forged Steel Shapes, Plates, Bars and Strip (A 123-73) specification, zinc thickness on ¼-in.-diameter bar must be approximately 4 mils (2.3 oz/ft^2). Based on the loss of zinc coating on coated rebars in this study, one could surmise that failures of concrete castings containing 4-mil coated zinc rebars might begin to appear after approximately 11 to 12 years.

General Observations

Concrete failures always radiated outward from the rebar, which represented the path of least resistance. There were no instances where the concrete block was observed to have cracks radiating towards the center of the casting.

The use of admixtures had an adverse effect on the performance of reinforced concrete.

All of the reinforced rod materials displayed a greater resistance to corrosion in concrete prepared with 7.4 gal of water per sack than in concrete with 8.0 gal of water per sack.

The water-cement ratio of concrete is believed to have a significant influence on the ability of concrete to resist chloride penetration (migration). Today, recommended standards call for a water to concrete ratio (w/c) of 0.40 (4.5 gal of water per bag of concrete) with a minimum of 2 in. concrete cover of the reinforcing rods and w/c 0.50 (5.6 gal of water per bag of concrete) with 3 in. concrete cover.

Conclusions

This investigation has shown that concrete cracking and spalling can develop from many causes, for example, thermal gradients, insufficient cover, poor quality concrete, admixtures, rebar material, and exposure location.

The results show that the cracking can be minimized and spalling eliminated by the use of metallic-coated reinforcing rod material. Also, the results show that a 1-mil nickel coating is sufficient to achieve this improvement in performance.

References

[*1*] Tremper, B., Beaton, J. L., and Stratfull, R. F., "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, Bulletin 182, Highway Research Board, 1958.

[*2*] "Deck Holes Plague Bridge," *Engineering News-Record,* p. 48, 1959.

[*3*] Rosa, E. B., McCollum, B., and Peters, O. S., "Electrolysis in Concrete," Technical Paper No. 18, U.S. Bureau of Standards, 1913.

[*4*] Ost, Borje and Monfore, G. E., *Materials Protection,* Vol. 13, No. 6, 1974, p. 21.

[*5*] Clear, K. C. and Hay, R. E., "Time to Corrosion of Reinforcing Steel in Concrete Slabs, Volume 1, Effect of Mix Design and Construction Parameters," Report No. FHWA-RD-73-32, Interim Report, Federal Highway Administration, April 1973.

[*6*] Boyd, W. K. and Tripler, A. G., Jr., *Materials Protection,* Vol. 7, No. 10, 1968, p. 44.

[*7*] Pourbaix, M., *Atlas of Electrochemical Equilibria in Aqueous Solutions,* translated from the French by J. A. Franklin, Pergamon Press, New York, 1966, pp. 409–410.

[*8*] Griffin, D. F., "Effectiveness of Zinc Coating on Reinforcing Steel in Concrete Exposed to a Marine Environment," Technical Note N-1032, U.S. Port Hueneme, Calif., July 1969.

[*9*] Woods, Hubert in *Symposium on Significance of Tests and Properties of Concrete and Concrete-Making Materials, 2nd ed., ASTM STP 169 A,* American Society for Testing and Materials, 1966, pp. 230–238.

A. R. Cook[1] *and S. F. Radtke*[1]

Recent Research on Galvanized Steel for Reinforcement of Concrete

REFERENCE: Cook, A. R. and Radtke, S. F., "**Recent Research on Galvanized Steel for Reinforcement of Concrete,**" *Chloride Corrosion of Steel in Concrete, ASTM STP 629,* D. E. Tonini and S. W. Dean, Jr., Eds., American Society for Testing and Materials, 1977, pp. 51-60.

ABSTRACT: This paper presents data which show that galvanized rebar is superior to untreated steel. Through extensive testing and laboratory experimentation, this paper provides data to resolve the question of whether or not there is an improvement in galvanized rebar's corrosion resistance over unprotected steel. Tests were performed on bridge structures ranging in age from 3 to 23 years.

KEY WORDS: corrosion, concretes, reinforcing steels, galvanized materials, chlorides, nomographs

Today there is much concern about the protection of reinforced concrete structures which are exposed to aggressive environments. This concern becomes most evident for bridge decks where there is an extensive chloride buildup due to deicing salts, ocean salt spray, and salt water in the concrete mix aggregate. The cost of premature replacement of structures has increased to astronomical proportions when inflation, material shortages, and rising labor costs are taken into consideration.

Hence, this study was initiated to provide data on the longevity of galvanizing and to obtain a measure of its effectiveness for this particular application.

Procedure

The test program was designed to obtain a comparison of the relative performance between black and galvanized reinforcing steel from full-scale bridges with combinations of galvanized and black steel rebar where available.

The selected bridges were the following: (*a*) Longbird Bridge, Bermuda

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(galvanized steel only); (*b*) Boca Chica Bridge, US 1 near Key West, Florida; (*c*) Seven Mile Bridge, US 1 between Marathon and Key West, Florida; and (*d*) Long Dick Creek Bridge, Ames, Iowa.

By testing concrete specimens from galvanized and untreated steel structures in the same bridge, an objective and reliable analysis of the contrasting materials under similar conditions can be made.

Results

Longbird Bridge Inspection

Built in 1953, this bridge was 21 years old at the time of inspection. The bridge is a single-span concrete deck approximately 18 by 20 ft (5.5 by 6 m) with a 2-in. (5-cm) asphalt overlay.

Petrographic examination of cores failed to reveal any evidence of steel corrosion, and the concrete showed no impending distress. The galvanized coating had approximately 60 to 75 percent of the original coating remaining, using the most conservative evaluation. The chloride analysis produced the results summarized in Table 1. The data were corrected for a measured nonevaporable water content of 5.59 percent and are based on a measured unit weight of 3740 lb/yd^3 (1696 kg/m^3) of concrete.

TABLE 1—*Results of chloride measurements at various depths at Longbird Bridge.*

Depth at Which Specimen Was Taken, in.	lb Cl$^-$/yd^3 Concrete
0 to ¼	3.31
¾ to 1	2.04
1½ to 1¾	1.98
2¼ to 2½	1.68[a]
3 to 3¼	2.24

[a] Denotes level of top steel.

At the bar level, there were 1.7 lb of chloride ion per cubic yard (1.0 kg/m^3) of concrete which is above the critical level of 1.1 to 1.3 lb/yd^3 (0.65 to 0.77 kg/m^3).

The electrical potentials are summarized in Fig. 1. Readings were taken on a 4-ft (1.22-m) grid and range from 370 to 450 mV to the copper-copper sulfate reference electrode.

The metallographic examination showed a well-defined delta layer (90 percent zinc, 10 percent iron) bounded by an outer, uniform layer of small oriented zeta crystals (94 percent zinc, 6 percent iron). The inter-

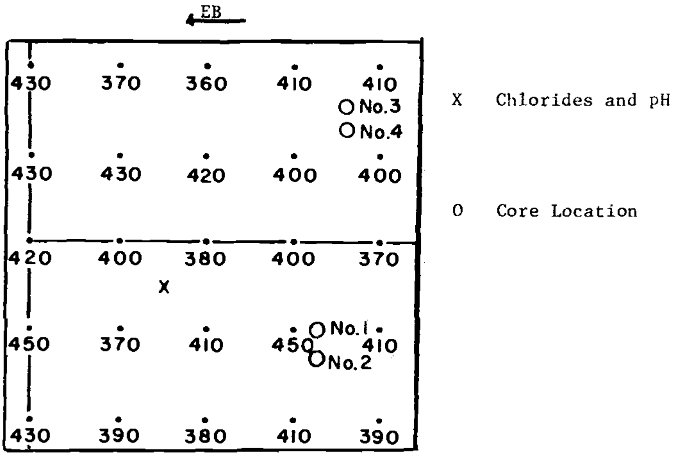

FIG. 1—*Results of electrical potential measurements for Longbird Bridge (potentials expressed in millivolts).*

metallic alloy thickness was 5.2 mils, while the total coating weight was 3.6 oz/ft^2 (1098.6 g/m^2) of surface. Moderate corrosion of the free zinc layer had occurred. However, the mixture of free zinc containing zeta crystals still made up nearly one half of the remaining coating.

Boca Chica Bridge Inspection

Three years after being erected, the first and third spans from the south end were examined. These are simple spans supported by prestressed concrete girders. The first span contains galvanized and the third span contains black steel. The third span contained 0.9 to 1.0 lb (0.53 to 0.59 kg/m^3) of chloride per cubic yard which is slightly below the cited threshold level of 1.1 to 1.3 (0.65 to 0.77 kg/m^3), above which corrosion of black steel is probable. While below the chloride corrosion threshold, corrosion had commenced on the black reinforcing steel. On the first span containing galvanized reinforcing steel, no evidence of corrosion was present, although the chloride level was twice as high as in those locations where untreated steel was used.

Concrete mix inspection showed that cement factor 504 lb/yd^3 (298.872 kg/m^3, 6 bags/yd^3, 4.587 bags/m^3) had a water cement ratio = 0.44 (5 gal/bag (21.134 litres/m^3)) and an air content of 5.8 percent.

The concrete cover (Fig. 2) showed no subsurface defects or delaminations in either span.

The chloride analysis provides data tabulated in Table 2. The figures were corrected to compensate for nonevaporable water contents of 3.55 percent for concrete containing galvanized and 3.52 percent for untreated

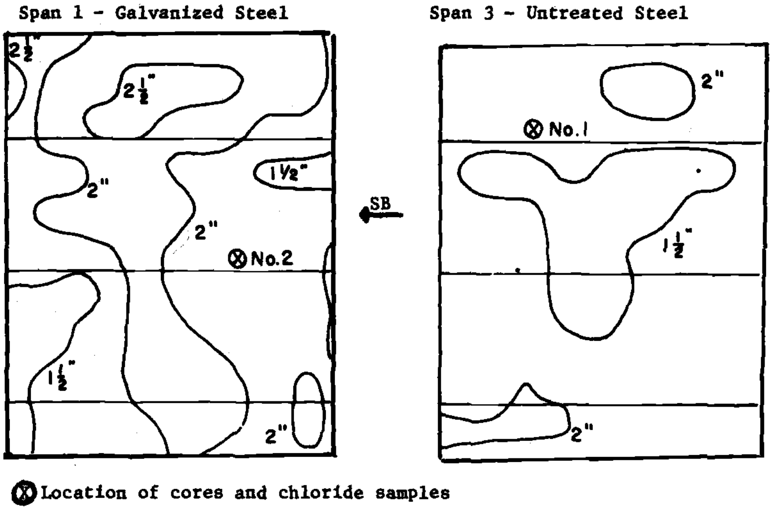

FIG. 2—*Diagram showing cover over top transverse steel in the two spans.*

TABLE 2—*Results of chloride measurements.*

Depth at Which Specimen Was Taken	lb Cl^-/yd^3 Concrete	
	Span 1 (galvanized steel)	Span 3 (untreated steel)
0 to ¼ in.	1.70	1.45
¾ to 1 in.	2.70	1.06
1½ to 1¾ in.	1.95[a]	0.91[a]
2¼ to 2½ in.	1.30	0.91
3 to 3¼ in.	0.83	0.53
3¾ to 4 in.	1.64	1.10
6½ to 7¼ in.	2.20[a]	0.99[a]
7¼ to 8 in.	2.32	1.35

[a] Denotes levels of reinforcing steel.

steel. Once again, data were obtained from portion core specimens taken from each span.

Electrical potentials to a copper-copper sulphate half cell expressed in millivolts are shown in Fig. 3.

Seven Mile Bridge Inspection

The section of the bridge examined was erected in 1972. Therefore, the specimens inspected were three years old. This section is located in the northbound lane in replacement panels Nos. 6 and 7 from the south end

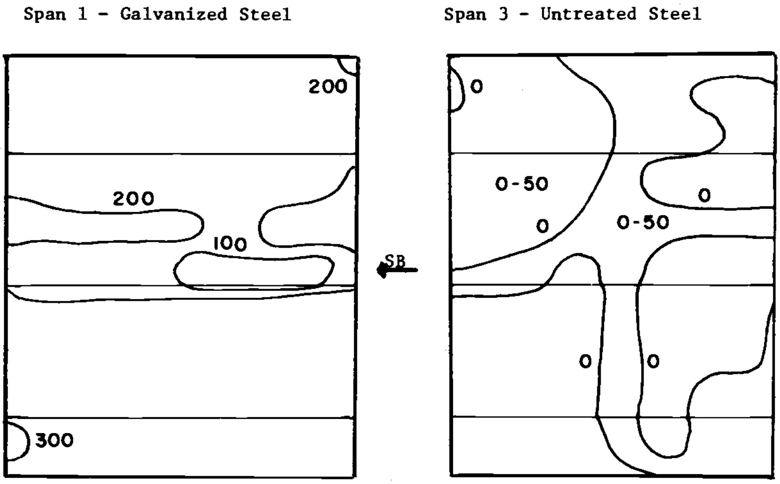

FIG. 3—*Diagram of distribution of electrical potentials in top mat of reinforcing steel for the two spans (potential expressed in millivolts).*

of the bridge. The panels are 12 by 5 ft (3.675 by 1.524 m) and include the curb section of the deck with a 1-in. (2.54-cm) asphalt overlay.

The reinforcement bars in both panels exist in a corrosive environment for untreated steel. The chloride content at the reinforcement level for galvanized steel was 1.43 lb/yd^3 (0.85 kg/m^3) and 2.53 and 5.31 lb/yd^3 (1.5 and 3.1 kg/m^3) for concrete containing black steel. As shown later, electrical potential examination indicated that active corrosion of the black reinforcing steel had commenced. Petrographic examination confirmed that corrosion occurred, and, more importantly, distress was associated with the black steel. However, in the case of the galvanized steel, there was no sign of corrosion or distress, in spite of the high chloride levels. Only a very superficial reaction of concrete on the galvanized coating was found.

There was no concrete mix inspection performed. Hence, no information was reported.

The coating weight on the galvanized steel was 4.5 oz/ft^2 (1.375 kg/m^2) of surface which gave a total thickness of 7.7 mils.

A uniform layer of delta alloy (90 percent zinc, 10 percent iron) 0.4 mils thick exists at the steel surface. The coating consists almost entirely of a zeta layer (94 percent zinc, 6 percent iron) at 7.3 mils thick. Electrical potential on 4-ft (1.2-m) linear spacing was: No. 6, untreated steel, 430, 440, 450 mV and No. 7, galvanized steel, 280, 280, 310 mV.

The data tabulated in Table 3 provide chloride levels corrected for measured nonevaporable water. The values for the nonevaporable water were 4.25 percent for concrete containing galvanized steel and 4.20 percent for

TABLE 3—*Results of chloride measurements at various depths at Seven Mile Bridge.*

	lb Cl$^-$/yd^3 of Concrete	
Depth at Which Specimen Was Taken, in.	Panel 6 (untreated steel)	Panel 7 (galvanized steel)
0 to ¼	...	1.83
1¾ to 2	2.38	1.43[a]
3 to 3¼	2.53[a]	1.57
9 to 10	5.31[a]	2.20
10 to 11¼	4.85	2.34

[a] Denotes level of reinforcing steel.

concrete containing black steel. The No. 6 panel was found to have a unit weight of 3880 lb/yd^3 (2301 kg/m^3), while the No. 7 panel was found to have a unit weight of 3800 lb/yd^3 (2254 kg/m^3).

Long Dick Creek Bridge Inspection

This bridge was constructed in 1967. The examination was performed when the structure was seven years old.

Both galvanized and black steel performed satisfactorily and are not presently exposed to a corrosive environment. Although this structure will be of greater interest in the future, following the onset of more aggressive conditions, it is worth mentioning that the concrete is of high quality (7.55 bags/yd^3 (5.78 bags/m^3)), and relatively extensive cracking in the deck surface was found. Further, cracking appears to be equally distributed for both galvanizing and black steel (Fig. 4).

For the concrete mix, the cement factor was 710 lb/yd^3 (421 kg/m^3), the water cement ratio was 0.40 to 0.41 (4.5 to 4.6 gal/bag (1.1 litre/bag)), and the air content was 5.2 to 6.2 percent.

Bridges Not Actively Corroding

The results of the Portland Cement Association (PCA) study of bridges which were not yet subject to extremely aggressive conditions are not included in this report. These structures will take on added importance in the future when salt concentrations have attained critical levels.

Test Slab Experiment

The results of a carefully designed exposure test performed by PCA are as follows:

There are four elevated slabs in an outdoor test plot at the PCA Re-

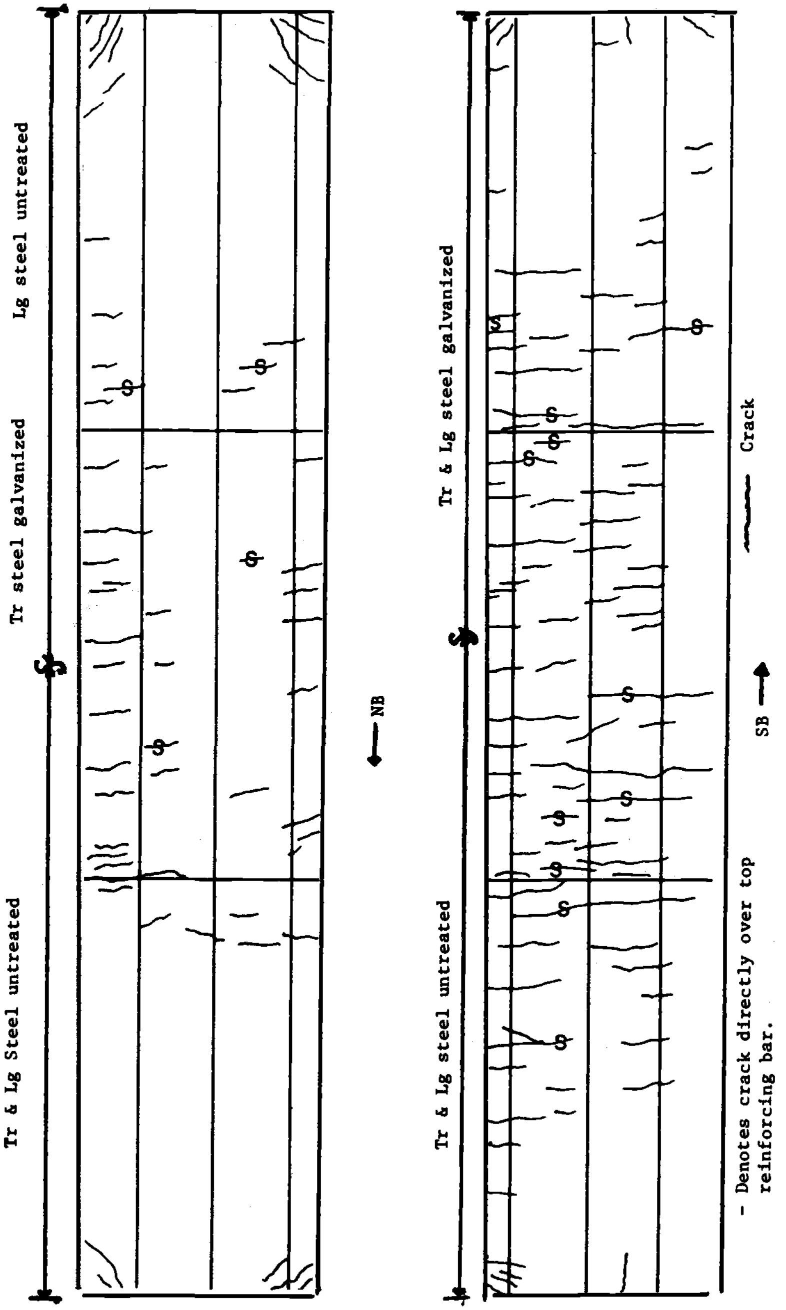

FIG. 4—*Diagram showing locations of cracks visible at wearing surface.*

search Center in Skokie, Ill. The slabs are 4 ft by 5 ft by 6 in. (1.219 by 1.524 by 0.152 m). Each slab is treated with calcium chloride deicer salts about 15 times each winter at the rate of 0.1 lb/ft^2 (0.5 kg/m^2). These slabs carry slotted dikes which permit snow to collect but allow the deicer solution to drain, thus simulating bridge deck conditions.

The results of this experiment are consistent with field experience. Exposed since 1963, slab No. 6 containing untreated steel had a ½-in. (1.3-cm) cover. It shows extensive rusting and spalling at the top wearing surface. Indication of steel corrosion began in 1965.

In contrast, slab No. 29, first exposed in 1968, containing galvanized steel with ½-in. (1.3-cm) cover, shows no evidence of distress after six years of exposure.

Slabs No. 2 and 28 have 1½-in. (3.8 cm) of clear cover over the top reinforcing steel. Slabs No. 6 and 29 have a cover of ½ in. (1.3 cm). In slabs No. 28 and 29, both mats are of reinforcing and galvanized steel. The concrete mix design provided for a cement factor of 564 lb/yd^3 (335 kg/m^3, 6.0 bags/yd^3, (4.6 bags/m^3). The water cement ratio was 0.43 to 0.44 (4.8 to 4.9 gal/bag (1.1 to 1.2 litre/bag)), and the air content was 5.5 to 6.2 percent.

The results of electrical potential measurements are shown in Fig. 5. The potentials are given in millivolts for each measured location because of the small size and the relatively small range for each slab.

Surface defects and subsurface delaminations are shown in Fig. 6. Only where black reinforcing steel was used with ½-in. (1.3-cm) cover was distress detected.

Results of pH measurements are presented in Table 4.

The results of the chloride analysis are summarized in Table 5. All data were corrected for a measured nonevaporable water content of 3.3 percent and are based on a measured unit weight of 4050 lb/yd^3 (2402 kg/m^3) of concrete.

Chloride tests on both plates confirm that the top units are both exposed to an environment corrosive to unprotected steel. In slabs No. 28 and 29 containing galvanized rebar, only superficial corrosion of the galvanized coating was observed.

TABLE 4—*Results of pH measurements.*

Depth at Which Specimen was Taken, in.	pH at Location Indicated in Slab Indicated			
	No. 2	No. 6	No. 28	No. 29
0 to ¼	12.5	12.5	12.7	12.7
½ to ¾	12.7	12.7[a]	12.7	12.8[a]
1½ to 1¾	12.8[a]		12.8[a]	

[a] Denotes level of top steel.

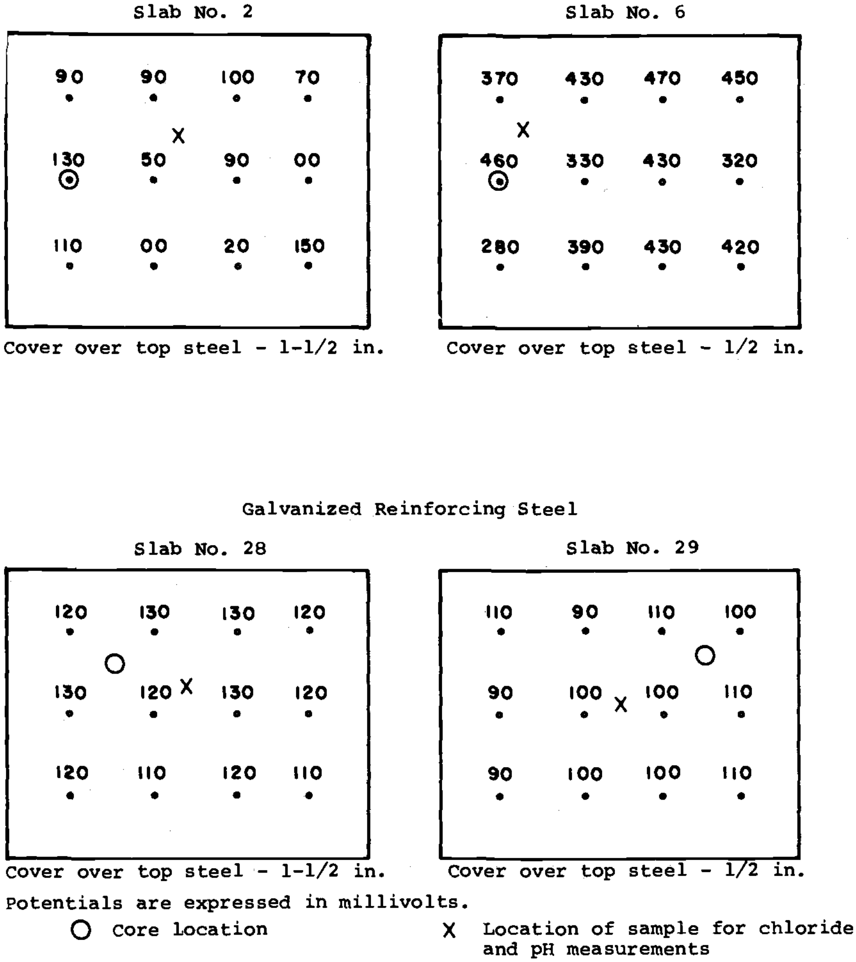

FIG. 5—*Results of electrical potential measurements for slabs in the PCA outdoor test plot.*

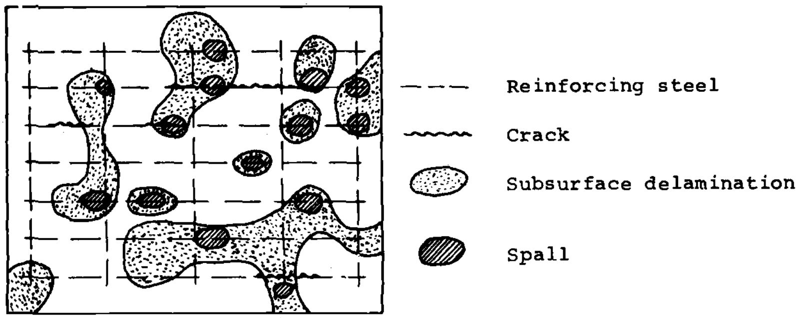

FIG. 6—*Schematic diagram of slab No. 6 showing extent of distress.*

TABLE 5—*Results of chloride analyses.*

Depth at Which Specimen Was Taken, in.	lb Cl^-/yd^3 of Concrete in Slab Indicated			
	No. 2	No. 6	No. 28	No. 29
0 to ¼	8.6	13.5	7.8	5.9
½ to ¾	4.7	5.8[a]	2.5	1.5[a]
1½ to 1¾	1.6[a]	...	0.5[a]	...

[a] Denotes level of top steel.

Discussion

The performance of galvanized steel structures examined in this study was considered to be excellent to outstanding. Despite adverse environmental factors, galvanized steel withstood the ravages of hostile chloride-bearing environments very effectively. In spite of chloride levels well above the threshold of corrosion for black steel, galvanized steel was only superficially affected. Electric potentials were also lower for galvanized steel, minimizing the battery action effect.

Conclusion

Evidence is cited from both field testing and laboratory results showing that galvanized rebar is an effective protection system. Based on the foregoing, the weight of the evidence is strongly in favor of galvanizing reinforcement bar increasing the service life of a bridge deck well beyond what black steel will provide in a highly aggresive environment.

P. C. Peterson[1]

Concrete Bridge Deck Deterioration in Pennsylvania

REFERENCE: Peterson, P. C., "**Concrete Bridge Deck Deterioration in Pennsylvania,**" *Chloride Corrosion of Steel in Concrete, ASTM STP 629,* D. E. Tonini and S. W. Dean, Jr., Eds., American Society for Testing and Materials, 1977, pp. 61-68.

ABSTRACT: Concrete bridge deterioration is a serious problem in Pennsylvania. The causes of cracking, scaling, and spalling are related to prevalent design and construction practices. Although improved criteria and specifications will help solve the problem, new materials and techniques are needed. The Pennsylvania Department of Transportation has used galvanized and epoxy-coated reinforcement bars and latex-modified concrete overlays extensively.

KEY WORDS: corrosion, concretes, reinforcing steels, bridge decks, cracking, scaling, spalling, deicers, galvanized materials, epoxy coatings, latex

Premature failure of concrete bridge decks is a serious problem in the Commonwealth of Pennsylvania, if for no other reason than the fact that the Pennsylvania Department of Transportation (PennDOT) has to maintain approximately 14 000 bridges on 45 000 miles of highways. Most bridge decks are exhibiting significant deterioration after 7 to 10 years and must be replaced before 20 years.

The problem hit home in early 1972 with the impending failure of the deck of the University Avenue Bridge, a major structure that carries an expressway over the Schuylkill River in the heart of Philadelphia. The bridge has 9 spans with a total length of 1647 ft.

In July of 1972, a contract was let to replace the deck. The work involved maintaining traffic of 80 000 vehicles a day on nearby local roads, placing 3000 yd^3 of concrete and 800 000 lb of galvanized reinforcement, and overlaying the completed deck of 10 000 ft^2 with latex-modified concrete, ¾ in. thick.

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It was at that time that PennDOT decided that it could no longer afford to continue with the unfortunately long-held philosophy of solving the problem by simply improving design criteria and quality control of construction. New materials and concepts had to be tried on every new bridge built, even if these ideas were not yet completely proven effective.

That was a hard decision, but it was more difficult to close a major urban expressway for four months, spend $1.5 million, or $150/yd^2 to maintain just one 15-year-old-bridge. This project was controversial and provided front page newspaper reading in Philadelphia.

Today, with the benefit of research and just plain hindsight, bridge engineers know why so many concrete bridge decks are failing. Many interesting and very worthwhile articles and papers on the subject have been published. A concise summary of the causes of bridge deck deterioration is presented, emphasizing those factors that, based on practical field experience, seem to be most important. In addition, the major actions that the Pennsylvania Department of Transportation has taken to correct the problem are listed.

The deterioration can be divided into three classifications: cracking, scaling or the loss of surface mortar, and spalling or potholes. Salt is a contributing cause to scaling and the predominate cause of spalling. Chlorides and cracking are not related, but cracks will readily result in potholes because of salt intrusion.

Cracking

Cracking is more prevalent on decks of skewed bridges and those with longer spans and continuous beams. It is common to find twice as much cracking in the deck of a continuous structure than in a comparable simple bridge.

Generally, because of the low level of designed stress in bridge decks, flexural cracking due to loads is not encountered. About 80 percent of all cracks are transverse, that is, perpendicular to the line of traffic. Usually the crack occurs in line with the transverse top reinforcement.

Most surface cracking seems to be due primarily to concrete volume changes caused by poor construction practices, including concrete that is too wet, placed too slowly, poorly finished, and improperly cured. These practices result in longitudinal shrinkage and the subsidence of the concrete around the top reinforcement, the direct causes of cracking.

Fresh concrete cast into a deck continues to consolidate after it is vibrated and finished. The deck itself continues to deflect and vibrate as additional concrete is placed to complete the span. Aggregate particles in the concrete tend to settle to the bottom. Water is displaced and bleeds to the top. The twins mats of reinforcement in the deck disrupt this subsidence, causing differential consolidation; that is, more settlement occurs between the bars than over them. The wetter the concrete, the more pronounced will be this difference. Over a given bar, concrete is caused to separate as particles tend to flow to one side or the other to lower settled areas between the bars. Plastic shrinkage, drying shrinkage, and thermal stresses find relief in this area of local weakness and cause a crack, particularly if the steel is close to the surface and particularly if the concrete is very wet and thus subject to more shrinkage [*1*].[2]

Placement and finishing time affect the rate of cracking. The higher these values are, the greater the amount of cracking. A rapid rate of concrete placement provides more time for the concrete to remain in a plastic condition so that the total pour can be completed without the likelihood of damage. The finishing should be completed as rapidly as possible with a minimum amount of manipulation of the surface after consolidation. Extra finishing also makes the surface more susceptible to scaling. The reduction in the amount of needed finishing of the structural deck is one of the advantages of using overlays [*2,3*].

Concrete may also crack due to rapid evaporation of moisture during the early stages of hardening. If the evaporation rate is much greater than 0.1 lb of water per square foot per hour, such cracking will almost certainly occur unless precautions are taken. For example, if the air temperature is 90°F and the relative humidity is 50 percent with the wind blowing at 15 mph and the concrete itself at 80°F, the rate of evaporation is approximately 0.1 lb of water per square foot per hour, the acceptable limit. If the weather is the same as in the previous example, but the concrete temperature is 90°F, just 10 degrees more, the rate of evaporation is 0.23 lb of water, twice the acceptable value [*1*]. Rapid drying is such a serious problem that, in 1974, PennDOT adopted the following specification:

> In hot weather, the aggregates and/or the mixing water shall be cooled as necessary to maintain the concrete temperature at the time of placing within the limits of 50 F to 90 F, except bridge deck concrete.
>
> The temperature of concrete at the time of placement in bridge decks shall be between 50 F and 85 F. In addition, when the air temperature is 70 F or greater, the temperature of the concrete shall not be more than 5 F higher than the air temperature.
>
> No bridge deck concrete shall be placed when the predicted relative humidity at the job site is below 35%.
>
> The contractor will be allowed to complete the placement to a bulkhead, at a predetermined location, in the event humidity or weather conditions change unexpectedly.

While not ideal, it was felt that this specification was an important first step in the right direction. In any event, it met with some initial resistance, but it is now accepted and followed.

The type of formwork used is a factor. A study performed by The

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[2] The italic numbers in brackets refer to the list of references appended to this paper.

Pennsylvania State University determined that bridge decks formed with conventional plywood forms had more than three times as much transverse cracking than decks formed with corrugated metal stay-in-place (SIP) forms. The decreased crack intensity resulting from the use of SIP forms could possibly be the result of a slower loss of water from fresh concrete, or because the deck is stiffer since SIP forms develop composite action in the lateral direction [*4,5*].

The timely application of curing is especially vital in the prevention of shrinkage cracks. Wet curing should be applied as soon as the visible bleed water has evaporated. It may be necessary during hot weather to use a membrane cure or an evaporation retardant to protect the surface during the time that wet burlap cannot be applied without marring the finish.

Cracking must be controlled. As the years go by, water, dirt, and salt wash into the cracks. This action is increased by the flexing of the bridge superstructure under live loads. The crack is not the problem; it is the salt that gets into it [*1,3*].

Scaling

Scaling or surface mortar deterioration is comprised primarily of wear and disintegration due to weathering of weak mortar. It can virtually be eliminated by the use of high-quality air-entrained concrete. Low air content is always associated with scaling. However, more important than the total amount is the requirement that the air bubbles be properly spaced and uniformly distributed throughout the depth of the slab. This is why improper finishing such as overworking and overwetting causes deterioration. After construction, periodic applications of linseed oil further help prevent scaling from developing.

Surprisingly, scaling is not directly related to traffic. Some bridges with average daily traffic counts of over 10 000 vehicles have no surface mortar deterioration, and others have deteriorated and nondeteriorated spans adjacent to one another. Therefore, it is possible to construct wear-resistant decks. On the other hand, scaling is related to the type of deck forms. The Pennsylvania State University study determined that, while decks constructed with SIP forms exhibit less cracking, they have about three times as much scaling as decks conventionally formed or poured on adjacent prestressed concrete box beams. This may be caused by the fact that metal forms block the evaporation of moisture from the undersides of the deck so that, during its lifetime, a deck with SIP forms is continually wetter than other decks [*1,4*].

Although not a corrosion process, scaling is indirectly caused by salt and other factors. When newly poured concrete dries, capillary tubes are left by the evaporating water. Concrete with a low water-cement ratio will have smaller and fewer potentially vulnerable capillary cavities and will therefore be more resistant to freezing weather. Air entrainment helps because, in any system of voids, moisture tends to move from the larger voids to the smaller ones. Since air-entrained voids are far larger than the capillary tubes, they remain essentially free of water and are therefore available as points of pressure relief when the moisture starts to freeze in winter.

Deicing salts make the situation worse. As snow and ice melts on the deck, the temperature immediately below the surface is reduced significantly by the heat of fusion of ice. The principal is the same as that for an old-fashioned ice cream freezer. This fact may cause a saturated subsurface zone to freeze, rupturing the cement gel, even though there is melted ice on the top surface of the deck. In addition, the concentration of salt crystals in the capillary tubes cause the development of osmotic pressures. However, if a deck has had a chance to age before the first application of salt, the likelihood of scaling is greatly reduced [*1*].

Spalling

Of the three types of deterioration, spalling or fracture planes are the most serious and unfortunately the most commonplace. Spalling, of course, severely affects the riding quality and structural integrity of the deck. The predominate cause is the corrosion of reinforcement. Salt water can readily enter into a transverse crack over a rebar and attack the steel directly. But concrete itself is permeable, and this permeability is particularly great when the water-cement ratio of the mix is high. In a controlled laboratory experiment conducted by the Federal Highway Administration (FHWA), slabs constructed with 0.50 and 0.60 water-cement ratio concrete had 3 ½ and 4 times as much chloride at a depth of 1 in. as did slabs poured with 0.40 concrete [*1,6*].

The FHWA has also shown the importance of the amount of concrete over the reinforcing steel. For typical bridge deck concrete (water/cement (W/C) = 0.50), with 1-in. rebar cover, only 7 laboratory-controlled salt applications were required to induce rebar corrosion. With 3 in. of cover, the steel did not corrode after 330 applications. Researchers in California demonstrated that, for each additional 1 in. of depth, the chloride content would be reduced by approximately one half [*6,7*].

In order to ensure at least 2 in. of cover in the completed deck, PennDOT in 1974 changed its design criterion from 2 to 2 ½ in. A survey of existing bridges indicated that this action was necessary to ensure that the average cover will be greater than 2 in. at the 90 percent confidence level. The Department has emphasized to its contractors and inspectors the importance of good construction practices that will result in the accurate placement of steel in the hardened deck. As part of the final inspection of every bridge deck, pachometer (an instrument used for the precise and rapid location and measurement of reinforcing bars encased in hardened concrete) readings are taken at 5-ft centers. Real improvement has been made, and PennDOT intends to develop a performance specification for this aspect of bridge construction soon.

The high pH (12 to 13) of concrete normally protects the reinforcement from corrosion. The soluble calcium hydroxide (lime) liberated by the hydrating cement forces the formation of ferric oxide or mill scale on the surface of the steel. This reaction stops when a film has been formed. However, when soluble chlorides are introduced into the concrete, the protective high pH environment begins to neutralize. When sufficient chloride is present to destroy the necessary alkalinity and the normal passivity of the reinforcement steel, galvanic cells are created, and rust results. In atmospheric corrosion, the cells are microscopically spaced, and the corrosion appears as a continuous film. In chloride-contaminated concrete, the steel corrodes in a manner that is commonly referred to as macroscopic, where the corroding area may be spaced several feet from the cathode or noncorroding area [*8,9*].

In any event, rust has been found to occupy twice the volume as the original material and to exert a force of 4700 psi, many times the tensile strength of concrete. Laboratory testing showed that less than 1 mil of corrosion loss was sufficient to crack ⅞-in.-thick concrete. This is what causes a pothole to be formed over the reinforcement [*1,7*].

Research by the FHWA indicates that the total amount of chloride at the level of the rebar needed to start corrosion ranges between 1.1 and 2.0 lb of chloride per cubic yard of concrete. PennDOT has discovered that many times this condition exists in new bridges after only two winters [*8*].

Research in California indicates that, once the concrete deck becomes contaminated with chloride, spalling is not controlled by chloride content beyond that needed to depassivate the steel, but is controlled by other variables, for example, moisture content that specifically relates to the corrosion process [*10*].

Deicing Salts

Since deicing salt is the main cause of bridge deck deterioration, it is interesting to note that PennDOT spreads between 600 000 to 750 000 tons of salt onto its bridges and highways each winter. The cost of the salt alone is about $9 million. In addition to the sodium chloride, about 10 000 tons of calcium chloride is used each winter, along with 2 million tons of abrasives. Pennsylvania motorists expect and their elected officials demand bare pavements during the winter.

This policy is expensive. For a district comprising Philadelphia and the four surrounding suburban counties, the Pennsylvania Transportation Commission has programmed bridge deck replacement projects totaling over $25 million. Many more projects will have to be done in the future. Unfortunately, PennDOT's financial resources are already severely strained so that it is not certain when much of this work can be started.

Deck Protection Features

Although there is a backlog of deteriorated decks that must be replaced, the Department is making every effort to ensure that the new decks being built are the most durable that can be constructed. Many products and techniques, particularly membrane systems, have been tried, and new ideas and materials are being developed (for example, internally sealed or wax-impregnated concrete). Most experimental experience has been with galvanized and epoxy-coated reinforcement steel and latex-modified concrete overlays.

In 1974, the Department received prices for 460 000 yd^2 of latex-modified concrete overlay, 24 200 000 lb (465 000 yd^2) of galvanized reinforcement, and 5 500 000 lb (241 000 yd^2) of epoxy-coated rebars. The total deck area under construction that year was 1 167 000 yd^2 or 165 lane miles. In 1975, bids were received for only 74 lane miles of bridge decks. About 32 percent of these were overlaid with latex concrete; 54 percent contained galvanized rebars, and 14 percent epoxy coated reinforcement.

Although costs varied, the average price recieved for galvanizing the reinforcement bars was 14^c/lb and 39^c for the epoxy coating. Latex-modified concrete cost about $20/yd^2 with an additional $7 for surface preparation.

Bridge deck deterioration is a problem that must be solved, and PennDOT is working very hard to do just that.

References

[*1*] "Concrete Bridge Deck Durability: Synthesis of Highway Practice 4," National Cooperative Highway Research Program, Highway Research Board, 1970.

[*2*] Cady, P. D. et al, "Final Report on Durability of Bridge Deck Concrete: Part 1, Effect of Construction Practices on Durability," The Pennsylvania State University, University Park, Pa., Sept. 1971.

[*3*] "A Critical Review of Certain Aspects of Bridge Design and Construction Practices," seminar report, Howard, Needles, Tammen & Bergendoff, Kansas City, Mo., 1969.

[*4*] Cady, P. D. and Carrier, R. E., "Final Report on Durability of Bridge Deck Concrete: Part 2, Moisture Content of Bridge Decks," The Pennsylvania State University, University Park, Pa., Oct. 1971.

[*5*] Cady, P. D. et al, Final Report on Durability of Bridge Deck Concrete: Part 3, Condition of 249 Four-year-old Bridge Decks," The Pennsylvania State University, University Park, Pa., Dec. 1971.

[*6*] Clear, K. C. and Hay, R. E., "Time to Corrosion of Reinforcing Steel in Concrete Slabs: Vol 1, Effect of Mix Design and Construction Parameters," Interim Report, Federal Highway Administration, Washington, D.C., April 1973.

[*7*] Spellman, D. L. and Stratfull, R. F., "Chlorides and Bridge Deck Deterioration," Highway Research Record No. 328, 1970, pp. 38–49.

[*8*] Clear, Ken, ***Public Roads,*** Vol. 39 No. 2, Sept. 1975, pp. 53–61.
[*9*] Kliethermes, J. C., "Repair of Spalling Bridge Decks," Highway Research Record No. 400, 1972, pp. 83-92.
[*10*] Stratfull, R. F. et al, "Corrosion Testing of Bridge Decks," Transportation Research Record, No. 539, 1975, pp. 50–59.

R. I. Frascoia[1]

Vermont's Experience with Bridge Deck Protective Systems

REFERENCE: Frascoia, R. I., "**Vermont's Experience with Bridge Deck Protective Systems,**" *Chloride Corrosion of Steel in Concrete, ASTM STP 629*, D. E. Tonini and S. W. Dean, Jr., Eds., American Society for Testing and Materials, 1977, pp. 69–81.

ABSTRACT: Intrusion of moisture and deicing chemicals into reinforced concrete bridge decks results in premature deterioration of the concrete and corrosion of the reinforcing steel. Vermont's search for an effective waterproofing system which would seal bridge decks and thus prevent intrusion of the moisture and chlorides has resulted in the application and evaluation of 29 different membrane systems since 1971. A number of the protective membranes have been in service for sufficient time to draw conclusions on their effectiveness. The results of followup evaluations show that several systems have provided complete protection to the bridge decks while a number of other materials have prevented chloride intrusion except in areas where deicing chemicals are often trapped due to the ponding effect of curb lines and expansion dams. Such results suggest that the simple and inexpensive materials could be specified for sealing the major portion of a deck surface if a compatible and impervious membrane material is placed over poorly drained areas. Difficulties encountered with the application of sophisticated and expensive membrane systems have resulted in a theory that the search for the most effective waterproofing system should concentrate on uncomplicated and less costly materials which could be applied without difficulty.

The results of laboratory tests indicate that liquid-applied membranes offer substantial protection, even when pinholes and bubbles occur in the coatings, when contrasted with the chloride intrusion which occurs in untreated concrete. The tests also show the relationship between resistivity readings and chloride penetrations and suggest that readings well below the generally accepted standard of 500 000 Ω may be acceptable.

KEY WORDS: corrosion, concretes, reinforcing steels, deterioration, bridge decks, chlorides, membranes, field testing, performance evaluation, resistance

The problem of bridge deck deterioration has generated a concentrated effort to design, construct, and protect both new and old structures with a variety of different protective systems. The methods currently in use or

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under study include two-course bonded construction utilizing low-slump concrete, latex, polymer impregnation, or wax beads, epoxy-coated and galvanized reinforced steel, cathodic protection, neutralization or removal of chlorides, penetrating sealers, mastic asphalt, and waterproofing membranes. Many reports have been written covering nearly all of the varied protective systems.

Of the many techniques being tried, bridge deck waterproofing has had the most widespread use. Vermont is one of many states which have concentrated on the installation and evaluation of membrane systems under the Federal Highway Administration's National Experimental and Evaluation Program No. 12 Bridge Deck Protection Systems. Vermont's experiences have included the use of 29 different systems on 70 new bridge decks since 1971. The materials have included ten preformed sheet products, seven epoxy systems, five hot-applied materials, five polyurethanes, and two cold-applied built-up membrane systems. Fifty five of the protective systems were closely monitored during the applications, and detailed initial reports were issued. Yearly field evaluations are being conducted on the individual systems after the structures have been exposed to two years of traffic and deicing salt applications. The field applications have included resistivity tests, steel potential readings, moisture strip readings, and the recovery of concrete specimens for chloride analysis. Paved but otherwise untreated approach slabs of the experimental bridges are being used as control sections for comparing chloride penetration through the pavements and membranes.

Laboratory Evaluation of Membrane Materials

Experimental membrane installations are generally preceded by a series of laboratory tests developed to determine if the materials merit field trails. The tests include flexibility, crack-bridging ability, resistance to pinholing, resistance to puncture, bond to concrete substrate, and adhesion of bituminous pavement to membrane. Of the many laboratory tests performed, none appear capable of predicting the service life of a membrane system. The tests often disclose that many of the materials previously and currently proposed for use as membrane systems do not possess all of the properties required to satisfactorily seal off all areas of a bridge deck and retain complete impermeability.

Although the value of conducting basic laboratory tests can not be disputed, it should be pointed out that some of the tests are too severe and thus result in the elimination of potentially satisfactory membrane materials. An example is the cold-temperature flexibility test where a material is generally required to resist cracking when bent around a 1-in. mandril at 0°F or lower. Although such flexibility characteristics would be of value, somewhat less flexible materials may well be completely adequate for sealing off even those structures where some cracking and flexing may be anticipated.

The current methods of using the resistivity test[2] in both laboratory and field evaluations could also be subject to additional study. The test is usually accepted as a standard for determining membrane performance with regard to porosity. Readings in excess of 100 000 Ω, and generally, 500 000 Ω are considered necessary for a system to be acceptable. Although high Ω readings would be preferred, low readings on a membrane system would not necessarily mean the material would allow significant quantities of chlorides to penetrate into the concrete. This theory is based on test results obtained on 32 polyurethane and epoxy-coated test slabs which have received nearly continuous ponding applications of a 3 percent chloride solution since October 1973. Five sample slabs were treated with a single 50-mil polyurethane coating. Low initial and followup resistivity readings were recorded on all test areas due to pinholes, craters, and bubbles which were present in the coatings at an average rate of $232/\text{ft}^2$ (Table 1). Yet, although the average resistivity reading was less than 10 000 Ω, only 3.5 lb of chloride were detected in the top ½ in. of the concrete specimens after 770 days. Similar findings were obtained on slabs treated with single epoxy coatings which resulted in holes or bubbles at the rate of $132/\text{ft}^2$. Although the resistivity readings averaged only 6000 Ω, chloride concentrations in the top ½ in. of concrete were found to be only 1.9 and 2.8 lb after 440 and 730 days of ponding, respectively.

In contrast to the treated slabs, chloride concentrations averaging 22.7 lb were detected after 440 days in the top ½ in. of four test slabs which received no protective coating. The nearly continuous ponding applications of 3 percent chloride solution for two years is believed to be similar to 20 years exposure in Vermont's typical roadway environment where deicing chemicals are annually applied at the approximate rate of 30 tons per two-lane mile.

A two-year study has also been undertaken in an attempt to determine the limitations of the field-conducted resistivity test. Initial results on 22 bridges indicate the nondestructive test is 70 percent reliable when compared with the chemical analysis of concrete specimens taken from resistivity test locations.

Experimental Membrane Applications

To date, 46 of the 55 experimental membrane applications made in Vermont have been reported in detail.[3] The information has covered deck

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[2] Spellman, D. L. and Stratfull, R. F., "An Electrical Method for Evaluating Bridge Deck Coatings," Highway Research Record No. 357, 1971.

[3] Frascoia, R. I., "National Experimental & Evaluation Program—Bridge Deck Protective Systems," Initial Reports 72-10 and 73-1, "Experimental Bridge Deck Membrane Applications in Vermont," Reports 74-4 and 75-2.

TABLE 1—*Relationship between resistivity results and chloride penetration through liquid-applied membranes.*

Membrane Treatment	No. of Test Slabs	Holes or Bubbles per ft²	lb Cl at 0 to ½ in. Depth after 730 Days	Ω Resistance		
				High	Low	Average
Polyurethane						
1 coat	5	232	3.5	30 000	550	9 800
2 coats	5	47	1.2	380 000	2 800	98 300
3 coats	5	31	2.0	5 000 000	37 000	802 000
4 coats	1	5	0.6	7 000 000	95 000	3 365 000
Epoxy						
1 coat	3	132	2.8	10 000	1 800	6 000
2 coats	3	33	3.7	28 000	1 200	11 700
3 coats	3	8	0.9	400 000	1 400	82 500
4 coats	2	1	2.4	160 000	5 000	45 000
			lb Cl at Specified Depths, in.			
			0 to ½	½ to 1	1 to 1½	1½ to 2
No treatment	4 test slabs	440 days	22.7	18.5	13.5	11.2
No treatment	4 test slabs	730 days	21.5	22.2	21.0	18.7

construction data and surface condition, membrane product information, laboratory test results, observations made during the applications, weather conditions, resistivity readings, and discussions on the applications, including tentative recommendations with regard to continued use of each system. As followup evaluations and related laboratory tests have been completed, the information and any new recommendations have been included in new reports.

Generally, the initial conclusions drawn after placement of the materials have remained valid. However, as our overall experience with membrane systems has broadened, some changes in theory have occurred. Following are discussions on each general membrane type based upon experiences and evaluation results to date. A summary of observations is also included in Table 2.

Tar Emulsion with and without Glass Fabric

Tar emulsion was placed on nearly all new bridges constructed in Vermont from 1960 to 1971. The necessity of repairing deteriorated concrete adjacent to curb lines and expansion dams within ten years of construction has proven that the material is not capable of protecting concrete in areas where poor drainage or ponding occurs. However, field evaluations of four- to seven-year-old decks treated with two coats of tar emulsion has disclosed no evidence of chloride intrusion in areas where proper drainage occurs due to roadway grade, superelevation, or parabolic. Such areas often comprise approximately 85 percent of a deck surface. This suggests that tar emulsion could be used in combination with a 3-ft-wide application of a satisfactory, compatible sealant along curb lines and expansion dams. A major advantage of tar emulsion is that the deck surface should be wet down immediately prior to the application whereas nearly all other membrane systems are moisture sensitive and require dry conditions. Other advantages include ease of application, low cost, and the ability to resist damage if and when the bituminous overlay must be replaced. Weak points include poor flexibility, as well as inability to seal poorly drained areas, as stated earlier.

The use of glass fabric with tar emulsion is not recommended at this time, based on chloride concentrations found in curb lines areas and at 5, 10, and 15-ft offsets on experimental applications made in 1972 and 1973. Such findings suggest that the glass fabric encourages moisture penetration by producing a wicking action.

Epoxy Systems

The application and evaluation of seven epoxy systems has resulted in a general lack of confidence in such materials. Those which have been ex-

TABLE 2—*Product evaluation summary.*

Observations	Hot Asphalt and Glass Fabric	Mastic Asphalt and Gussasphalt	Standard Preformed Membranes (3 Products)	NCHRP Project 12-11 Preformed Membranes (5 products)	Tar Emulsion With and Without Glass Fabric	Hot-Applied PVC Polymer	Epoxies (Solvent Cut Coal Tar-Modified 100% Solids)	Polyurethanes Modified and 100% Solids
Surface preparation required	wash and sweep	wash and sweep	wash and sweep	wash and sweep, sand blast[2]	wash and sweep	wash and sweep	sand blast or acid etch	sweep or acid etch or both
Moisture sensitive	yes	yes	yes	yes	no	yes	most cases	yes
Ease of application	difficult	difficult	easy to average	difficult	very easy	average	average	average
Flexibility and crack bridging properties	poor	poor	good	excellent	poor	good	poor	good
Bond and seal at curb	fair/poor	poor	fair/good	fair/good	good/poor	good	good/poor	good
Bubbles or pinholes or both in membrane	yes	yes/no	no	no	no	yes	yes	yes
Resistivity reading prior to paving	46 000 to 71 000	infinity	infinity	14 000 to infinity (some materials conductive)	500 to 6000	infinity on protection system	1100 to 1 200 000	480 000 to 2 600 000
Bond between membrane and concrete	good	system included bond breaker	fair to good	fair to good	excellent	good	good	good
Bond between pavement and membrane	good	NA*	good	good	good	fair to good	fair to poor	poor unless roofing sheet or protection boards used
Protection system required over membrane	no	NA	no/possible	yes to promote adhesion between pavement and membrane	no	yes	no	yes (most cases)
Pavement subject to blistering or cracking during installation due to membrane	no	NA	yes	yes	no	slight	no	no
Postconstruction problems with pavement and membrane	no	no	yes/no	yes/no	no	slight	no	no
Cost per square yard not including pavement	$3.75	$62.35	$4.50	$10.65	$1.32, 3.50 with fabric	$4.00	$9.42	$5.19
Performance based on followup evaluations	fair to good	unsatisfactory	fair to good	insufficient service life to draw conclusions	good without fabric except along curb-lines and dams	insufficient service life to draw conclusions	poor to fair	fair to good
Recommendations	await continued followup evaluations	not recommended for further use	continue use of 2 systems with liquid curb line sealant. Await followup on third system	not recommended for further use unless other systems prove to be unsatisfactory	use in combination with compatible curb line sealant on structures not subject to severe cracking delete use of fabric	recommend further experimental use	restrict use, await continued follow-up evaluations	1 of 4 systems recommended limit other materials to use on repair projects or as curb line sealant

*NA = not applicable.

posed to three winters of deicing chemical applications have disclosed slight to moderate chloride intrusion in all areas. Applications exposed to two winters have generally disclosed chloride penetration only along curb lines. Disadvantages of epoxy materials include the requirement for special preparation of the deck surface, the tendency of such materials to pinhole or blister, their general lack of flexibility, age-hardening characteristics, and high in-place cost. The single advantage of epoxies is the stability they provide both on steep grades and by not creating difficulties with the pavement applications.

Polyurethane Membranes

As a group, the liquid-applied polyurethanes offer the greatest potential of any system for sealing curb line areas. Other features include excellent flexibility plus resistance to the type of blistering which might result in difficulties with the pavement applications. The material is subject to moisture vapor pressures outgassing from the concrete which often results in the formation of holes and bubbles in the coating. The rate of pinholing or bubbling can be expected to vary with the density and moisture content of the concrete and with changes in the air temperature. The size and shape of any holes or bubbles which remain in the cured membrane, thus affecting the coating porosity, will often vary with the viscosity and cure rate of each material and with individual applications. Many of the polyurethane systems currently available do not develop sufficient toughness to resist puncture damage and therefore require the use of protection boards or roll roofing for protective purposes. The protection system also serves as a means of promoting adhesion between the membrane and the bituminous overlay.

Only one polyurethane system is currently recommended for further use in Vermont following satisfactory initial results on three applications. The system reduces the effects of outgassing by requiring that the concrete be painted black to prematurely raise the surface temperature of the deck and by restricting the 100-mil polyurethane application to the time of day when the air and deck temperatures level off or begin declining. The system also features greater resistance to puncture than similar products and includes the use of an asphalt cement tack coat for promoting adhesion between the membrane and bituminous overlay. The system has not been in service for sufficient time to determine performance based on a chloride analysis of concrete cores.

Hot Asphalt and Glass Fabric

A three-coat application of hot asphalt combined with two layers of glass fabric has been used with varying success in a number of New

England states for appoximately 20 years. Vermont's experience has been limited to two applications which have provided nearly complete protection against chloride intrusion through two winters' installation. Further use is being withheld until long-term performance can be determined.

The system features low in-place cost and good adhesion to the primed concrete, and does not create stability problems with the bituminous overlay. The poorest feature of the roofing-grade asphalt is its complete lack of flexibility at cold temperatures and its potential for leakage at the curb face due to shrinkage of the asphalt and bituminous pavement during periods of cold temperature.

A hot-applied rubberized-asphalt membrane which was placed on five decks is not recommended for future use due to followup test results which indicate chloride intrusion after three and four winters. The material also was affected by outgassing pressures and created a pavement stability problem under traffic when placed on moderate grades.

Mastic Asphalt and Gussasphalt

A mastic asphalt and Gussasphalt overlay was placed on a single bridge deck during the 1974 construction season. Initial observations and test results over an 18-month period indicated the production and application of the materials were successful, considering the experimental nature of the application. However, the system resulted in failure during the second winter when the seal along the curb lines was lost and approximately 400 lineal feet of random cracks occurred in both the membrane and Gussasphalt. The cause of the failure has not been determined, but it is suspected that it was due to a combination of very low temperatures and rapid temperature changes.

Preformed Sheet Membranes

Vermont currently specifies the three "standard" preformed sheet membrane systems on nonexperimental bridges, and allows the contractor to select the preferred system. The desirable characteristics of the three systems include minimum surface preparation, ease of application with improper installation easily visible to the engineer, controlled membrane thickness, resistance to pinholing, good flexibility, and a low in-place cost averaging \$4.50/yd^2.

A potential weak point with the sheet materials involves obtaining a complete and lasting seal along the deck-curb joint and lower portion of the curb section which consists of a rough granite face on most bridges. Although all three systems have the potential for waterproofing such areas, the end result depends to a great extent on the care and expertise of the workmen making the installation. For that reason, a compatible liquid polyurethane sealant is specified for use along the membrane perimeter and vertical curb face on two of the three systems. A second problem area concerns the occasional formation of blisters during or after the pavement application. Those which occur during paving are believed to be caused by solvents trapped in the primer or by small concentrations of moisture which collect beneath the membrane and consequently turn to a vapor or gas when exposed to the high temperatures of the bituminous overlay. A reduction in bituminous laydown temperatures will eliminate most initial blistering and also lessen the potential for damaging the materials during placement and compaction of the pavement. The blisters which occur after the pavement has been placed are caused by concrete outgassing pressures which lift the membrane off the deck at areas where adhesion to the concrete is poor. Field test results to date indicate that two of the standard sheet membranes are providing the desired protection. Cores taken from a bridge treated with the third system disclosed chloride contamination in wheel path areas and adjacent to the curb line. If similar results are noted on other structures, further use of the material would not be recommended.

Other preformed systems which have been applied include an uncured ethylene-propylene diene monomer (EPDM) membrane and a 45-mil vinyl-neoprene system. Both materials were bothered by blister formations but otherwise appeared satisfactory. Neither system has been in service long enough to obtain valid performance data.

Hot-Applied Poly (Vinyl Chloride) Polymer

Two applications of a single component, hot-applied poly (vinyl chloride) (PVC) polymer membrane have resulted in a recommendation for continued experimental use. The system can be applied without difficulty using the proper heater-field extruder and features a good seal along the curb line plus low basic material cost. Bubbles and pinholes do occur in the coating but their effect is difficult to determine since the material is covered with a roll roofing protection course. The pavement may be subject to stability problems during application due to the thermoplastic characteristics of the membrane. All problems were averted on the second installation by reducing pavement mix temperatures and by obtaining initial compaction with a light-duty roller.

NCHRP Project 12-11 Systems

The five preformed sheet systems selected as most promising in the National Cooperative Highway Research Program (NCHRP) Project 12-11 were placed in Vermont as part of the Phase II study. All five systems were well designed and displayed excellent physical characteristics. How-

TABLE 3—*Chloride concentrations recorded in bridge decks protected with experimental membrane systems.*

Bridge Number and Product	Date Applied	Winters Salted	Chloride Content in Parts per Million Offset From Curb and Specimen Depth					
			1 ft		5 or 10 ft		15 ft	
			0 to 1 in.	1 to 2 in.	0 to 1 in.	1 to 2 in.	0 to 1 in.	1 to 2 in.
1 tar emulsion	7-20-71	4	138[a]	67	37	35	43	44
2 hot rubberized asphalt	8-18-71	4	52	56	82	50	63	51
3 tar emulsion	7-20-71	4	164	136	36	33	35	34
4 hot rubberized asphalt	8-18-71	4	60	51	35	32	46	37
6 tar emulsion and glass fabric	7-14-72	3	86	67	42	35	46	35
7 tar-modified polyurethane	7-31-72	3	63	52	46	45	45	52
8 tar emulsion and glass fabric	7-27-72	3	48	35	118	66	61	45
9 solvent cut epoxy	7-12-72	3	296	89	101	56		
10 cold tar epoxy	7-29-72	3	117	64	82	84		
11 65-mil preformed sheet	8-14-72	3	53	40	44	43	157	95
12 tar emulsion and glass fabric	5-25-72	3	56	48	52	45	46	29
14 tar emulsion and glass fabric	7-24-72	3	183	85	38	40	45	45
15 asphalt-modified polyurethane	5-13-72	3	53	40	32	37	31	38
16 100% solids epoxy	7-20-72	2	50	31	55	36	22	41
17 asphalt-modified polyurethane	8-23-72	2	29	24	36	32	30	24

18 hot asphalt and glass fabric	5-16-73	2	57	43	24	32	42	29
19 100% solids epoxy	4-20-73	2	78	58	45	39	43	29
20 hot asphalt and glass fabric	5-23-73	2	26	31	21	27	32	33
21 tar emulsion and glass fabric	5-7-73	2	42	44	37	43	41	45
22 solvent cut epoxy	5-7-73	2	127	69	38	34	55	39
23 cold tar epoxy	4-21-73	2	30	29	40	35	39	32
24 75-mil preformed sheet	5-2-73	2	37	39	40	46		
25 70-mil preformed sheet	4-16-73	2	32	46	44	21		
approach slabs (9)		3			88	58		
approach slabs (9)	no treatment	2			60	32		

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[a] Underlining indicates areas with chloride concentrations over base levels of 18 to 46 parts per million (ppm). Divide ppm by 250 to obtain chloride concentrtions in lb/cubic yard of concrete.

ever, their applications were difficult, making it appear doubtful that the systems could be placed properly under typical field conditions when an application specialist might not be present. Actual or potential problems included critical air curing requirements, difficulties in obtaining complete coverage with bonding adhesives, inadequate provisions for sealing rough granite curb lines and difficulties in adhering protection systems. Blisters or cracks occurred in the initial pavements placed over all five systems, but it is theorized that most of the problems would not have occurred if a thicker initial course of pavement had been placed and lower pavement temperatures had been maintained.

As a group, the four cured rubber sheet systems and the single PVC polymer membrane are not recommended for further use because of their difficulty in application and high in-place costs. However, use of several of the systems would be reconsidered if the membrane systems now being specified prove to be unsatisfactory.

Summary

Evaluations have been conducted on 14 membrane systems which have been in service for periods of two to four years. The results of the field tests including chemical chloride analysis of concrete specimens (Table 3) indicate that several systems have provided complete waterproofing protection to the bridge decks. In addition, a number of other materials have prevented chloride intrusion in all areas except where proper drainage is prevented due to the ponding effect of curb lines and expansion dams. Such results suggest that the simple and inexpensive materials could be specified for sealing the major portion of a deck surface if a compatible and impervious membrane material is placed over poorly drained areas.

Field tests are now being rerun on the 14 membrane systems, and annual evaluations are beginning on 12 additional products which have now been in service through two winters of deicing chemical applications. Based upon laboratory performance and observations made during the applications, a number of the materials are expected to prove satisfactory. The materials include two of the three standard preformed membranes plus a polyurethane and a PVC polymer system. Other sophisticated systems may also prove to be satisfactory but, due to the difficulties with their application and potential for improper placement, the use of less complicated materials is recommended.

Laboratory tests and field applications have disclosed that air and vapor pressures which outgas from the concrete often result in the formation of holes or bubbles in liquid-applied coatings. The rate of pinholing or bubbling can be expected to vary with the density and moisture content of the concrete and with changes in the air temperature. The size and shape of any holes or bubbles which remain in a cured membrane will often vary with the viscosity and cure rate of each material and with individual applications. Although pinholes and bubbles may be present in a membrane, the results of laboratory tests (Table 1) indicate that substantial protection is still obtained when contrasted with the chloride intrusion which occurs in unprotected concrete. The test results also show the relationship between resistivity readings and chloride penetrations and suggest that readings well below the generally accepted standard of 500 000 Ω may be acceptable in many cases.

Experience has shown that most serious problems which occur with membrane applications are directly related to the pavement applications, or more specifically, the paving procedures and pavement design. Agencies contemplating the use of preformed membranes or thermoplastic liquid-applied systems are strongly encouraged to alter their normal procedures to fully comply with the recommendations of the membrane's manufacturer. Initial cracking and blistering of such pavement-membrane systems could be eliminated, in many cases, by reducing bituminous mix temperatures to 275 °F or lower, by placing thicker pavement courses, and by applying initial compaction effort with lightweight rollers. Wearing courses over membrane systems should be a minimum of 2 in. thick, with 3 in. preferred. If placed in more than one lift, the first course should be 1½ in. thick.

Overall experiences indicate that a number of membrane systems are capable of providing the desired protection. However, the potential for improper placement and other related problems with individual applications of even the simplest materials should be sufficient to discourage membrane usage in areas where a lack of sufficient care and attention might be anticipated.

W. P. Kilareski[1]

Epoxy Coatings for Corrosion Protection of Reinforcement Steel

REFERENCE: Kilareski, W. P., "**Epoxy Coatings for Corrosion Protection of Reinforcement Steel,**" *Chloride Corrosion of Steel in Concrete, ASTM STP 629,* D. E. Tonini and S. W. Dean, Jr., Eds., American Society for Testing and Materials, 1977, pp. 82-88.

ABSTRACT: The bridge deck deterioration problem has been attributed to the corrosion of the reinforcement steel. Epoxy coating of reinforcement steel provides a positive means of preventing salt-induced rebar corrosion. The epoxy coating process is a well-established manufacturing procedure capable of full-scale bridge deck production.

KEY WORDS: corrosion, concretes, reinforcing steels, bridge decks, deterioration, deicers, epoxy coatings

Today's highway and bridge engineers are faced with a voluminous number of bridge decks that are deteriorating at an accelerated rate. Bridge decks that are expected to perform adequately for 30 years or more have shown signs of extensive deterioration after 5 years of service. Recent studies indicate highway departments alone pay over 70 million dollars a year for the repair of bridge decks, and this expenditure is swelling at a rapid pace.

Premature deterioration, in most cases, can be attributed to the corrosion of bridge deck reinforcing steel. The corrosion process is an electrochemical mechanism by which a metal is reduced to its natural ore. The increased use of deicing materials that has taken place in the past decade is usually blamed for this condition. Sodium and calcium chlorides, in particular, enter into solution with the moisture present on bridge decks. This solution permeates the concrete deck and forms an electrolyte which is a necessary component for corrosion. As the reinforcement steel corrodes, the red rust expands in volume. This expansion in volume exerts internal forces on the concrete. When the tensile strength of the concrete

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is exceeded, cracking and spalling result. The spalling mechanisms caused by deicing chemicals can be seen in Fig. 1.

Recent findings indicate a chloride ion concentration of approximately 2 lb/yd^3 (1.18 kg/m^3) of concrete is enough to promote active corrosion of the steel.[2] Chloride tests performed on several bridges throughout the state of Pennsylvania revealed that decks with only one winter's salting already had chloride levels as high as 2 lb/yd^3(1.18 kg/m^3) of concrete at the depth of the rebar.[3] Older decks had chloride concentrations ranging from 2 lb/yd^3(1.18 kg/m^3) to as high as 15 lb/yd^3(8.85 kg/m^3) of concrete. With chloride concentrations this high in the majority of the bridges, highway departments can expect to replace or repair thousands of decks over the next five years.

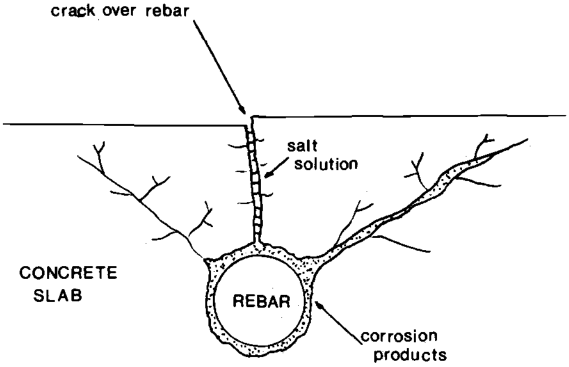

FIG. 1—*Genesis of a spall* ("Concrete Bridge Deck Durability," National Cooperative Highway Research Program, Highway Research Board, 1970).

A variety of products and an array of methods exist for the prevention and control of corrosion of bridge deck reinforcement steel. The methods include protective organic coatings, galvanization, impermeable concrete overlays, waterproof membranes, wax-sealed concrete, polymer-impregnated concrete, and cathodic protection.

The Federal Highway Administration, in their specifications IM 40-6-72 and IM 21-6-72, require that all bridge decks offered for their approval shall have some method of corrosion protection for the reinforcing steel.

One of the most positive methods of corrosion protection is the coating of the reinforcing steel with an epoxy material. Both dry powered and

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[2] Clear, K. C. and Hay, R. E., "Time-To-Corrosion of Reinforcing Steel in Concrete Slabs," Federal Highway Administration, Office of Research and Development, Vols. 1, 2, April 1973.

[3] Kilareski, W. P., "Epoxy-Coated Reinforcement Steel," Pennsylvania Department of Transportation, Bureau of Materials Testing and Research, 1973.

liquid epoxies have been used for several years as a means of corrosion protection for numerous metal products. However, it was not until 1973 that the first full-scale experiment was conducted with powdered epoxy-coated rebars for bridge deck construction.

First Epoxy-Coated Rebar Bridge Deck

During 1973, the first bridge was constructed with the use of epoxy-coated rebars. The 15-span, steel multigirder and plate girder bridge is located in West Conshohocken, a suburb of Philadelphia, on LR 1010, Traffic Route I-476. The bridge carries four lanes of traffic over the Schuylkill River, with all of the spans constructed of conventional black steel except spans No. 3,4,5, and 6 of the northbound lanes. These four spans contain the epoxy-coated reinforcement steel. The span lengths are as follows: span No. 3—119 ft (36.17 m), span No. 4—112 ft (34.04 m), span No. 5—161 ft (48.94 m), and span No. 6—112 ft (34.04 m). The bridge has a travelway of 52 ft (15.7 m).

Since 1973, Pennsylvania has constructed several more bridge decks with epoxy-coated reinforcement steel. Also, the states of Delaware, Florida, Idaho, Iowa, Kentucky, Maryland, Minnesota, Missouri, Nebraska, Nevada, New Jersey, New Mexico, North Carolina, Ohio, Oregon, Virginia, West Virginia, Wisconsin, and the District of Columbia are all participating in the Federal Highway Administration National Experimental Evaluation Program (NEEP) Project No. 16.[4]

Since five years of environmental exposure is considered necessary to provide substantial data on the performance of the epoxy-coated rebars, these particular research projects are envisaged as a long-term endeavor.

Materials

The National Bureau of Standards conducted extensive testing of 47 organic epoxy coatings.[5] Of the 47 tested, the National Bureau of Standards only recommended 4 coatings for use on reinforcement steel. The coatings were selected for their ability to show an inertness toward chloride ions, good bond srength, flexibility, and film integrity. Initially, two companies provided the epoxy material for the first bridge deck and the immediate projects. Since that time, the National Bureau of Standards has prequalified ten epoxy powders. Three of the powders are not economically feasible; thus, there are seven epoxy powders available for production.[4]

[4] "Interim Report No. 2, NEEP No. 16 Coated Reinforcing Steel for Bridge Decks," FHWA Notice N 5080.48, Federal Highway Administration, March 1976.

[5] Beeghly, H. F., Clifton, J. R., and Mathey, R. G., "Nonmetallic Coatings for Concrete Reinforcing Bars," *Coating Materials,* NBS Technical Note 768, U.S. Department of Commerce, National Bureau of Standards, April 1973.

The epoxy material and the coating process involved with the first bridge were required to meet the "Interim Specification for Epoxy Coated Reinforcing Steel" which was distributed by the Federal Highway Administration for the NEEP Project No. 16. This specification has since been modified to reflect the changes in the state of the art of the coating process. Many states have written individual specifications to meet their own needs.

Coating Process

For several years, epoxy coatings have been applied to underground pipe used by utility companies and the petroleum industry. Several companies located throughout the nation specialize in the coating of pipe and reinforcement steel.

The process of coating the rebar is a simple and efficient procedure. Initially, the rebars are individually preheated, usually by open flame, to approximately 450°F (232°C). This primary heat aids in the removal of the mill scale, rust, and grease by the shot mill which is the second step in the process. The rebar, which is cleaned to a near-white finish, can be cleaned with either shot, grit, or a combination of both. After the rebar has been cleaned, it then goes into an oven where it remains for about 30 min. During this time, the rebar temperature is brought up to a constant 450°F (232°C) or whatever temperature is specified for the application of the epoxy powder.

The reinforcing bar is then passed through a set of electrostatic spray guns that apply the charged dry epoxy powder onto the heated rebar. Normally, a coating thickness of 7 mils is applied to the rebar. The rebar then usually travels from the spray gun to a water quench bath (some epoxy rebars are air cooled). During this time, the epoxy melts, flows, and cures on the rebar.

Once the rebar has passed through the cooling tank, it is tested with a holiday detection device that electrically examines the rebar for minute cracks or pinholes in the coating. If a holiday is detected, the area is marked for easy identification during the touchup procedure. The rebars are conveyed from the holiday detection device to a rack area where the holidays are touched up. Rebar coating is repaired with epoxy mix that is compatible with the electrostatically sprayed epoxy. After the rebars are repaired, they are bundled and readied for shipment to either the bridge or steel fabricator.

Since the first project, several coating companies have made modifications to their plants in order to handle and coat the rebar with epoxy in a more efficient manner. Improvements were made to the shot mill with the intent of providing a much cleaner rebar. Conveyor systems have been improved so that the smaller-diameter, more flexible rebar could be accommodated. Several coaters have designed new coating systems that allow them to coat several rebars at a time, thus increasing production. Also, many coaters now have the capability of coating prefabricated reinforcement steel.

Fabrication

The reinforcement steel for the first bridge deck and a few of the subsequent bridge decks was fabricated after the coating process. Rebars of lengths varying from 20 to 30 ft (6.08 to 9.12 m) were bent several at a time on a hydraulic mandrel capable of bending various radii and angles of bend. With the ability to coat prefabricated rebar, the early difficulties with cracked coatings in the bend areas have been eliminated.

Construction at the Bridge Site

The reinforcing steel is handled and set in the normal construction manner with a few exceptions. The trucks are loaded and unloaded with the use of nylon slings instead of the usual steel slings to prevent the rebars from being scuffed. All supporting chairs are epoxy coated, and all rebars are tied with plastic-coated tie wire for the same reason. The epoxy coating is not damaged from workmen walking on the rebar mats during the setting of the steel and the placement of the concrete. If the rebars are cut for expansion joint construction, the ends can be repaired with the approved patching material.

In-Service Testing

Epoxy-coated rebars in several bridge decks in the NEEP No. 16 project were prewired for a resistance measurement device prior to the placement of the concrete. This device is used to measure the ability of the coating to resist the penetration of deicing chloride. If the coating should break or crack, the resistance measured across the coating will be very low. A high resistance indicates that the coating is performing adequately. This type of test is similar to the test used with various waterproofing membranes. The only difference in the test is that measurements are made with an ac Ω meter rather than the standard dc Ω meter. Initial laboratory reports indicated the results obtained with the dc Ω meter were not reproducible. Resistance values fluctuated by merely changing scales on the dc Ω meter. Tests made with the dc Ω are encouraging; however, more testing is necessary to warrant the test method.

Observations

Rebar Preparation

It is absolutely necessary for a rebar to be free of scale, dirt, and grease to ensure a proper bond between the epoxy and the steel. During the initial production run, the rebars were not preheated before entering the shot mill. Consequently, the shot was not cleaning the rebars sufficiently, and scale left on the rebar was causing excessive cracking when the rebars were bent. Usually, all cracks were propagated from the deformation valleys of the bar, indicating the unsatisfactory cleaning of the rebar in this area. The problem was corrected, and subsequent coating operations were performed without difficulties. Since the first project, many coaters have improved their ability to clean the rebar by adding additional shot mills on the production line.

Repair of Defects

Most epoxy specifications require that all damaged or exposed areas, whether they are sheared ends, cracks, cuts, or holidays, be patched promptly with an approved material. Initially, every attempt was made to obtain a 100 percent holiday-free coating; however, it was soon learned that an exorbitant amount of handwork was required to repair every deficit. Therefore, to be more realistic, only major defects or damaged areas should be repaired. Cracks larger than 0.003 in. (0.076 cm), sheared ends, and large damaged areas obviously need repair. Minute holidays and scuff marks from the fabricator's bending do not need to be touched up since the majority of the bends will be buried in the parapet area of bridge and thus are not exposed to a large amount of salt.

Recent research by the Federal Highway Administration indicates that major nicks and cuts in the epoxy coating do not lead to accelerated deterioration of the concrete. Thus, the amount of repair work can be reduced. Extra care should be taken in handling the rebars. Rough or abusive handling will scuff and cut the epoxy coating. In particular, when the rebar is bent on the steel mandrel, the coating is excessively scuffed by the various rollers needed to bend the bar.

When the bars are bundled, special cardboard protection is used to keep the steel straps from cutting the coating. Also, nylon slings are used for all loading and unloading. These extra efforts help considerably in the prevention of normal handling damage.

Conclusions

The use of epoxy-coated reinforcement steel for the prevention of corrosion-induced bridge deck deterioration has been recommended by the National Bureau of Standards and the Federal Highway Administration.

The epoxy coating procedure is an established manufacturing process capable of full-scale quality production of coated reinforcement steel for bridge decks.

Early problems such as cracked coatings and excessive touchup have been eliminated with better coating techniques and handling methods.

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and R. W. Previte[2]

A Corrosion Inhibitor Formulated with Calcium Nitrite for Use in Reinforced Concrete

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REFERENCE: Rosenberg, A. M., Gaidis, J. M., Kossivas, T. G., and Previte, R. W., "**A Corrosion Inhibitor Formulated with Calcium Nitrite for Use in Reinforced Concrete,**" *Chloride Corrosion of Steel in Concrete, ASTM STP 629,* D. E. Tonini and S. W. Dean, Jr. Eds., American Society for Testing and Materials, 1977, pp. 89-99.

ABSTRACT: The scarcity of good aggregate in Japan led to an investigation of the use of a corrosion inhibitor for reinforced concrete. Corrosion current measured on small test specimens soaked in limewater for one day with an impressed voltage of + 400 mV with respect to the saturated calomel electrode shows that the introduction of calcium nitrite diminishes the corrosion of reinforcing steel in the presence of admixed sodium chloride. Long-term tests on larger specimens partially immersed in saturated sodium chloride solution indicate that admixed calcium nitrite can prevent the open circuit potential of reinforcing steel from entering the active region. Calcium nitrite was found to enhance the strength of concrete. Calcium nitrite complies with the requirements of ASTM Specification for Chemical Admixtures for Concrete (C 494-71). The use of a calcium-based admixture is expected to sidestep the issue of alkali-aggregate reactions and subsequent deterioration of concrete.

KEY WORDS: corrosion, concretes, reinforcing steels, admixtures, alkali-aggregate reactions, chlorides, inhibitors, portland cement, strength of materials

The scarcity of good aggregate in Japan has encouraged the use of marine sand in concrete. Chlorides from the sand accelerate the corrosion of reinforcing steel in concrete. This work was initiated to develop a corrosion inhibitor for use in reinforced concrete. Calcium nitrite is an effective corrosion inhibitor which does not adversely affect the concrete.

Corrosion problems in concrete are also serious in the United States.

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Approximately $500 million is spent annually to repair bridge decks damaged by corrosion. Concrete structures such as seawalls, piers, offshore oil rigs and storage stations, prestressed and poststressed concrete, roof decks made with lightweight concrete, and concrete in which chlorides have been used also run the risk of corrosion. These corrosion problems are being attacked by many techniques such as cathodic protection, metal coatings (epoxy or zinc), nonmetallic reinforcing, nonchloride deicing salts, electro-osmotic removal of chloride, waterproof membranes, and reduction of concrete permeability (using polymers, wax, fly ash, expansive cement, or low water-cement ratio). No commercial corrosion inhibitor is being used to protect steel reinforcing in concrete in the United States.

Some laboratory studies of corrosion inhibitors in concrete have been published over the last several years [*1-3*],[3] but little is known about the effectiveness of corrosion inhibitors at high pH. All these studies showed the nitrite inhibitors to be effective in controlling corrosion in concrete; however, deleterious side effects were noted with the sodium salt. Spellman and Stratfull [*4*] explored the variables of corrosion in concrete. Of 15 methods possible for testing the corrosion of steel in concrete, they preferred the use of a partial immersion corrosion test, in which the time to active potential is determined. A recent paper by Griffin [*5*] reports the state of the art on corrosion inhibitors for reinforced concrete.

Experimental Procedure

Electrical Measurements

Equipment—Figure 1 shows the circuit diagram used for determining corrosion inhibition by calcium nitrite. The potentiostat was a Wenking Model 68FRO.5. When open-circuit potentials were measured in the Stratfull test, the steel bar and saturated calomel electode (SCE) reference were connected through a high impedance (1×10^7 Ω) voltmeter.

Specimen Preparation—Two types of specimens were used:

1. For measurement of corrosion inhibition at impressed voltage of +400 mV with respect to SCE the ASTM Test for Air Content of Hydraulic Cement Mortar (C 185-75) mortar mix was prepared with various levels of admixed sodium chloride and calcium nitrite. Air content was held to 7.0 ± 1.0 percent; flow was held to 83.0 ± 3.0 percent. Water-cement ratio was 0.73 with a local Type I cement. Mortar was placed in plastic containers [1⅝ in (41 mm) diameter by 3 in. (76 mm) high]. A No. 3 steel reinforcing bar [⅜ in. (9.5 mm) diameter], sandblasted and waxed at both ends to expose only 2 in. (50 mm) of length, was inserted in the center. Curing took place overnight at 74 ± 1 °F (23.5 ± 0.5 °C) in closed 1-gal cans con-

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[3] The italic numbers in brackets refer to the list of references appended to this paper.

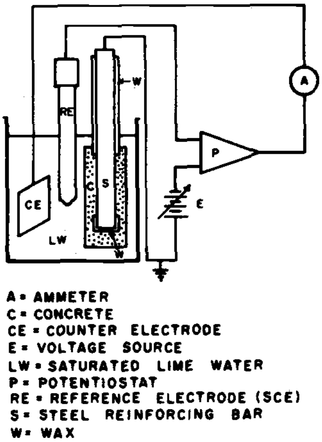

FIG. 1—*Circuit diagram for reinforcing bar corrosion experiments.*

taining an open container of water. Electrical testing was performed at 24 ± 1 h after casting.

2. Larger specimens were made for the partial immersion tests. Mortar specimens prepared as just described (but in double-sized batches) were cast in wax-coated cardboard containers measuring 2¾ by 2¾ by 10 in. (70 by 70 by 254 mm) supported by a plywood outer form. A steel reinforcing bar cut to length with an acetylene torch, ground at the end to remove slag and unevenness, was sandblasted and centrally inserted so as to be 2 in. (50 mm) above the bottom of the mold. Specimens were demolded after one day and cured.

Procedure—The smaller specimens were demolded approximately 22 h after casting and soaked in saturated limewater for 2 h before electrical testing. Open-circuit potentials were measured. The potentiostat was then set to ±400 mV versus a SCE and current was measured after it had leveled out (usually 5 to 10 min). This current is reported in Fig. 2.

The larger specimens were demolded the day after casting and cured in a 100 percent relative humidity chamber at 75 °F (23.9 °C) for seven days. Some specimens were then soaked in approximately 4 in. of saturated sodium chloride solution. Some specimens were dried at 120 °F (45.1 °C) and 0 percent relative humidity to constant weight (four days) before soaking. Open-circuit potentials were recorded versus a SCE reference three times a week. These data are reported in Fig. 3.

Effects on Physical Properties of Concrete

ASTM Specification for Chemical Admixtures for Concrete (C 494-71) Compliance—See Table 1.

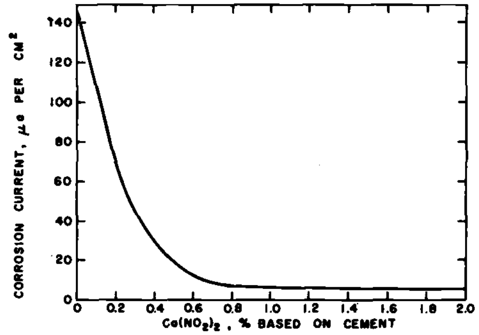

FIG. 2—*Corrosion of reinforcing steel in mortar containing calcium nitrite and 1.2 percent sodium chloride based on cement.*

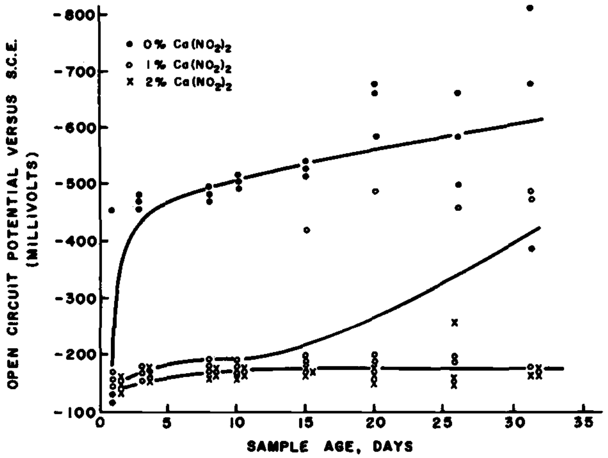

FIG. 3—*Open circuit potentials of reinforcing steel in mortar.*

Acceleration of Set—The use of chilled starting materials resulted in a concrete temperature of 60°F (15.6°C). Cement factor was 517 lb/yd^3 (306.7 kg/m^3). The concrete was cured for 70 h at 50°F (10°C), then according to ASTM Specification C 494 for the remaining period. Setting time was determined according to ASTM Test for Time of Setting of Concrete Mixtures by Penetration Resistance (C 403-70) (see Table 2 for results).

Nonacceleration of Set—A retarder (sodium gluconate) was used to nullify the acceleration of set as determined by ASTM Test C 403, modified. The modification consists of leaving the coarse aggregate out of the recipe instead of sieving it out of the concrete. (Table 3).

Strength as a Function of Addition Rate—At 517 lb of cement per cubic

TABLE 1—*ASTM Specification C 494 compliance tests.*

	Reference	Admixture	Reference	Admixture
Cement,[a] lb/yd^3 (kg/m^3)	517 (306.7)	517 (306.7)	517 (306.7)	517 (306.7)
Sand, lb/yd^3 (kg/m^3)	1389 (824.1)	1389 (824.1)	1250 (741.6)	1250 (741.6)
Gravel, lb/yd^3 (kg/m^3)	1752 (1039.5)	1752 (1039.5)	1752 (1039.5)	1752 (1039.5)
Water, lb/yd^3 (kg/m^3)	279.2 (165.6)	274 (162.6)	268.8 (159.5)	268.8 (159.5)
DARAVAIR™, fluid oz/100 lb (ml/kg)	...	...	1.1 (0.746)	1.1 (0.746)
Calcium nitrite, %	...	2	...	2
Water/cement ratio	0.54	0.53	0.52	0.52
Slump, in. (mm)	3.0 (76)	2.5 (64)	2.75 (70)	2.5 (64)
Air content, %	1.5	1.8	4.6	5.1
Unit weight, lb/ft^3 (kg/m^3)	145.5 (2330.9)	145.3 (2327.7)	140.8 (2255.6)	140.2 (2246.0)
Cement factor, lb/yd^3 (kg/m^3)	515.9 (306.1)	515.8 (306.0)	518.8 (307.8)	516.7 (306.6)
Setting time (C403)				
initial h:min.	5:21	3:25		
final h:min.	7:10	5:05		
Compressive strength, psi (kPa)				
1 day	1138 (7847)	1643 (11328)	1001 (6902)	1428 (9846)
7 days	3926 (27070)	4927 (33972)	3277 (22595)	4376 (30172)
28 days	5495 (37888)	6324 (43604)	4635 (31958)	5191 (35792)
Flexural strength, psi (kPa)				
1 day	436 (3006)	512 (3530)	403 (2779)	487 (3358)
7 days	627 (4323)	646 (4454)	528 (3641)	560 (3861)
28 days	714 (4923)	730 (5033)	685 (4723)	710 (4895)
Relative durability factor, ASTM Test for Resistance of Concrete to Rapid Freezing and Thawing (C 666-76), 300 cycles	...	...	100	107
Shrinkage, %	0.031	0.030	0.034	0.0024

[a] An equal blend of three cements by weight.

TABLE 2—*Acceleration with calcium nitrite.*[a]

			Setting Time	
Cement Brand	Admixture % by Weight of Cement	3-Day Compressive Strength, psi (kPa)	Initial h:min.	Final h:min
A	none	1525 (10515)	8:45	12:21
A	calcium nitrite	1568 (10811)	6:00	10:20
A	calcium chloride	2151 (14831)	4:20	7:30
A	calcium nitrite	1924 (13266)	3:05	6:55
A	calcium chloride	2624 (18092)	2:10	4:55
B	none	1467 (10115)	8:38	...
B	calcium nitrite	1576 (10867)	5:24	9:05
B	calcium chloride	2220 (15307)	3:16	5:00
B	calcium nitrite	2075 (14307)	3:12	5:42
B	calcium chloride	2562 (17665)	2:15	3:40

[a] Water-cement ratio was 0.56 to 0.57; slump was 4.0 ± 0.5 in. (100 ± 12 mm); air content was 1.95 ± 0.25 percent.

TABLE 3—*Setting time performance of calcium nitrite with a retarder.*

		Setting Time	
Retarder, %[a]	Calcium Nitrite %[a]	Initial, h:min.	Final, h:min.
None	none	3:50	5:35
None	2	1:25	2:05
0.100	2	3:05	4:10
0.150	2	3:55	5:50
0.175	2	5:45	6:50

[a] Based on cement.

yard (306.7 kg/m^3), strength rises as addition rate increases at 1, 4, and 28 days (Table 4).

Reaction of Calcium Nitrite with Portland Cement

Using a procedure described previously [6], calcium nitrite pickup was determined as a function of time in the presence of hydrating cement. Either of two cements (5 g) in water (100 g) containing 0.27 or 0.55 percent anhydrous calcium nitrite, after three days, removed from aqueous solution only about one eighth of the nitrite originally present.

Calorimetry

The effect of calcium nitrite on the heat of hydration was investigated by the method reported by Edwards and Angstadt [7]. The change in the

TABLE 4—*Effect of calcium nitrite on strength development.*

Admixture[a]	Addition Rate, %[b]	Compressive Strength, psi (kPa)		
		1 Day	7 Days	28 Days
None	...	1299 (8957)	3401 (23450)	5032 (34696)
Calcium nitrite	2	1615 (11135)	4538 (31290)	5732 (39522)
Calcium nitrite	3	1959 (13507)	4964 (34227)	5907 (40729)
Calcium nitrite	4	2294 (15817)	5337 (36799)	6383 (44011)
Calcium nitrite	5	2367 (16320)	5324 (36709)	6493 (44769)

[a] Water-cement ratio was 0.58 ± 0.01; slump was 4.0 ± 0.25 in. (100 ± 6 mm); air content was 1.55 ± 0.25 percent.

[b] Based on cement.

heat of hydration was found to be similar to that caused by an accelerator (Fig. 4).

Results and Discussion

It has been well established that corrosion is an electrochemical phenomenon [*8*]. Iron goes into solution as ferrous ion at the anode as oxygen is reduced to hydroxide at the cathode. Subsequent precipitation and further oxidation produce various forms of iron oxide.

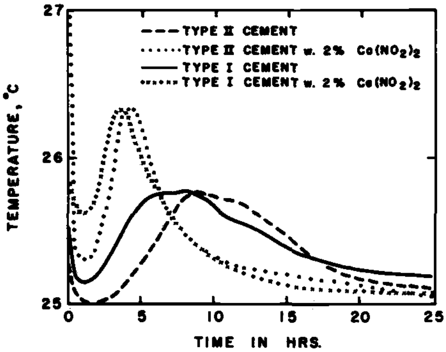

FIG. 4—*The hydration profile of portland cement with added calcium nitrite.*

It is instructive to follow the course of the corrosion reaction by conducting it in a gel containing indicators, for example, phenolphthalein and a ferricyanide. The phenolphthalein turns red at the cathode where hydroxyl ion is produced, and the ferricyanide produces a blue precipitate with ferrous ion at the anode. In general, one or two localized anodes develop the blue color, while the rest of the iron is involved in the cathodic reaction.

A more quantitative approach to the question of corrosion is supplied by measurement of the corrosion current at a fixed voltage. While potentials from +100 to +700 mV with respect to saturated calomel could be used, we chose +400 mV for our work (Fig. 2). The specimens were entirely submerged in saturated limewater and were tested 24 h after preparation. We experienced difficulties in conducting this test at later ages.

Therefore, we went to larger specimens and more aggressive soaking media. Partial immersion resulted in a varying concentration of chloride ion and oxygen along the reinforcing bar such that optimum conditions existed somewhere for corrosion initiation. This method is an extremely accelerated test compared to field conditions. In general, the higher the calcium nitrite dosage rate, the greater the protection that can be conferred (Fig. 3).

Steel is used to reinforce concrete because, in general, steel does not corrode in concrete [*9*]. Iron acquires passivity at pH 12.5 with great ease because only a small amount of oxygen is required to exceed the critical current [*10*]. In more pictorial terms, the insolubility of the protective oxide film at pH 12.5 enables a small amount of corrosion to plug whatever gaps exist in the film.

Certain ions, among them the chloride ion, penetrate the passive film. Transient iron chloride complexes, such as those found by Foley [*11*], then may transport ferrous ions some distance from the anode. The formation of iron oxide precipitate away from the anode allows corrosion to continue.

The nitrite ion prevents the migration of ferrous ion away from anodic areas. Iron metal does not react with the nitrite ion [*12*], at least not in measureable quantities. Ferric ion and ferric oxide are likewise unaffected by nitrite ion. However, ferrous hydroxide and ferrous ions react to form gaseous nitric oxide and precipitate a hydrated iron (III) oxide. When we allowed an iron nail or a section of reinforcing bar to stand in an agar solution containing 2 percent calcium nitrite and 2 percent sodium chloride with phenolphthalein and potassium ferricyanide, we observed the occurrence of a small amount of corrosion (indicated by a blue precipitate) which stopped quickly, being followed by the appearance of small gas bubbles. These bubbles, presumably nitric oxide, which occurred only at the anodes, are a result of the interaction of nitrite ions with ferrous ions. The ferric oxide precipitate, which also forms, stifles the corrosion process, according to the third equation in Fig. 5. In Fig. 5, the first anodic reaction represents the normal inertness of steel in concrete, showing ferrous ions rapidly converted to ferrous hydroxide and stifling further corrosion. In the presence of chloride ion (second equation), however, passive film breakdown and transient complex formation contribute to ferrous hydroxide deposition at some farther distance, and corrosion becomes an ongoing process.

ANODE REACTIONS IN CONCRETE

$$Fe^{++} + 2\,OH^- \xrightarrow{k_1} FeO \cdot (H_2O)_x$$

$$Fe^{++} + Cl^- \xrightarrow{k_2} [FeCl\ Complex]^+ + OH^- \longrightarrow FeO \cdot (H_2O)_x + Cl^-$$

$$2Fe^{++} + 2OH^- + 2NO_2^- \xrightarrow{k_3} 2NO + Fe_2O_3 + H_2O$$

$$-\frac{d[Fe^{++}]}{dt} = k_1[Fe^{++}][OH^-]^2 + k_2[Fe^{++}][Cl^-] + k_3[Fe^{++}]^2[NO_2^-]^2[OH^-]^2$$

FIG. 5—*Anode reactions in concrete: the mechanism of corrosion inhibition by calcium nitrite.*

In the presence of nitrite (third equation), the competition for ferrous ions is shifted toward stifling corrosion because of the reaction of nitrite with ferrous ion to produce ferric oxide. The calcium nitrite inhibition is a rapid oxidation and precipitation of ferrous ion which maintains a relatively impervious film. The consumption of ferrous ions is then illustrated (in the last equation of Fig. 5) as the sum of three competing reactions.

According to this mechanism, when the nitrite ion concentration is below the level required to maintain passivity, the chloride ion-accelerated corrosion proceeds, depleting the small amount of nitrite present. When the nitrite ion concentration is large enough, the inhibition reaction proceeds rapidly and closes off the anode with a layer of ferric oxide; no further nitrite is consumed. Cohen and Pyke [*12*] found that, for levels of nitrite sufficient to cause passivity, only an imperceptible amount of nitrite is needed to form an oxide film on the iron and stifle further corrosion.

The literature of corrosion inhibitors contains references to dangerous corrosion reactions brought about by low inhibitor concentrations. For example, a storage tank or piping may be treated with inhibitor additions at a level too low to give complete protection. In such a case, corrosion may begin at a weak spot and be accelerated by the cathodic reaction common to many inhibitors. As it is consumed, inhibitor will be added to maintain this too-low level, and localized corrosion will eventually lead to a leak, or, in other words, a failure.

The failure of reinforced concrete is a result of disruption of the concrete which follows from corrosion product buildup. It has already been shown that, when the nitrite concentration falls below that required for total protection, partial protection is conferred until the nitrite is consumed (Fig. 2). At a low pH, in the absence of air, with aggressive concentrations of chloride and low concentrations of nitrite, a cathodic reaction will take place. Cohen and Pyke [*12*] demonstrated the formation of ammonia in a cathodic reduction experiment. This cathodic reaction requires three unusual conditions in order to occur in concrete:

1. The amount of nitrite ion must be so low that the anode reaction cannot be stifled.

2. The concentration of oxygen in solution must be very low.
3. Most unlikely of all, the pH of the system must be low.

Thus, low concentrations of calcium nitrite may not effectively close off the anode to further reaction, but will not accelerate corrosion for even a short time in the concrete system.

Sodium salts may cause expansion of concrete when certain aggregates are used. Deterioration of concrete in pavements, bridges, and other structures due to expansive reactions between the aggregates in the concrete and the alkalies in the system has occurred throughout the United States and elsewhere. The most common cause is the alkali-silica reaction.

According to the Powers-Steinour [*13*] model, reactive siliceous material in the presence of alkali and calcium hydroxide from cement forms either a solid nonexpansive calcium-alkali-silica complex or a solid alkali-silica complex which can expand by imbibition of water. The relative concentrations of the calcium hydroxide and alkali and the available surface of reactive material determine which material will form. When the alkali concentration is low enough (below 0.6 percent) the initial product of the reaction is nonexpansive; when the alkali concentration is higher, the initial product of reaction is the expansive one [*14*].

Calcium nitrite introduces more calcium ions into the system and, according to the Powers-Steinour model, should reduce the expansive tendency of sodium ion which enters the system from sea sand or road deicing salts. It would appear unwise to add sodium ions in the corrosion inhibitor. Other expansive reactions are known with carbonate aggregates. As yet, these have not been studied.

Table 1 shows calcium nitrite at 2 percent to be in compliance with ASTM Specification C 494, standard specification for chemical admixtures for concrete. Calcium nitrite seems to have little effect on the air content and slump. The setting time is shortened, and the 28-day strength is increased by approximately 15 percent. Table 4 demonstrates the effect of calcium nitrite at higher addition rates. The strength of the concrete continues to rise with increasing addition rate. The introduction of calcium nitrite to concrete may be useful not only as a corrosion inhibitor but also as a strength enhancer. As can be seen in Table 3, the acceleration of set caused by the introduction of the nitrite ion, if not required, can be offset by the use of a small amount of conventional retarder.

Our smaller specimens were made up with sodium chloride at a level in the mortar that would result in 4.2 lb/yd^3 of concrete (2.5 kg/m^3) which is the level that would result from the use of wet marine sand. We have seen that calcium nitrite offers adequate protection in this case. Spellman and Stratfull [*15*] observed maximum chloride ion concentrations at 1, 2, and 3 in. (25, 50, and 75 mm) below the surface of 12.5, 4.77, and 1.07 lb/yd^3 (7.4, 2.8, and 0.63 kg/m^3). Thus it appears that if adequate attention is paid to (*a*) cement content, (*b*) water-cement ratio, (*c*) thickness of cover, and (*d*) concrete permeability, calcium nitrite can offer substantial corrosion protection.

Conclusions

Although corrosion inhibitors have been used in industry for many years, little is known about their effectiveness in concrete.

There are two reasons for this.

1. Up until now, no inhibitor has provided a viable alternative to other means of corrosion control in concrete.
2. New admixtures must pass rigid performance criteria before being approved for use in concrete.

We have found in the laboratory that calcium nitrite can be used as an effective means of combating corrosion, that it passes the requirements of ASTM Specification C 494, and that it improves the strength of the resulting concrete.

Acknowledgments

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We are grateful to Dr. Jose Ginér and Professor R. T. Foley for their help in unraveling the complexity surrounding that mechanism through which calcium nitrite protects steel in concrete.

References

[*1*] Gouda, V. K. and Monfore, G. E., *Journal of the Portland Cement Association Research and Development Laboratories*, Vol. 7, No. 3, 1965, pp. 24–31.

[*2*] Treadaway, K. W. J. and Russell, A. D., *Highways and Public Works*, Vol. 36, 1968, pp. 19–21, 40–41.

[*3*] Craig, R. J. and Wood, L. E., *Highway Research Record*, No. 328, 1970, pp. 77–88.

[*4*] Spellman, D. L. and Stratfull, R. F., *Highway Research Record*, No. 423, 1973, pp.27–45.

[*5*] Griffin, D. F., *American Concrete Institute Publication*, No. SP-49, 1975, pp. 95–102.

[*6*] Rosenberg, A. M., *Journal of the American Concrete Institute*, Vol. 61, No. 10, 1964, p. 1268.

[*7*] Edwards, G. C. and Angstadt, R. L., *Journal of Applied Chemistry*, Vol. 16, 1966, p. 166.

[*8*] Uhlig, H. H., *Corrosion and Corrosion Control*, Wiley, New York, 1971.

[*9*] Evans, U. R., *The Corrosion and Oxidation of Metals*, Edward Arnold Publishers, Ltd., London, 1960, pp. 302–308.

[*10*] Pourbaix, M., *Atlas of Electrochemical Equilibria*, Pergamon, Oxford, 1966, p. 315.

[*11*] Foley, R. T., *Journal of the Electrochemical Society*, Vol. 11, No. 22, 1975, p. 1493.

[*12*] Cohen, M. and Pyke, R., *Journal of the Electrochemical Society*, Vol. 93, No. 63, 1948.

[*13*] Powers, T. C. and Steinour, H. H., *Journal of the American Concrete Institute*, Vol. 51, No. 6, 1955, pp. 497, 785.

[*14*] Stanton, T. E., Jr., *Engineering News-Record*, Vol. 124, No. 5, 1940, p. 59.

[*15*] Spellman, D. L. and Stratfull, R. F., *Highway Research Record*, No. 328, 1970, pp. 38–49.

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The Use of Concrete Polymer Materials for Bridge Deck Applications

REFERENCE: Kukacka, L. E., "**The Use of Concrete Polymer Materials for Bridge Deck Applications,**" *Chloride Corrosion of Steel in Concrete, ASTM STP 629,* D. E. Tonini and S. W. Dean, Jr., Eds., American Society for Testing and Materials, 1977, pp. 100–109.

ABSTRACT: The rapid deterioration of concrete bridge decks as a result of the increased use of deicing salts is one of the most severe problems facing the highway industry today. One possible solution to the problem is the use of concrete polymer materials. The materials of prime interest are polymer-impregnated concrete and polymer concrete, both of which have excellent durability and strength properties. Three potential applications (repair of deteriorated bridge decks, polymer impregnation of new bridge deck surfaces, and full impregnation of precast deck panels) have been studied in laboratory and field tests, and the results have been encouraging. These results and economic considerations are described in the paper. The work in each of the areas is currently being implemented by the Federal Highway Administration.

KEY WORDS: corrosion, concretes, reinforcing steels, bridge decks, chlorides, composite materials, durability, impregnating, maintenance, polymerization, strength of materials

Deterioration of concrete bridge decks is caused by deicing salts, corrosion of reinforcing steel, freeze-thaw deterioration, spalling, and heavy traffic loads. This represents the most severe problem facing the highway industry today. In addition to being costly, bridge deck maintenance is difficult to perform under hazardous traffic conditions and presents delays and safety hazards to the traveling public because of lane closures, detours, traffic jams, and the increased risk of accidents. One approach to the problem is the use of concrete polymer materials in new highway construction and for repair work. The materials of prime interest are

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polymer-impregnated concrete (PIC) and polymer concrete (PC), both of which have excellent durability and strength properties.

PIC consists of a precast or cast-in-place portland cement concrete impregnated with a monomer system that is subsequently polymerized *in situ*. The polymer tends to fill the porous void volume of the concrete, which results in significant improvement in strength and durability properties.

PC consists of an aggregate mixed with a monomer or resin that is subsequently polymerized in place.

A third type of material, which represents a further development of PIC, is being utilized as a means of preventing chloride intrusion into cast-in-place bridge decks. This is a precast concrete that has been partially impregnated to a finite depth with a monomer that is subsequently polymerized. A typical section of partially impregnated highway concrete is shown in Fig. 1.

Properties of Concrete Polymer Materials

PIC is the most highly developed of the concrete polymer composites, and the greatest improvements in structural and durability properties have been obtained with this material. For a concrete mix that produces specimens with a compressive strength of 5000 psi (35.2 MN/m^2), values > 20 000 psi (141 MN/m^2) are generally obtained after impregnation. Design values for PIC that cover the range of monomer systems used and many types of concrete have been published [*1*].[2] These values are as follows: compression 15 000 psi (105.6 MN/m^2), direct tension 1000 psi (70.4 MN/m^2), modulus of rupture 1300 psi (9.1 MN/m^2), shear 750 psi (5.3 MN/m^2), modulus of elasticity 6×10^6 psi (4.2×10^4 MN/m^2), and Poisson's ratio 0.2.

Equally significant improvements in durability have been obtained. Resistance to abrasion and cavitation are enhanced. The water absorption is reduced by >99 percent, and the resistance to chemical attack and freezing and thawing is enormously improved.

PIC is relatively impermeable to chlorides, and its potential for preventing reinforcing steel corrosion and surface scaling has been demonstrated. After 267 daily salt applications, the maximum chloride concentration found at a depth of 1 in. (2.54 cm) was essentially zero [*2*].

Partially impregnated concrete is a variation of PIC which is designed for durability rather than high strength. This permits a saving in the amount of monomer and in ease of impregnation as compared with fully impregnated concrete. Laboratory tests have indicated that a penetration

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[2] The italic numbers in brackets refer to the list of references appended to this paper.

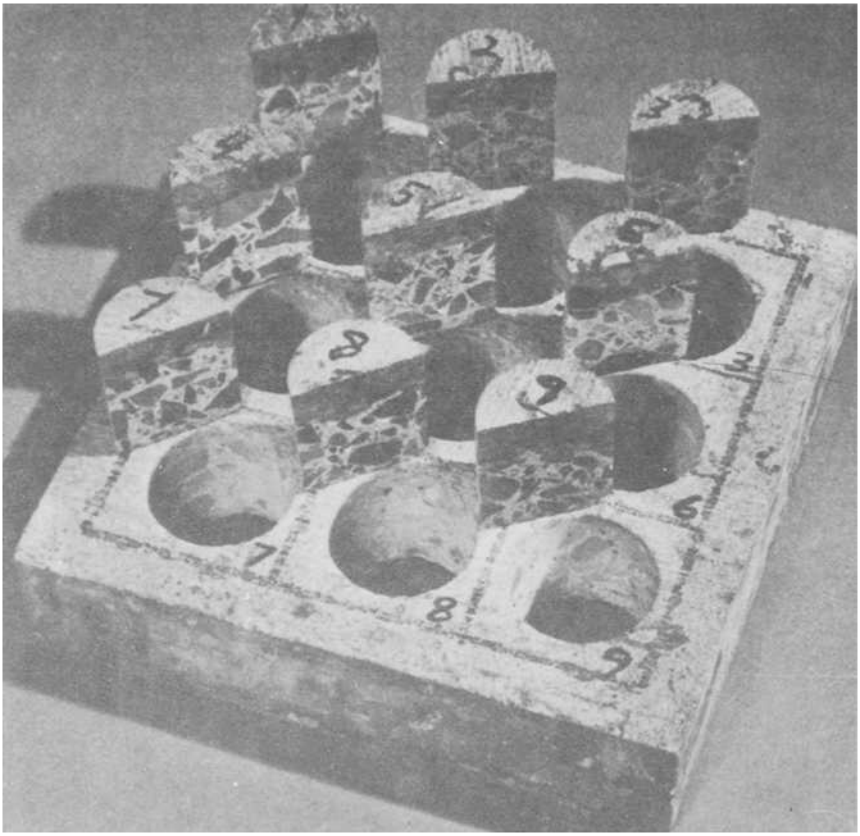

FIG. 1—*Cross sections of core specimens taken from a partially impregnated concrete slab.*

depth of 1 in. (2.54 cm) is adequate to prevent chloride penetration into the concrete [*2*]. Good resistance to abrasion and scaling is also obtained.

Due to its high strength, durability, and rapid curing rate at ambient temperatures, PC is being used on highways and bridges as a rapid patching material. Its potential for use as an overlay material is being evaluated. The material consists of a monomer and an aggregate. It is mixed and placed using techniques similar to those used for portland cement concrete [*3*]. Full strength is attained immediately after the polymerization reaction is completed. Depending upon the concentrations of promoter and initiator and the ambient temperature, this time can vary from a few minutes to approximately 4 h. Compressive strengths up to 20 000 psi (141 MN/m^2) have been obtained when a methyl methacrylate-trimethylolpropane trimethacrylate monomer mixture was used in conjunction with a high-quality aggregate [*1*]. Other structural property values include a tensile splitting strength of 1400 psi (9.9 MN/m^2), modulus of elasticity of 5.3 × 10^6 psi (3.7 × 10^4 MN/m^2), and a Poisson's ratio of 0.23. The creep is less than that of a high-quality concrete.

The durability properties of PC are equally impressive. Water absorptions of <1 percent are normally obtained, and a high resistance to freeze-thaw damage is apparent.

Bridge Deck Applications

Each of the materials just described are being evaluated to determine their applicability to highway structures. These studies are summarized in the following sections.

Precast Bridge Deck Panels

A program studying the use of precast prestressed PIC panels for bridge decking was conducted by the Federal Highway Administration (FHWA), Bureau of Reclamation (USBR), and the Prestressed Concrete Institute. This application of PIC was chosen as one possible solution to the problem of bridge deck deterioration. However, in addition to utilizing the resistance of PIC to chemical and freeze-thaw attack, the bridge decking system was structurally designed to take advantage of the strength and other physical properties of PIC.

A series of prestressed panels 16 ft long by 4 ft wide by 0.5 ft thick (4.9 by 1.2 by 0.15 m) were impregnated and tested to determine their static and fatigue behavior on a simulated bridge girder configuration. The results of the tests showed that the panels met the design requirements.

An estimate of the cost of using PIC for precast bridge deck panels was made. On the basis of the assumptions used, a cost of \$9.22/ft^2 (\$99/m^2) was calculated for the completed deck. The estimate included fabrication, polymerization, erection, overhead, and profit. The cost of girder modifications to accommodate the precast panels was also included. Costs for components such as girders, diaphragms, curbs, and rails were not included [*1*].

The average 1973 cost in the state of Colorado for a cast-in-place bridge deck with a waterproofing membrane and asphalt overlay was \$5.05/ft^2 (\$54.35/m^2) [*1*]. Bridge engineers have stated that the cost differential is acceptable if the product is available. Typical bridge deck service life is only 15 years, and major maintenance is often required as early as 2 years after construction. The cost of this maintenance possibly exceeds the increased initial cost of PIC.

Partially Impregnated Concrete

The USBR [*4*] and the University of Texas [*5*] have developed methods for the partial impregnation of new bridge deck surfaces. A penetration depth of 1 in. (2.54 cm) appears adequate to prevent chloride penetration into the concrete and subsequent corrosion of the reinforcing steel. The USBR work culminated in October 1974 with the field treatment of a full-size bridge deck in the Denver area. Since that time, full-size bridges have been treated in Idaho, California, and Texas. Plans have been formulated for the impregnation of two others in Texas [*6*] and one in West Virginia.

The partial impregnation process consists of drying the concrete, cooling it, applying monomer to the surface, and allowing it to soak into the concrete, and heating the concrete to cure the monomer [*4*].

Detailed procedures used for the treatment of a bridge in Texas have been published. The bridge, which was 17 years old and in good condition, was treated in the following manner [*6*]. A thin (0.25 in. (6.35 mm)) layer of sand was applied to the bridge prior to drying. With the use of an insulated enclosure and hot air, the deck was dried for approximately 6 h after the average surface temperature reached 260°F (127°C). The concrete was then allowed to cool overnight to 90°F (32°C).

A monomer mixture consisting of 100 parts by weight of methyl methacrylate (MMA), 5 parts trimethylolpropane trimethacrylate (TMPTMA), and 0.5 parts of a polymerization initiator azobis isobutyronitrile (AIBN) was used for impregnation.

The monomer was applied to the concrete at rates ranging from 0.6 lb/ft^2 (2.9 kg/m^2) to 1.1 lb/ft^2 (5.4 kg/m^2). A typical method for applying the monomer is shown in Fig. 2. In order to guard against premature polymerization, the test area was shaded during the soaking period which lasted 5 h.

The monomer was polymerized using steam which was injected into the enclosure. During the curing operation, the maximum surface temperature attained was 194°F (90°C). The entire curing operation required 2.5 h.

After curing, the sand layer was brushed off (Fig. 3). Cores taken from the deck indicated that the depth of impregnation ranged from 1 to 1.9 in. (2.54 to 4.8 cm), with the greatest depth occuring in the area where the largest amount of monomer was applied.

The cost for polymer impregnation may be estimated from bids recently received by the Texas State Department of Highways and Public Transportation [*6*]. One bridge is 751 ft (229 m) long with a 64-ft (19-m) roadway. The bids for this work ranged from \$5 to \$15/yd^2 (\$6 to \$17.94/m^2). Bids for the second bridge (207 by 42 ft) (63 by 13 m) were higher. Prices ranged from \$10 to \$50/yd^2 (\$11.96 to \$59.80/m^2).

FIG. 2—*Application of monomer during the partial impregnation of a cast-in-place bridge deck.*

FIG. 3—*Removal of sand layer after the partial impregnation of a cast-in-place bridge deck.*

Polymer Impregnation of Deteriorated Concrete

Methods for using monomer impregnation in repairing highly deteriorated and delaminated concrete have been demonstrated in the field, and the work has reached the implementation stage [7].

Highly deteriorated concrete can be readily impregnated with monomer and polymerized to form a composite with strength at least equal to that of the original concrete and with good bonding to the adjacent material. Since the presence of moisture prevents good bonding to the aggregate, it is essential to dry the concrete to a moisture level of <3 percent prior to impregnation. This can be accomplished by blowing hot air across the top surface or, if complete closing of the structure is not possible, through and on the underside of the deck.

Prior to impregnation, the underside of the deteriorated bridge deck must be sealed in order to prevent loss of monomer through cracks. This has been accomplished by applying a coating of standard polyester resin on the underside surfaces containing cracks. The resin can be applied with a brush or a paint roller. Curing is completed within 10 min.

Impregnation with a 95 percent by weight MMA-5 percent by weight TMPTMA mixture has been performed by using a ponding technique in conjunction with a $\sim$0.5-in. (1.3-cm) layer of dried sand which served as a wick and an evaporation barrier. A four-day soaking period was sufficient to fully saturate a 6-in. (15.2-cm)-thick highly deteriorated structural deck.

Polymerization of the monomer was easily accomplished by heating the deck to 160°F (71 °C) with hot air. Overnight heating was adequate to ensure complete polymerization. The deck can be open to traffic during this operation since vibration during polymerization does not affect the properties of the composites. Core specimens taken from reconstituted decks have had compressive strengths of approximately 5000 psi (35.2 MN/m^2). A typical core is shown in Fig. 4.

Since only a limited number of small-scale field tests have been performed, only the material costs associated with drying, impregnation, and polymerization processes have been determined. Assuming a monomer loading of 15 percent by weight, the materials required to reconstitute a highly deteriorated 6-in. (15.2-cm)-thick deck will cost approximately $6.50/ft^2 ($70/m^2). Although this cost is high, considerable savings in manpower costs should be possible because the deteriorated deck does not have to be stripped and replaced. This will shorten the repair period. Traffic control costs would also be reduced since vehicles can be maintained on a portion of the structure during the repair process. Therefore, the use of PIC, with its high-strength, fast-curing characteristics and good durability, can result in minimum traffic delays and improved safety and,

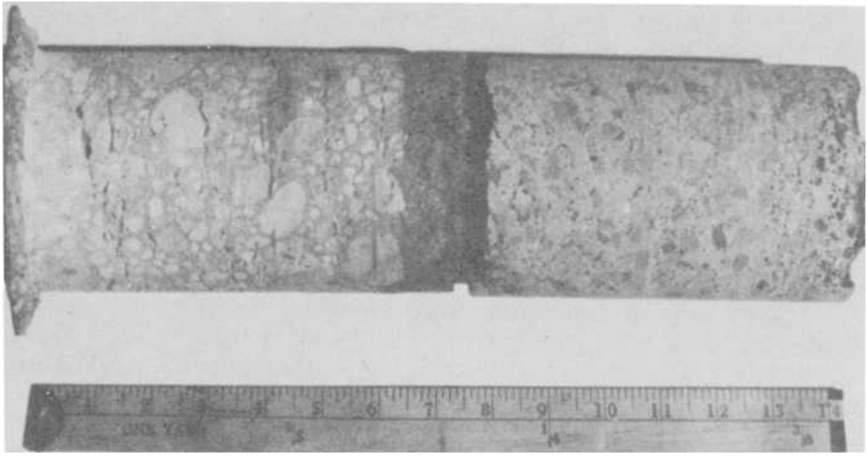

FIG. 4—*Core from deteriorated bridge deck after monomer impregnation.*

in some cases, may eliminate the necessity of constructing expensive detours. In such cases, the material cost becomes insignificant.

Polymer Concrete Patching Materials

Because the material can be cast in place, cures quickly at ambient temperature, and develops high strength, PC appears to be suitable for use in the repair of highway structures where traffic conditions allow closing of the area for only a few hours.

Field testing of PC materials containing MMA-TMPTMA and polyester-styrene has been in progress for approximately five years, and to date no detrimental effects have been noted. The most severe test performed to date was the repair of two large holes (measuring 30 by 2 by 1.25 ft (9 by 0.6 by 0.4 m) and 20 by 2 by 1.25 ft (6 by 0.6 by 0.4 m)) on the Major Deegan Expressway in New York City [*8*]. To date, no significant deterioration is apparent, and it is hoped that the use of PC has resulted in permanent repair of the bridge deck.

When compared with the material cost of portland cement concrete, PC appears expensive. Earlier estimates [*9*] ranged between \$170 and \$300/yd^3 (\$222 and \$392/m^3). In the New York City work, the cost was \$260/yd^3 (\$340/m^3). Mixing and placement costs were \$248/yd^3 (\$324/m^3). The PC material cost represented only 4 percent of the total project cost which is insignificant when repairs on major arterial highways are performed. Other concretes that cure just as rapidly but which appear to be less durable than PC would have similar installation costs but higher future maintenance costs.

The repairs on the Major Deegan Expressway were completed within eight working days, and all the work was confined to nonpeak hours (10 a.m. to 3 p.m.). At least one traffic lane on a roadway was kept open at all times. This, in conjunction with a detour, resulted in minimum inconvenience to the public.

Conclusion

The deterioration of concrete bridge decks and pavements presents many highway organizations with major problems of providing safe and satisfactory riding surfaces. Concrete polymer materials, with their excellent durability and strength properties, are beginning to be applied to highway construction and maintenance and offer potential benefits of an increase in service life and a reduction in the costs, safety hazards, and inconveniences in performing maintenance and repair work.

PIC is highly resistant to freeze-thaw damage and water penetration and provides protection from deicing salt penetration and resultant corrosion of reinforcing steel. A precast, prestressed PIC bridge deck system has been developed that incorporates the advantages of a precast, prestressed system for rapid construction, strength, and durability.

A cost estimate for the PIC precast bridge deck system indicated that the initial cost may be as much as 83 percent greater than the cost of a membrane and asphalt-topped cast-in-place deck. Considering long-term costs of both construction and maintenance, the economics of using PIC becomes more favorable. This is particularly true for structures in which the concrete may not be fully satisfactory because of conditions causing rapid deterioration and frequent repair.

A surface-impregnation technique has been developed as a method of protecting newly constructed concrete bridge decks from damage resulting from the use of deicing salts. A method and equipment capable of impregnating a concrete deck with polymer to a depth of at least 1 in. (2.54 cm) have been developed. Three decks have already been impregnated, and several others are planned.

The techniques for repairing highly deteriorated and delaminated bridge decks by monomer impregnation have been demonstrated in field tests in which a 6-in. (15.2-cm)-thick section of deck was impregnated. Cores taken from the reconstituted deck had compressive strengths and water absorptions of 5000 psi (35.3 MN/m^2) and 1.6 percent, respectively. Based on these results, plans to repair an entire deck are being formulated by the New York State Department of Transportation.

Field evaluation of the use of polymer concrete as a means of rapidly filling holes in major arterial highway bridges has been in progress in New York City for 28 months. No signs of deterioration are apparent.

The New York State Department of Transportation plans to make further use of this technique.

References

[1] Cowan, W. C. et al, *Introduction to Concrete Polymer Materials, Supplement No. 1,* FHWA-RD-75-527, Federal Highway Administration, Nov. 1975.

[2] Clear, K. C. and Hay, R. E., *Time to Corrosion of Reinforcing Steel in Concrete Slabs, Vol. 1, Effect of Mix Design and Construction Parameters,* FHWA-RD-73-32, Federal Highway Administration, April 1973.

[3] Kukacka, L. E. et al., *Introduction to Concrete Polymer Materials,* BNL 19525 and FHWA-RD-75-507, Brookhaven National Laboratory and Federal Highway Administration, Dec. 1974.

[4] Smoak, W. G., *Partial Impregnation of New Concrete Bridge Deck Surfaces, Interim Users' Manual of Procedures and Specifications,* FHWA-RD-75-72, Federal Highway Administration, June 1975.

[5] Fowler, D. W., Houston, J. T., and Paul, D. R., *Polymer-Impregnated Concrete for Highway Applications,* Research Report 114-1, The University of Texas at Austin, Feb. 1973.

[6] Webster, R., Fowler, D. W., and Paul, D. R., *Partial Polymer Impregnation of Center Point Road Bridge,* Research Report 114-5, The University of Texas at Austin, Jan. 1976.

[7] Kukacka, L. E., Fontana, J., and Steinberg, M., *Concrete-Polymer Materials for Highway Applications, Final Report,* BNL 50462 and FHWA-RD-75-86, Brookhaven National Laboratory and Federal Highway Administration, June 1975.

[8] Kukacka, L. E., Mediatore, R., Fontana, J., Steinberg, M., and Levine, A., *The Use of Polymer-Concrete for Bridge Deck Repairs on the Major Deegan Expressway,* BNL 19672 and FHWA-RD-75-513, Brookhaven National Laboratory and Federal Highway Administration, Jan. 1975.

[9] Kukacka, L. E., Fontana, J., and Steinberg, M., *Polymer Concrete,* Transportation Research Record, 542, 1975, pp. 20–28.

G. W. Steele[1] *and J. M. Judy*[1]

Polymer-Modified Concretes in Bridge Deck Overlay Systems

REFERENCE: Steele, G. W. and Judy, J. M., "**Polymer-Modified Concretes in Bridge Deck Overlay Systems**," *Chloride Corrosion of Steel in Concrete, ASTM STP 629,* D. E. Tonini and S. W. Dean, Jr., Eds., American Society for Testing and Materials, 1977, pp. 110–115.

ABSTRACT: Premature failure of portland cement concrete bridge decks is of national concern, particularly in those areas where chloride residues accumulate in the concrete from winter deicing operations. West Virginia's research and field usage indicates that the application of a realtively thin (1 to 2-in.) overlay of portland cement concrete or mortar containing a latex modifier may result in a significant increase in the useful life of bridge decks. Eighteen structures, including both new construction and renovation of existing decks, have received the latex-modified concrete overlay. Results to date have been most encouraging.

KEY WORDS: corrosion, concretes, reinforcing steels, bridge decks, latex, deterioration

The performance of portland cement concrete bridge decks continues to be a major concern, having been identified as one of the more serious problems in the construction and maintenance of the highway system. Much of the bridge deck deterioration seems to be attributable to corrosion of reinforcing steel from the action of chloride residues resulting from deicing operations [*1–3*].[2]

Many systems are currently under evaluation in an effort to alleviate the problem. Such systems include waterproofing membranes, intended to intercept the deicing chemical before it can enter the concrete; epoxy coatings on the reinforcing steel to isolate it from the chloride ions; impregnating the hardened concrete with polymers or wax to preclude the movement of moisture through the concrete; and the installation of

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[2] The italic numbers in brackets refer to the list of references appended to this paper.

cathodic protection to neutralize the effect of the chlorides. To date, trial use of these methods indicates varying degrees of success.

West Virginia's research and field usage has indicated that the application of a relatively thin (1 to 2-in.) overlay of portland cement concrete or mortar containing a latex modifier may result in a significant increase in the useful life of bridge decks [*4–6*]. The formulated latex admixture used to date has been a polymeric emulsion in water to which all stabilizers have been added at the point of manufacture. The polymer type is styrene butadiene. Using the normal precautions recommended by the manufacturer, no difficulties have been encountered in storage or handling of the materials.

Procedure

The first latex overlay was placed in West Virginia in 1961, using Department forces and conventional equipment under the supervision of Dow Chemical Company personnel [*4*]. Performance of this overlay was considered very good over the years.

Subsequently, 18 structures have received a latex-modified concrete overlay since 1970, with several more scheduled. These structures include both new construction and renovation of existing decks.

Mixing of the modified concrete under the latter program has been done with volumetric proportioning, continuous mixing equipment, although the specifications would also allow the use of stationary batch type mixers [*7–8*].

In addition to the latex modifier, the materials used include standard Type I portland cement, fine aggregate, coarse aggregate, and water. Typical properties of the component materials are shown in Table 1. The materials are proportioned to a water-cement ratio within the range of 0.35 to 0.40, and the air content may not exceed 10 percent. The latex solids-cement ratio is porportioned at 0.15 to 1. For mortar, the sand-cement ratio is proportioned at 3.25 to 1, dry weight, and for concrete, the sand-coarse aggregate-cement ratio is set at 2.5 to 2 to 1, dry weight. Latex-modified concrete is used only where depths of placement will not be less than 1¼ in. (32 mm).

Prior to applying the latex-modified wearing course, all unsound concrete is removed, including concrete around reinforcing steel which contains chlorides at or above the threshold of corrosion. The surface to be covered is uniformly sand blasted to remove any soft surface mortar. Final cleaning immediately before placement is done by air or water blasting.

Machine finishing is required, with the machine traveling on rigidly supported rails. The secondary screed may be either a vibrating or oscillating type, or a rotating cylindrical drum not to exceed 48 in. in length.

TABLE 1—*Component characteristics and cost data.*

Portland cement, Type I	
compressive strength, 7 days, psi	3400 to 4500
compressive strength, 7 days MPa	23.4 to 31.0
Latex	Dow SM 100 Modifier A
Fine aggregate	
type	river sand
soundness (sodium sulphate), %	5
Coarse aggregate	
type	limestone
size	AASHTO No. 8
Los Angeles abrasion	26
soundness (sodium sulphate), %	5
unit cost, yd^2	\$24.00 to \$33.00
unit cost, m^2 (exclusive of deck preparation)	\$20.00 to \$27.50

Some hand finishing with a wood float may be required to obtain a tight, uniform surface. When a tight, uniform surface has been achieved, it is usually textured by brooming to provide a gritty surface. Grooving is also permitted. Timing is critical in texturing. The surface will not hold a texture applied while the material is still fluid. However, if texturing is delayed too long, a plastic film will form on the surface which interferes with the texturing operation.

The overlay is cured with wet burlap for 24 h.

Discussion

A comprehensive evaluation of the overlay characteristics was begun with the 1970 program and is continuing [*5–6*]. Some of the measured characteristics include consistency, air content, setting time, compressive strength, freeze-thaw resistance, bond to concrete, length change, bond to steel, chloride penetration, corrosion of reinforcing steel, and visual observation of the decks under traffic.

The plastic concrete is extremely fluid, and consistency measurements are not possible using standard methods. Air content can be determined by standard methods, and air contents have been controlled in the range of 7 percent. Initial setting time is approximately 200 min, with a final setting time of about 260 min, as determined by proctor penetration.

Standard 6 by 12-in. (152 by 305-mm) cylinders were made and tested to determine an age-strength relationship. The compressive strength increased from 3000 psi (20.7 MPa) at 1 day of age to 5000 psi (34.5 MPa) at 28 days and 6000 psi (41.4 MPa) at 1 year. A typical age-strength relationship is shown in Fig. 1.

Freeze-thaw specimens were fabricated and tested in accordance with

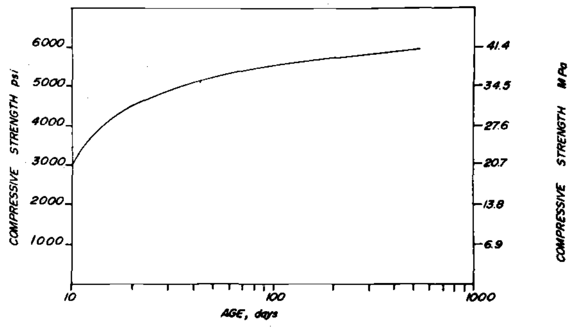

FIG. 1—*Age-strength relationship.*

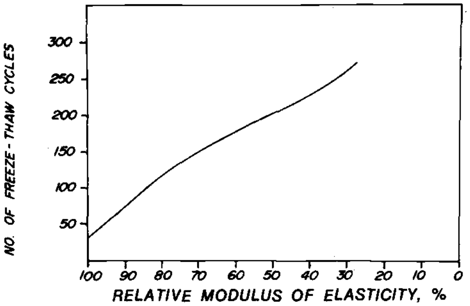

FIG. 2—*Resistance to freeze-thaw.*

ASTM Test for Resistance of Concrete to Rapid Freezing and Thawing (C 666-76). Figure 2 shows the relationship between freeze-thaw cycles and the relative dynamic modulus of elasticity. Approximately 175 cycles were required to obtain a dynamic modulus of 60 percent, resulting in a durability factor of 35. This compares favorably with conventional structural concrete used in West Virginia which has a durability factor in the range of 25 to 40 [9].

Bond to hardened concrete and to reinforcing steel appears adequate. After several years under traffic, investigation has revealed no debonding around steel or at the overlay-concrete interface. There is, however, evidence of incipient delamination approximately ¼ to ½ in. (6.4 to 12.7-mm) below the overlay concrete interface, although there is no evidence of surface distress to date.

Length changes as determined from standard laboratory specimens indicate an increase of approximately 0.034 percent over a 1-year period.

Soon after a placement, a profusion of tight pattern or shrinkage cracking may be observed. To date, such cracking has not progressed after its initial appearance. Under traffic, the deck surface develops surface voids which are similar in size and shape to entrapped air voids.

The surface friction of the overlays has been checked using a skid trailer conforming to ASTM Test for Skid Resistance of Paved Surfaces Using a Full-Scale Tire (E 274-70). The overlays show an average skid number of 40.4 at 55 mph (88.5 km/h) with individuals ranging from 30 to 50. This compares to an average skid number of 39.9 for the rigid pavements on West Virginia's Interstate System. A recommended minimum interim skid number for main rural two-lane highways is 31.5 [*10*]. It is anticipated that surface friction will remain adequate for these overlays.

Corrosion potential of the top mat of reinforcing steel is measured using the copper-copper sulfate half cell. In normal, sound reinforced concrete where the steel is not actively corroding, the half-cell voltage potential readings will range from 0.00 to 0.30 V. Where potential readings of 0.35 V or greater are encountered, the reinforcing steel is considered to be actively corroding.

Since the construction process requires removal of all unsound concrete and cleaning of exposed reinforcing steel, any readings of 0.35 V or greater would indicate that moisture and chloride ions are passing through the overlay and contributing to active corrosion. Such areas have been found in the decks investigated, especially along the centerline of the decks. These isolated sections are apparently the result of chloride penetration through construction joints and cracks. In addition, chemical analysis of incremental core specimens obtained throughout the depth of the overlay would indicate that chlorides are present throughout the overlays, although not in the quantities to be expected in conventional concretes [*11*]. A comparison is given in Table 2. While the overlays are not impermeable, the rate of chloride ion migration is such that spalling and delamination have not occurred to date. A typical deck section with overlay is shown in Fig. 3.

TABLE 2—*Comparison of chloride contents, latex concrete versus sound conventional concrete.*

	Percent Chlorides By Weight of Mortar		
Depth of Section in Core, in. (mm)	Latex Concrete Overlays[a]	Sound Concrete Unspalled Decks[b]	Sound Concrete Spalled Decks[b]
¼ (6.4)	0.02 to 0.10	0.07 to 0.18	0.16 to 0.29
½ (12.7)	0.003 to 0.02	0.05 to 0.17	0.16 to 0.26
1 (25.4)	0.002 to 0.03	0.03 to 0.08	0.08 to 0.14
2 (50.8)	0.002 to 0.03	0.01 to 0.04	0.02 to 0.05

[a] Data from Refs *4,6*.
[b] Data from Ref *11*.

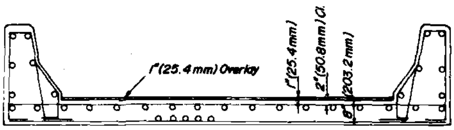

FIG. 3—*Typical deck section.*

Conclusions

Compared to other types of treatment under evaluation, the latex-modified concrete has been the most satisfactory of those in place long enough to have developed a history of performance. Postconstruction evaluations would indicate some cracking and some evidence of chloride penetration, although at a slower rate than in conventional concrete. Bond of overlay to substrate remains satisfactory, again with some evidence of incipient delamination. Overall appearance of the overlays remain good, with no serious distress to date.

Testing will continue to determine future effect of time and traffic on the overlays in place.

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the State or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

References

[*1*] "Concrete Bridge Deck Durability," Synthesis of Highways Practice 4, National Cooperative Highway Research Program, Highway Research Board, 1970.

[*2*] Hay, R. E., "The Bridge Deck Problem—An Analysis of Potential Solutions," *Public Roads*, Vol. 39, No. 4, March 1976.

[*3*] "Interim Report NEEP No. 12 Bridge Deck Protective Systems, Membranes, Polymer Concrete and Dense Portland Cement Concrete," Federal Highway Administration Notice N 5080.28, 21 Jan. 1975.

[*4*] "Bridge Deck Evaluation—Bridge No. 1604," Materials Inspection Report No. 608329, West Virginia Department of Highways, 20 Jan. 1971.

[*5*] "Bridge Deck Renovation Using a Latex Modified Concrete," Materials Research Report No. 712079, West Virginia Department of Highways, 28 Sept. 1972.

[*6*] "Bridge Deck Renovation Using A Latex Modified Concrete, Addendum," Materials Research Report No. 712079, West Virginia Department of Highways, 19 June 1973.

[*7*] "Special Provisions For Repair Of Portland Cement Concrete Bridge Decks With Latex Modified Portland Cement Compositions," West Virginia Department of Highways, 10 Aug. 1970.

[*8*] "Standard Specifications for Roads and Bridges," West Virginia Department of Highways, 1972.

[*9*] Esmer, Erkan, "Effects of Absorbed Moisture in Aggregates on Concrete Durability," Highway Research Project No. 33, Federal Highway Administration West Virginia Department of Highways, Oct. 1972.

[*10*] "Tentative Skid-Resistance Requirements for Main Rural Highways," Report 37, National Cooperative Highway Research Program, Highway Research Board, 1967.

[*11*] "Durability of Concrete Bridge Decks," a cooperative study, Bureau of Public Roads—Portland Cement Association, Final Report, 1970.

E. J. O'Connor[1]

Iowa Method of Partial-Depth Portland Cement Resurfacing of Bridge Decks

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REFERENCE: O'Connor, E. J., "**Iowa Method of Partial-Depth Portland Cement Resurfacing of Bridge Decks,**" *Chloride Corrosion of Steel in Concrete, ASTM STP 629,* D. E. Tonini and S. W. Dean, Jr., Eds., American Society for Testing and Materials, 1977, pp. 116–123.

ABSTRACT: Low-slump, high-quality concrete is used in Iowa for partial-depth patching and resurfacing of bridge decks. All unsound concrete is removed, as well as a minimum of ¼ in. (0.054 m) of the remaining sound concrete surface. Sand cement grout is scrubbed onto dry, prepared deck immediately prior to compacting low-slump concrete into place using vibrating screeds.

KEY WORDS: corrosion, concretes, reinforcing steels, bridge decks, highway bridges, concrete finishes, grout, maintenance, protective treatments, membranes

In the early 1960's, a survey of bridge deck conditions in Iowa found that spalling and delamination in the decks at the top reinforcing layer were increasing maintenance problems and causing loss of structural integrity in the bridge deck.

In 1963, a research and development project to investigate materials and methods for bridge deck repair and protection was begun. Three engineers, one each from the Offices of Materials, Design, and Maintenance, were assigned to the project. All of these offices have contributed to our current program.

Our first efforts started in 1963 and were aimed at developing a method for a permanent, partial-depth patching of spalled areas. We were looking for a material that would bond securely to the existing sound concrete, exhibit little or no shrinkage during or after curing, and have a high early strength to minimize inconvenience to traffic.

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Our experimental patching program included patches composed of:

1. Air-entrained portland cement concrete
2. Air-entrained concrete with water-reducing additives
3. Air-entrained concrete with a metallic additive to reduce shrinkage
4. Air-entrained concrete with polyvinyl acetate as a bonding agent
5. Air-entrained concrete with epoxy as a bonding agent
6. Latex-modified mortar
7. Epoxy-sand mortar, and
8. A mixture of aggregate and polymeric colorless binder, called "velsicol."

In the spring of 1964, an evaluation was made of the patching methods and materials used. Field tests of relative "bond strength" in tension were made on those patches that had not obviously failed during the winter. It was determined that our best results, at the lowest cost, were obtained with low-slump (¾-in.) portland cement (PC) concrete, with sand cement grout as a bonding agent, placed on a carefully prepared surface. During the spring and summer of 1964, maintenance personnel placed partial-depth concrete patches on several bridges.

After sawing around the areas to be patched and removing all unsound concrete, with pneumatic welding flux chipping hammer or hammer and cold chisel, steel plates with electric vibrators attached or vibrating screeds compacted and finished the low-slump concrete. Sand-cement grout was the bonding agent, and the patches were opened to traffic in 3 to 4 h. Our maintenance district bridge crews are still using this procedure for partial-depth patching.

During the patching program, it was found that not all bridges were adaptable to patching because there were large areas of high reinforcing steel (less than ¾-in. cover (0.18 m)). Also, the percentage of delaminated and spalled concrete showed that there would be more patches than original deck when the repair was finished. These results led to the resurfacing of an entire bridge deck with a thin layer of PC for the first time in 1964. A 240-ft (73-m) two-lane bridge with numerous surface spalls over shallow reinforcing was selected. Some reinforcing had only a ½-in. cover (0.13 m).

Evaluation of this project led to the conclusion that thin-layer or partial-depth resurfacing with low-slump concrete was feasible. Also, it was found that the same basic procedures used for patching were applicable.

We use the same basic specifications for our contract resurfacing projects, as summarized in the following section.

1. Remove all areas of delaminated or deteriorated concrete to sound concrete (Figs. 1 and 2).
2. Remove at least ¼ in. (0.07 m) of concrete from the remaining deck surface (Fig. 3).
3. Sand cement grout is scrubbed onto a dry bridge deck to serve as a

FIG. 1—*Pneumatic hammers (#30 maximum) used in initial removal of unsound concrete.*

FIG. 2—*Chipping hammers used in final cleanup of areas of unsound concrete and around unbonded reinforcing bars.*

FIG. 3—*Floor ready for resurfacing. Areas of unsound concrete removed and remaining concrete scarified.*

bonding agent immediately before placing ¾ in. (0.18 m) slump concrete. Concrete, a nine-sack mix, is mixed on the job site using rotating paddle-type mixers or a volumetric proportioning mixer (Fig. 4).

4. The deck is resurfaced with 1 ½ in. (0.36 m) of dense concrete, using a finishing machine with vibrating screed to achieve desired compaction (Fig. 5).

The first bridge resurfaced under a regular contract was completed in 1965. The structure was a 301 by 52 ft (91.7 by 15.9 m) continuous-welded girder built in Des Moines, Iowa, in 1960 and carried a heavily traveled city street over the freeway. It receives a liberal application of salt. The team from the Federal Highway Administration (FHWA), demonstrating a corrosion detection device, checked this floor in 1971. Except for four in one corner, the readings were in the vicinity of −0.20 V. The deck was checked in 1972 and 1974. In 1974, there were four readings between −0.31 and −0.36, and the remainder ranged from −0.14 to −0.26 V. Cores from the deck, obtained in 1974 to determine chloride content at various levels, indicated an equivalent of 5 lb of chloride per cubic yard (3 kg/m^3) in the top ¼ to ½ in. (0.07 to 0.13 m). At the 1 to 1¼-in. (0.25 to 0.32-m) level, the chloride content was 1.91 lb/yd^3 (1.13 kg/m^3). The old deck (under the resurfacing) has 1.74 lb of chloride per cubic yard.

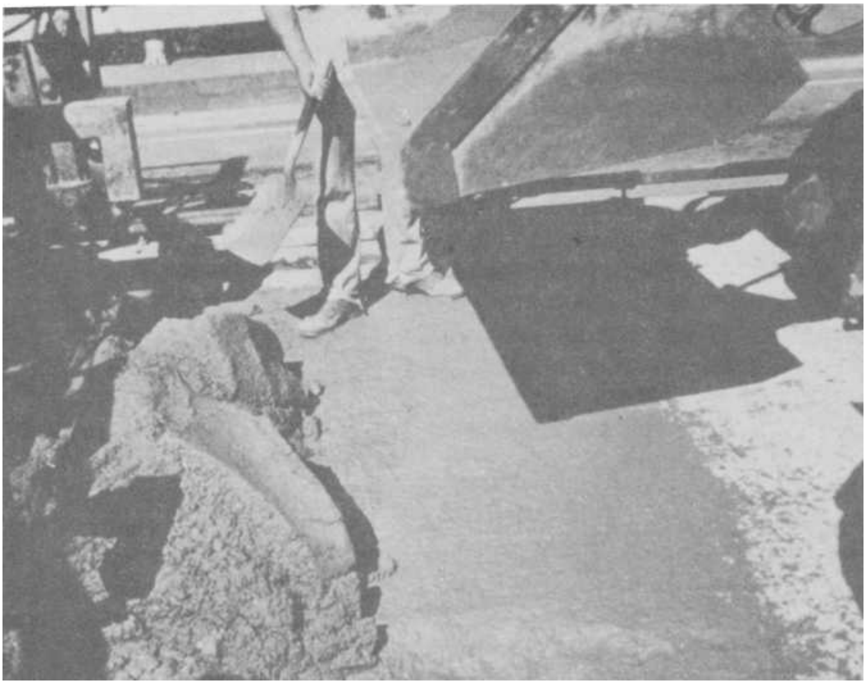

FIG. 4—*Low-slump concrete dumped in front of finishing machine on fresh grout.*

During 1974, cores were taken from 15 bridges that had been resurfaced since 1965.[2] Chloride concentrations in the old concrete at the level of reinforcing were as high as 8 lb/yd^3. This occurred on just one bridge. In most instances, the chloride content in the old decks of bridges investigated ranged from 2 to 4 lb/yd^3 (0.91 to 1.81 kg/m^3).

In cores taken from the resurfaced bridges in this study, it was found, as suspected, that the chloride content of the resurfacing generally decreased as its depth below the deck surface increased. However, there was no discernible relationship between the age of the decks or amount of salt applied, and the chloride content. There was no evidence of distress in these bridge decks.

We have investigated delamination that occurred in about two thirds of one resurfaced bridge not included in the study just discussed. This delamination was discovered about four years after the project was completed. Cores were secured, and we found that the delamination occurred in the old concrete, not in the bond line between the new and old concrete (Fig. 6). Chloride content of the old concrete approximately ½ in. below the

[2] Bergren, J. V. and Brown, B. C., "An Evaluation of Concrete Bridge Deck Resurfacing in Iowa," Special Report, Iowa Department of Transportation, June 1974.

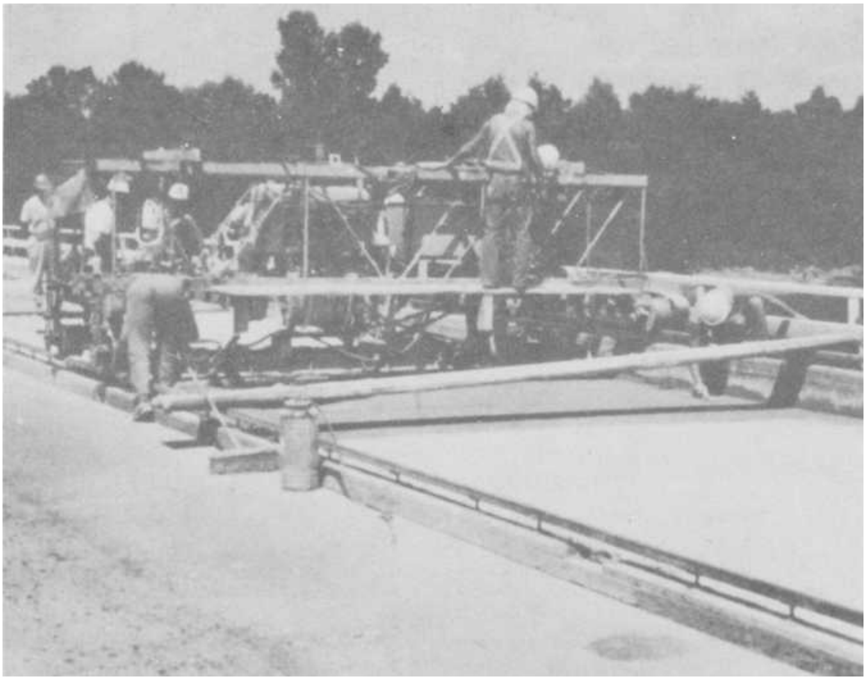

FIG. 5—*Weights and vibrators were added to this conventional finishing machine to compact and finish the dense concrete. There are now finishing machines commercially constructed for this specialized finishing operation.*

bond line ranged from 2.11 to 7.03 lb/yd^3 (1.25 to 4.17 kg/m^3). The average concentration was 3.6 lb/yd^3. Corrosion detection (half-cell potential) readings[3] were taken on a 4-in. grid (1.2 m). Only 15 of approximately 370 readings were over −0.35 V, and these were scattered. The majority of the readings were in the middle to low twenties. We have reached no definite conclusion as to the cause of delamination. The possibility of incomplete preparation prior to resurfacing needs to be reviewed. The floor has been injected with epoxy using the same type of equipment as was developed by Kansas,[4] and we have had no further reports of delamination.

Corrosion detection readings have been taken on two bridges found to have high chloride readings in the old concrete under the resurfacing. On one noted previously that had as much as 8 lb of chloride per cubic yard (3.6 kg/m^3), 54 readings were taken in the vicinity of the cores. Only 3 of

[3] Speelman, D. L. and Stratfull, R. F., "Chlorides and Bridge Deck Deterioration," Highway Research Record No. 328, 1970.

[4] Stratton, F. W. and McCollom, B. F., "Repair of Hollow or Softened Areas in Bridge Decks by Rebonding with Injected Epoxy Resin or Other Polymers," Kansas Report No. K-F-72-5, National Technical Information Service, Springfield, Va., July 1974.

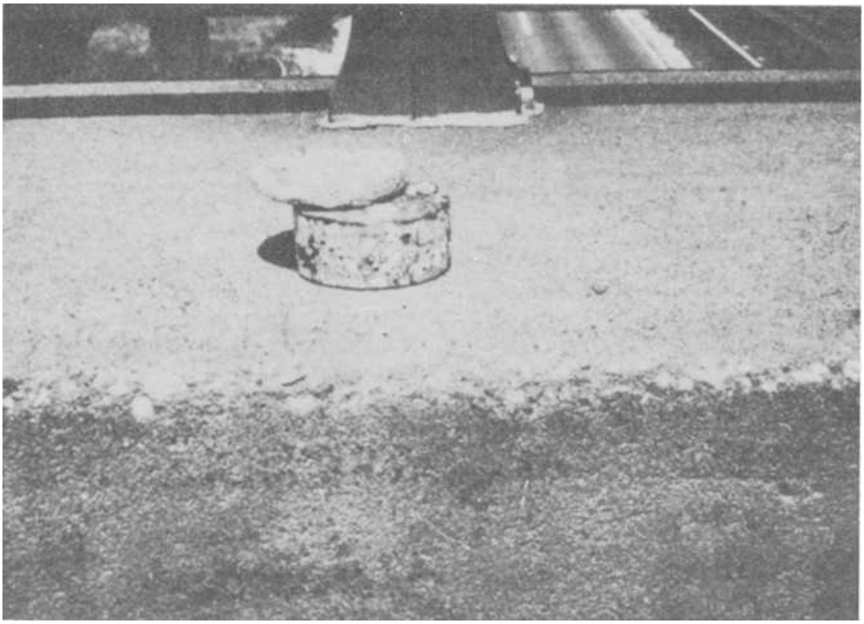

FIG. 6—*Core was placed surface side down to photograph. There were two laminations in the original deck below the resurfacing at this location: one just below bond line, the other approximately ½ in. (0.125 m) deeper.*

the 54 readings were over −0.35; the remainder were all below −0.30 V. On another bridge (resurfaced in 1968) with chloride content of about 5 lb/yd^3 (3 kg/m^3) in the old deck, 120 readings were taken. Only 9 of the 120 were between −0.30 and −0.35 V. There were none over −0.35 V.

Considering the high cement content and extreme consolidation of the overlay, although contaminated with chloride, there is possibly not enough moisture or oxygen at the reinforcing steel level to initiate corrosion or allow it to continue. Also, the relatively high cement content may inhibit corrosion and, consequently, spalling activity because of the high pH produced by the excess of free lime precipitated during hydration of the cement.[5]

As a result of these investigations conducted by the Iowa Department of Transportation Materials Department in regard to chloride concentration at various levels in bridge decks, and because Iowa has experienced excellent durability with the low-slump, dense concrete, the FHWA gave approval early in 1974 to let resurfacing contracts with alternative or optional methods. That is, the contractor was permitted to use either the Iowa low-slump concrete or latex-modified concrete as a protective membrane. In 1974, all contractors in Iowa chose the "Iowa method" for re-

[5] Reviewer's comment.

surfacing contracts. On one 1975 contract, the contractor chose the Dow system. The remaining 24 bridges were resurfaced with the Iowa method.

Iowa has had about twelve years of favorable experience with partial-depth bridge deck resurfacing using low-slump portland cement concrete with a sand cement grout bonding agent. The only additives used are for air entrainment and water reduction. We regard careful preparation; high-quality, low-slump concrete; and overall good workmanship to be essential features of the repair program.

Acknowledgments

The encouragement and confidence of Chief Engineer L. M. Clausen and key staff members during the development of this method of bridge deck resurfacing are appreciated. The cooperation and assistance of O. J. Lane, Jr., and John Whiting are also gratefully acknowledged.

J. B. Vrable[1]

Cathodic Protection for Reinforcing Steel in Concrete

REFERENCE: Vrable, J. B., "**Cathodic Protection for Reinforcing Steel in Concrete,**" *Chloride Corrosion of Steel in Concrete, ASTM STP 629,* D. E. Tonini and S. W. Dean, Jr., Eds., American Society for Testing and Materials, 1977, pp. 124–149.

ABSTRACT: Chloride-induced corrosion of the reinforcing steel in concrete is a major cause of premature concrete deterioration. This presentation describes the results of an investigation sponsored by the National Cooperative Highway Research Program (Project 12-13) into the use of cathodic protection as a method of suppressing the corrosion of reinforcing steel in highway bridge concrete decks. The principal cause of bridge-deck corrosion is salt used for snow and ice removal. In this study, factors affecting the cathodic protection of steel reinforcement in concrete were investigated, and a variety of impress-current and sacrificial-anode approaches to cathodic protection of bridge decks were examined in laboratory and model testing.

The results of electrochemical studies showed that corrosion can be controlled by the application of cathodic current to corroding steel in concrete. The minimum protective potential for cathodic protection was established. Studies showed that cathodic-protection current can result in a decrease in bond strength between the steel and the concrete. However, the decrease would be slight at the anticipated conditions of protection.

A method previously developed by the California Department of Transportation was examined in depth, and it was concluded that this method offered promise as an impressed-current system. A sacrificial-anode design with zinc anodes also was determined to hold promise.

KEY WORDS: corrosion, concretes, reinforcing steels, deterioration, chlorides

The increased use of deicing salts for the removal of snow and ice from the nation's highways has resulted in premature occurrence of spall and pothole damage on concrete bridge decks. Previous research has identified chloride-induced corrosion of the reinforcing steel (rebars) in the bridge decks as the major cause of deterioration [*1–3*].[2] The purpose of the present research has been to investigate the use of cathodic protection as a method of suppressing bridge-deck corrosion. This work was sponsored

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[2] The italic numbers in brackets refer to the list of references appended to this paper.

by the National Cooperative Highway Research Program (NCHRP) of the Transportation Research Board, National Research Council.

Because of the great amount of data produced during the 18-month research program, it was practical to include in this paper only the information needed to illustrate typical findings or significant points. Accordingly, the reader is cautioned against in-depth analysis of the conclusions presented on the basis of the fragmented results in this paper, but rather he is advised to consult the full NCHRP report [*4*].

Research Approach

The overall objective of the research was to develop a technically and economically feasible cathodic-protection method for steel-reinforced concrete bridge decks. The principal application of cathodic protection for steel-reinforced concrete has been for buried pipelines, where it has proved to be effective against the corrosive effects of soil and water [*4*]. However, the use of cathodic protection for bridge decks presents certain problems not encountered with buried structures. Foremost among the problems is the lack of space and suitable environment for the anode. Nevertheless, it was considered possible that a cathodic-protection method could be developed that would not only protect new bridge decks but would also suppress active corrosion in existing salt-contaminated decks, thereby extending their useful life.

Experimental Work

Electrochemical Studies

A series of electrochemical studies was conducted on steel exposed to simulated concrete environments to evaluate the feasibility of preventing corrosion of rebars in concrete bridge decks by cathodic protection. The specific objectives of these studies were to identify conditions under which rebars would be expected to corrode in concrete, to establish the potential criteria necessary to achieve cathodic protection under these environmental conditions, and to determine the hydrogen-evolution potentials of steel in these environmental conditions.

Hydrogen-Bubble-Evolution Potentials—Hydrogen-bubble-evolution potential ($E_{B(H_2)}$) measurements were conducted on rebar steel in various solutions by using the apparatus shown in Fig. 1. The information concerning $E_{B(H_2)}$ was developed because investigators of steel-in-concrete cathodic-protection applications recommend that the maximum potential value must be limited to about -1.1 V to copper-copper sulfate ($Cu/CuSO_4$)

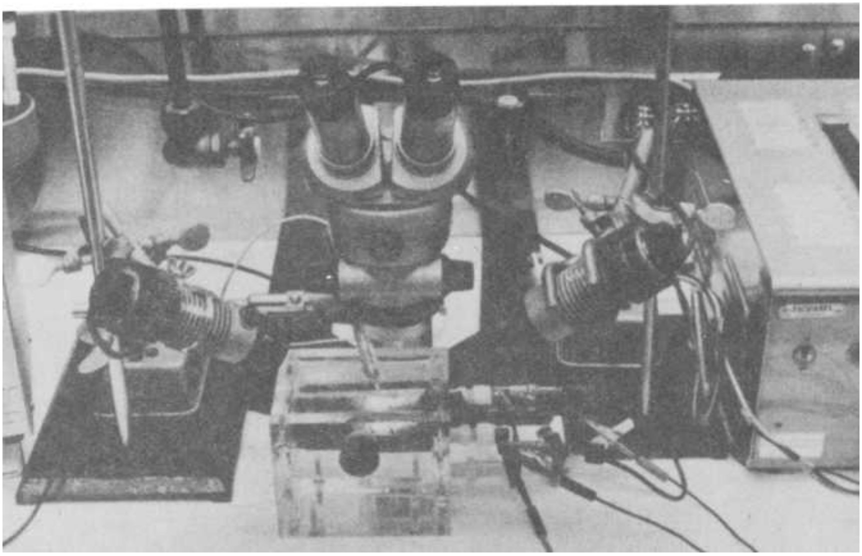

FIG. 1—*Photograph of assembly used to determine the hydrogen "bubble" evolution potential of rebar steel in simulated concrete environments.*

electrode [*5, 6*].[3] Reportedly, this is necessary to avoid loss of bond between concrete and rebars.

For this test, inspection of the specimen for bubbles was made after 12-h intervals of applied cathodic protection (constant potential), and, if none were observed, the potential was made more negative for an additional 12 h. The $E_{B(H_2)}$ measurements were conducted in a variety of environments in which the pH and Cl^- concentration were varied independently, and the results are summarized in Table 1. The data show the expected trend of shifting to more active potentials (more negative) as the pH of the solution was increased. It should be noted also that $E_{B(H_2)}$ was independent of Cl^- concentration. The values of $E_{B(H_2)}$ were found to be independent of whether the rebar electrode contained original mill scale or was acid pickled prior to the experiment.

Cathodic-Protection Criteria—The corrosivity of various simulated concrete environments (solutions of saturated $Ca(OH)_2$ to which 0.02, 1.0, 3.0, and 5.0 $\bar{M}$ sodium chloride (NaCl) was added) was ascertained by the use of AISI Grade 1036 steel electrical-resistance probes. Two exposure conditions were employed: (*a*) with the resistance probe exposed to a large freely conductive volume of corrodent; and (*b*) with the resistance probe immersed in limestone aggregate, which in turn was immersed in the corrodent as shown in Fig. 2. This latter condition was employed to

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[3] All potentials are reported and measured against a saturated $Cu/CuSO_4$ electrode.

TABLE 1—*Summary of hydrogen-bubble-evolution potentials for AISI 1036 carbon steel in various solutions.*

Solution	pH	$E_{B(H_2)}$,[a] V_{CuSO_4}
0.025 $\bar{M}$ Ca $(OH)_2$ (saturated)	12.5	−1.170
0.025 $\bar{M}$ Ca $(OH)_2$ + 0.02 $\bar{M}$ Cl^-[b]	12.3	−1.170
0.025 $\bar{M}$ Ca $(OH)_2$ + 6.1 $\bar{M}$ Cl^-	12.2	−1.170
5.36 $\bar{M}$ Cl^-[c]	9.4	−1.020
5.36 $\bar{M}$ Cl^-	7.9	−1.035
5.36 $\bar{M}$ Cl^-	4.0	−0.970

[a] $E_{B(H_2)}$ = hydrogen-bubble-evolution potential determined at ×20 magnification.

[b] $[Cl^-]$ added as NaCl.

[c] $[Cl^-]$ added as $CaCl_2$, pH adjusted with either HCl or Ca $(OH)_2$.

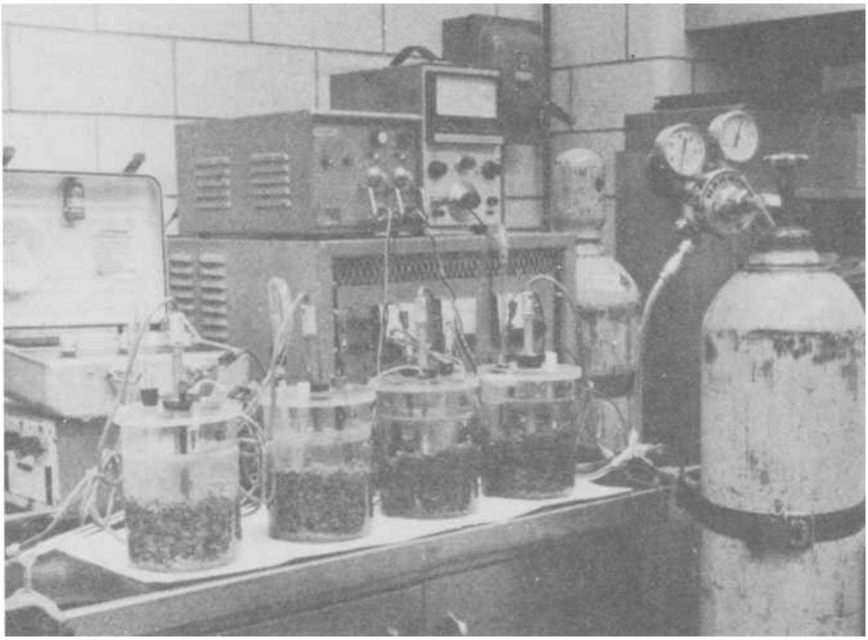


FIG. 2—*Photograph of assemblies used to study the corrosion behavior of resistance probes buried in limestone aggregate.*

stimulate contact corrosion due to the establishment of differential aeration cells [*6*]. The presence or absence of corrosive attack was confirmed by changes in the electrical resistance of the probes as measured by an appropriate instrument. The effectiveness of cathodic protection at a given applied potential was monitored by inspection of the probe-resistance/time plot. During periods of corrosion, a positive slope would be observed, whereas when the corrosive attack stopped, the slope of the plot went to zero.

As illustrated in Fig. 3, sustained corrosive attack was observed when the probes were immersed in aggregate at chloride levels greater than 0.02 $\bar{M}$. Once attack was established, the minimum cathodic applied potential necessary to arrest attack was evaluated by progressively increasing the cathodic potential (with a potentiostat) in steps of 30 to 50 mV. The protection potentials, $E_{\text{protection}}$, and associated current densities, i_c, for each environmental condition are summarized in Table 2. It is significant to note that $E_{\text{protection}}$ in each case was less than the commonly used −850 mV criterion used in the field [7] to protect buried steel in soil and water environments. Values within 140 mV negative to the steady-state corrosion potential were sufficient to achieve cathodic protection; specifically, the data obtained under the most severe condition in this study (5.0 $\bar{M}$ Cl^-) reveal that a potential of − 770 mV is an adequate criterion for

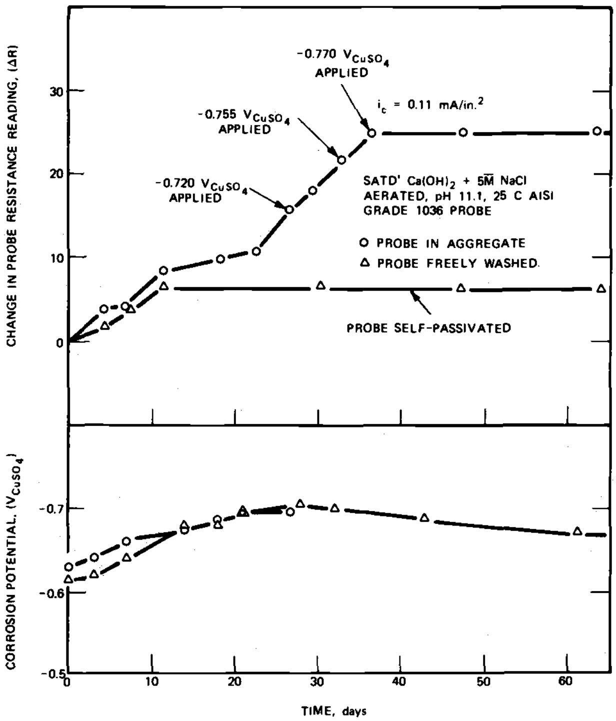


FIG. 3—*Probe resistance/time data for AISI Grade 1036 steel in halide media as a function of applied cathodic potential.*

TABLE 2—*Summary of cathodic-protection requirements for AISI 1036 carbon steel in saturated $Ca(OH)_2$ solution with various chloride contents.*

Solution	pH	E_{corr}, V_{CuSO_4}	$E_{protection}$, V_{CuSO_4}	i_c,[a] mA/in.^{2}
0.02 $\bar{M}$ NaCl (probe in aggregate)	11.83	−0.650	NC[b]	...
0.02 $\bar{M}$ NaCl (probe freely washed)	11.83	−0.500	NC	...
1.0 $\bar{M}$ NaCl (probe in aggregate)	11.86	−0.680	−0.770	0.44
1.0 $\bar{M}$ NaCl (probe freely washed)	11.86	−0.520	NC	...
3.0 $\bar{M}$ NaCl (probe in aggregate)	11.91	−0.690	−0.720	0.31
3.0 $\bar{M}$ NaCl (probe freely washed)	11.91	−0.670	NC	...
5.0 $\bar{M}$ NaCl (probe in aggregate)	11.1	−0.630	−0.770	0.11
5.0 $\bar{M}$ NaCl (probe freely washed)	11.1	−0.670	NC	...

[a] i_c = current density for complete cathodic protection.

[b] NC = no corrosion. Probe self-passivated over duration of test (65 days).

cathodic protection. It is recognized that the values of i_c in Table 2 are relatively high, but it should be pointed out that these values were decreasing with time and represent only the value after 15 to 30 days exposure. It is very probable in practice that i_c would decrease to much smaller values.

Cathodic Protection Exposure Tests

An important part of the research was directed towards the determination of possible adverse effects of cathodic protection on a concrete-to-rebar bond and on the concrete itself. Studies by previous investigators [*8-11*] indicated that there is a slight weakening of bond strength of rebars in concrete with cathodic protection.

For the study of the effect of cathodic protection on bond strength, a modification of the stub-cantilever-beam specimen described by Kemp et al [*12*] was used. Two concrete curing times, two and four months, were tested. The test conditions and exposure times for each are summarized in Table 3.

The effect of cathodic protection of the concrete was evaluated by studying the performance of 6 by 12-in. concrete-cylinder specimens (containing no rebars) in the splitting tensile test after exposure to various levels of applied current.

The equipment used for the rebar pullout tests is shown in Fig. 4. A horizontal jack, exerting a tensile force recorded by a load cell, was connected to the rebar by a clamping mechanism. Horizontal direct-current differential transformers were used for measuring longitudinal movements of the bar with respect to the concrete.

Effects on Bond Stress—The data reduction was concerned with examining the two most significant parameters of the bond tests: the ulti-

TABLE 3—*Test conditions: modified stub cantilever beam specimens.*[a]

Total Applied Current, Ah/ft^2	Days Exposure at Applied Current of		
	48 mA/ft^2[b]	480 mA/ft^2[b]	960 mA/ft^2[b]
	TWO MONTHS CONCRETE CURING TIME		
346	300	30	15
3460		300	150
6920			300
	FOUR MONTHS CONCRETE CURING TIME		
346		30	15
3460			150

[a] All specimens exposed in triplicate.

[b] 48 mA/ft^2 = applied current of 2.5 mA.
480 mA/ft^2 = applied current of 25.0 mA.
960 mA/ft^2 = applied current of 50.0 mA.

FIG. 4—*Overall view of equipment used for conducting load-strength tests on stub-cantilever-beam specimens.*

mate bond stress and the bond stress at 0.01-in. loaded-end slip, which would correspond to a "maximum tolerable" crack of 0.02 in. resulting from bond stress in an actual structure. The latter is generally considered to be the limit of "serviceability" in reinforced-concrete bond research.

To determine the statistically significant relationships of each dependent variable to the independent variables, a number of multiple-regression analyses were carried out. Also, several analyses of variance were performed to study the relative variability of the triplicate specimens.

For ultimate bond stress (30 specimens), current density, number of days, and total applied Ah/ft^2, all have some linear effect. However, since there is a direct relationship among those three variables and total ampere hours has the highest simple correlation coefficient (-0.16), it appears reasonable to attribute changes in ultimate bond strength to this variable. There was, however, no indication of an effect related to concrete curing time.

Figure 5 shows that there was about 10 percent decrease in ultimate bond strength after the application of 3460 Ah/ft^2 current. A further application of current to a total of 6920 Ah/ft^2 produced an additional reduction of about 10 percent in bond strength.

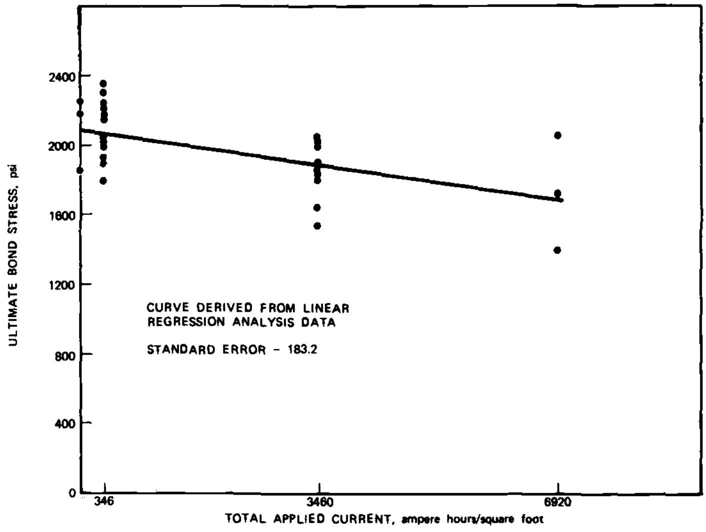


FIG. 5—*Total applied current, ampere hours/square foot effect of applied current on ultimate bond stress.*

It should be noted, however, that the 6920 Ah/ft^2 of current that was applied in this study, which was done to observe the effects of excessive amounts of current, is far in excess of any reasonable level of cathodic-protection current that would be applied to a bridge structure. The 6920 Ah/ft^2 is equivalent to about 75 years of protection at a very high level of current density.

When the effect of current density on ultimate bond strength is considered (Fig. 6), the loss in bond strength is much less with increasing current density than with an increasing total ampere hour/square foot of applied current (Fig. 5). This result supports the previous indication that

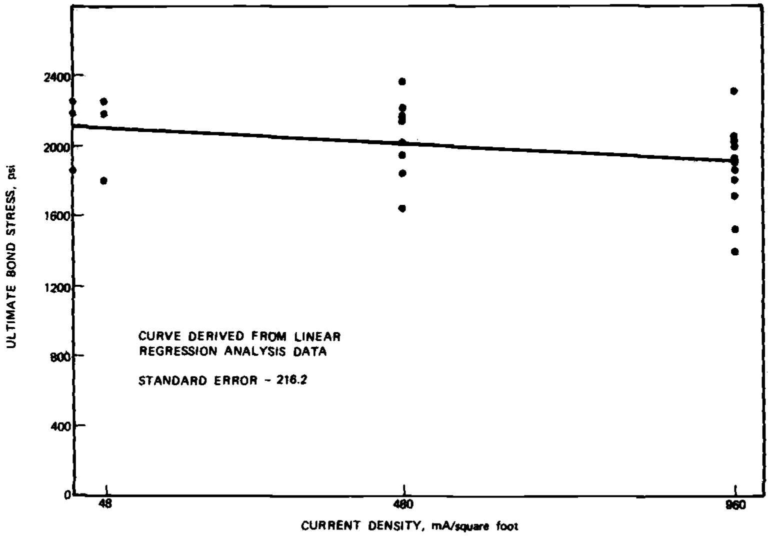


FIG. 6—*Effect of current density on ultimate bond stress.*

changes in ultimate bond strength are attributable to the total applied current (that is, the total number of coulombs passed).

In the case of bond stress to produce 0.01-in. loaded-end slip (30 specimens), no statistically significant relationships of dependent variables to the independent variables were found.

Inspection of the specimens that split along the reinforcing steel during the pullout tests showed indications of a concentration of light-colored material along the rebar, visible in the photograph in Fig. 7. This material

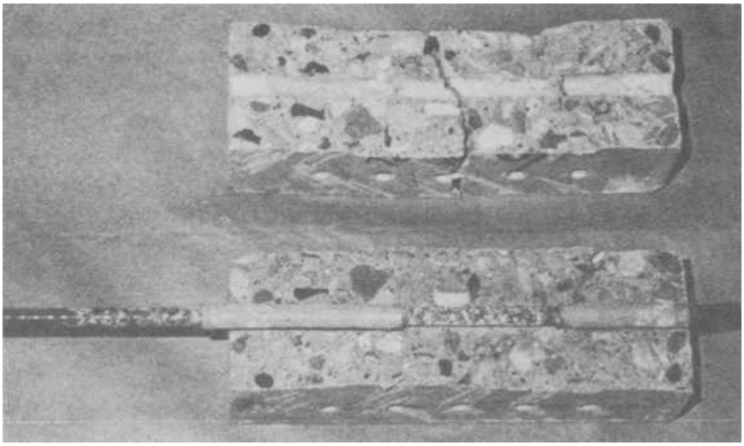


FIG. 7—*Failed stub-cantilever beam specimen after pullout test.*

was scraped off the surface of the steel and the concrete and, although not spongy, appeared to be somewhat softer than the surrounding concrete. It is possible that this was caused by the production of alkali hydroxides on the steel surface as opposed to loss of bond from hydrogen evolution. These hydroxides are believed to attack the calcium and aluminum silicates, yielding soluble silicates.

Effects on Concrete—After 300-day exposure to cathodic-protection current flow, visual inspection of the splitting tensile specimen employed in this test revealed no apparent degradation of the concrete. As can be seen from the results of the splitting tensile testing (Table 4), there was no relationship between applied current and the splitting tensile strength of the specimens. The major cause of the variations in performance ap-

TABLE 4—*Effect of applied current on splitting tensile strength.*

Specimen	Total Load, lb	Splitting Tensile Strength, psi	Comments
Control, 30 day cure			
A	56 500	500	
B	53 500	473	
C	46 500	411	
Avg	52 166	461	
300 days exposure to 0.5 mA applied current			
1	65 000	575	80% shear
2	60 000	531	75% shear
3	59 500	527	80% shear
Avg	61 500	544	
300 days exposure to 1.0 mA applied current			
4	49 000	434	sandy pocket
5	51 000	451	sandy section through center
6	65 000	575	100% shear
Avg	55 000	487	
300 days exposure to 2.0 mA applied current			
7	64 500	570	80% shear, good matrix
8	57 500	509	75% shear
9	63 000	558	90% shear
Avg	61 660	546	90% shear
Control, 300 day cure			
A	52 000	460	75% shear, erratic break
B	59 000	523	80% shear
C	51 000	452	80% shear, sandy matrix
Avg	54 000	478	

pears to be related to the quality of the test specimen (such as nonuniform distribution of aggregate and the type of subsequent fracture), not to the effects of the applied currents, as noted in Table 4.

Analog Model and Related Investigation

Concurrent with the aforementioned studies, a series of cathodic-protection tests was performed with a specifically constructed physical model of a reinforced-concrete bridge deck. The object of this work was to determine the essential electrical parameters of a cathodic-protection design for this application. The 12 by 12-ft model used in this testing incorporated most of the important structural details of an actual bridge deck.

Analog Model

In general, the test procedure with the analog model was to simulate either a top or soffit anode cathodic-protection design and then apply cathodic protection with either an impressed-voltage or sacrificial-anode method. The applied voltage was held constant until a steady state of electrical potential of the steel rebar network, top and bottom, was reached. Usually a series of tests consisting of progressively greater impressed voltages was conducted with a given set of design parameters. No cathodic protection was applied for several days between design changes to reduce the effects of residual polarization on the test results.

Once steady-state conditions were indicated, the polarized potential of the entire top and bottom rebar network was measured by a $Cu/CuSO_4$ electrode placed as near as possible to the rebars at the point of interest. The pictures in Fig. 8 show the manner in which the electrode was placed (*a*) from the top of the deck and (*b*) from the bottom.

The potential between the electrode and the steel was measured by a high-impedance voltmeter. These measurements were made at locations corresponding to a grid pattern, usually at 2-ft intervals. The rebar potential values were recorded on a grid sheet, and equipotential lines, usually at 100-mV intervals, were plotted. The resultant equipotential contour maps indicate the area of the rebar network with sufficiently polarized potentials to obtain cathodic protection. Also, the maps give an indication of the current distribution obtained as various voltage and design changes were made.

Analog Testing

Soffit Anode Investigation—The analog model was used to conduct tests to determine the effects of an impressed-current method utilizing soffit anodes and conductive backfill placed in a container affixed to the

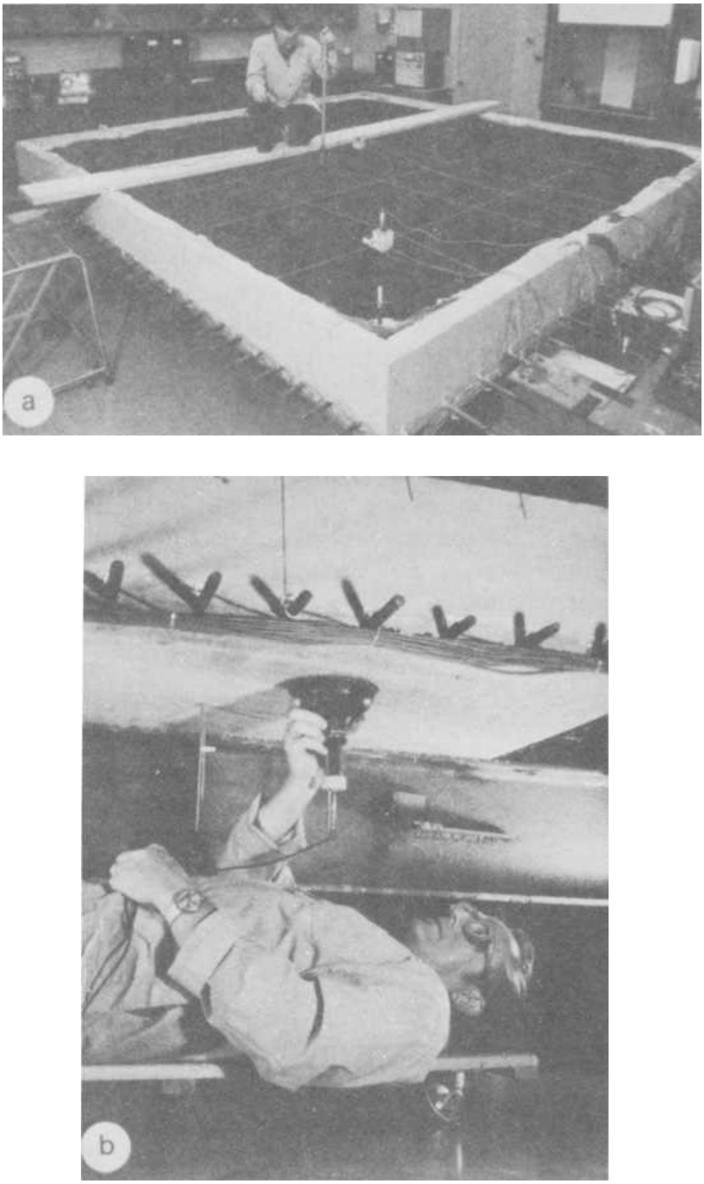


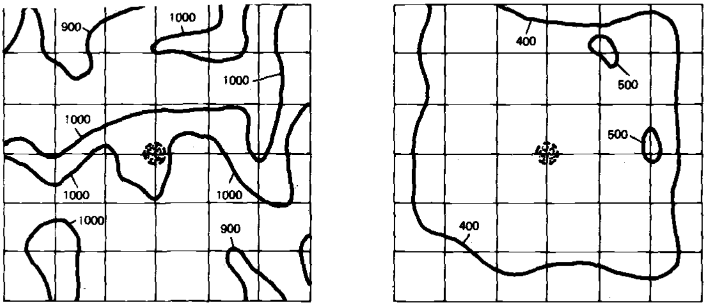
FIG. 8—*The manner in which the electrode for potential measurements was positioned* (a) *from the top of the analog model and* (b) *from the bottom. For bottom measurement, the electrode was modified with a plumber's helper containing a wet sponge to improve surface contact.*

underside of the deck. This work included investigations of the effects of different types, sizes, shapes, and locations of the soffit anode under different impressed voltages, both pulsed and continuously applied, on the rebar network potential gradient.

After considering the combined results of all phases of the soffit anode testing, it was concluded that a soffit anode location gave poor current distribution from the anode to the top rebar mat. With maximum impressed voltage, an area less than twice the anode-backfill contact area could be protected. Pulsing the impressed voltage lowered the total current needed to maintain the protected area but did not appreciably improve current distribution or extend the protected area.

Top-Anode Investigation—At the ouset of this program, it was learned that R. F. Stratfull of the California Department of Transportation was conducting an extensive investigation to develop a new cathodic-protection method to arrest corrosion of steel reinforcement in concrete bridge decks. The ingenious aspect of the Stratfull approach is the use of an electrically conductive asphaltic overlay on the deck to distribute the protective current over the top rebars. It was previously determined that cathodic protection of the top network of steel rebars would be adequate to prevent deterioration of concrete bridge decks.[4]

Experiments were conducted with an impressed current method and an anode placed in a conductive material over the deck surface. For this test, a 6-in.-diameter disk-shaped silicon iron anode was placed in a layer of coke breeze covering the surface of the model. The maps, Fig. 9, are



A. TOP MEASUREMENTS

B. BOTTOM MEASUREMENTS

FIG. 9—*Equipotential contour maps of bridge deck model with continuous cathodic protection applied at 4.7 V and 0.6 A. Two top anodes, as indicated on the 2-ft grid, in a 3-in. layer of coke breeze covering the entire deck surface. Contour units are millivolts negative to a Cu/CuSO$_4$ electrode. Potential values measured for top are distorted by coke breeze overlay.*

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[4] Protection of the top mat only is adequate if, at the time cathodic protection is added, there is no evidence of corrosion in the bottom layer of steel and if additional salts do not subsequently reach that steel.

typical of the results obtained with this design for cathodic protection. These tests show that cathodic protection of the top network of steel rebars can be readily accomplished with a very low impressed voltage and total current.

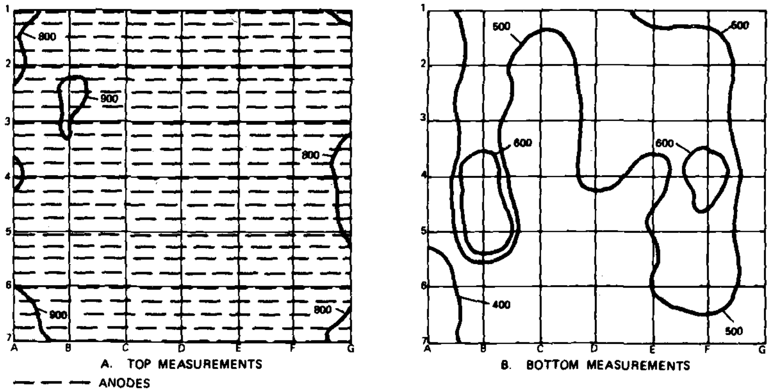
Essentially, a material to be used as the conductive component in the overlay for this approach to cathodic protection by the impressed-current method must be chemically inert and a good electrical conductor. Generally, those metals that would not corrode in this application, such as platinum, are not candidates for use because of their high cost.

Of the various nonmetals, the carbonaceous materials stand out as best for use in the conductive overlay. These materials, such as coke and graphite, meet the aforementioned essential requirements and have a long history of use as cathodic-protection ground-bed material. Graphite particles have been used in electrically conductive asphaltic concrete [*13*] used as snow-melting pavement. Coke, because it is more readily available and less costly, was preferred over graphite in this application.

Sacrificial-Anode Investigation—Insofar as it was demonstrated that cathodic protection of the top steel rebars can be achieved with a low applied voltage and small current, provided that the anode arrangement gives the necessary current distribution, it was concluded that the sacrificial-anode method of cathodic protection might be possible in this application.

Initial laboratory experiments conducted with magnesium and zinc alloys, to aid in predicting their service performance in the present application, showed that both metals are capable of providing cathodic-protection current when the concrete is moist and contains some chloride. Other tests were performed to evaluate the effects on the encasement material of corrosion of magnesium and zinc anodes encased in conventional paving materials. The results of this work show clearly that corrosion of the magnesium caused considerable damage to both encasement materials used, whereas the zinc did little damage to either the concrete or asphalt.

A sacrificial-anode design was evaluated consisting of extruded ribbons of magnesium cut into 12-ft lengths with each length placed across the top of the analog deck model. The maps shown in Fig. 10 illustrate the results of one of these tests. Figure 11 shows that the effect of increasing the number of magnesium anodes (reducing the distance between the ribbons) is to decrease the cathodic-protection circuit resistance while increasing the total anode current output. Overall, the results of this work suggested that the maximum anode spacing for complete protection of the top rebar mat is essentially one ribbon anode between each pair of adjacent top rebars. Because the driving voltage of a zinc anode system is much less than that of the magnesium system, it would appear that at least the same anode spacing indicated by the magnesium test would be required for a zinc anode system.



A. TOP MEASUREMENTS

B. BOTTOM MEASUREMENTS

FIG. 10—*Equipotential contour maps of bridge deck model with magnesium-ribbon-anode system. Anodes located as indicated (0.5-ft centers) on the 2-ft grid. Current output from anodes 920 mA. Contour units are millivolts negative to Cu/CuSO$_4$ electrode.*

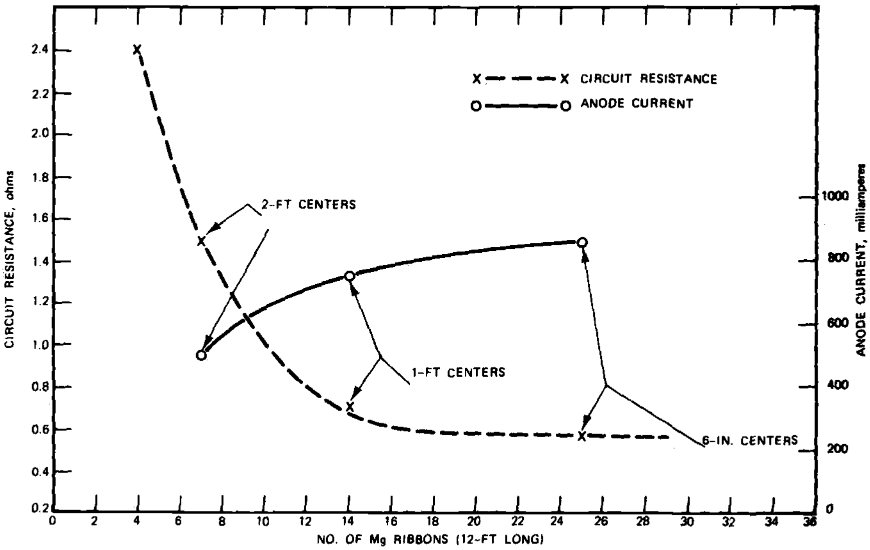


FIG. 11—*Analog model tests: effect of number of magnesium anodes on circuit resistance and anode current output.*

Prototype Deck Investigation

The investigation of electrical parameters using the analog model of a bridge-deck section gave indication of various cathodic-protection designs that have the potential of meeting the requirements for a technically and economically feasible cathodic-protection system for bridge decks. The subsequent step in the research program was a series of controlled tests of these designs with a realistic prototype of a typical bridge-deck section.

As with the analog, the scale and other important structural details for this deck, such as the size and location of the rebars, were typical of an actual bridge-deck section.

Four weeks after pouring the deck concrete, a salt treatment of the deck surface was started. The treatment procedure consisted of ponding a solution of 3 percent sodium chloride over the deck to a depth of $^{1}/_{16}$ in. each working day. This treatment was to cause salt intrusion into the concrete and was continued (about six weeks) until the start of the cathodic-protection testing.

Cathodic-Protection Testing

Distributed-Anode Method

The first cathodic-protection design tested on the prototype deck section consisted of an impressed-current system that depends exclusively on anode distribution (no conductive overlay) to give the necessary cathodic-protection current spread to the top rebar steel. The anodes used in this test were prepared from a continuous platinum-surfaced columbium wire of 0.062-in. diameter.

Four platinum/columbium wire anodes were installed about 0.5 in. below the deck surface in a 0.25-in.-wide groove on 4-ft centers. After the groove was cut, a small amount of cement mortar was placed in the slot. Next, the platinum/columbium wire anode was placed in the slot and held by tapping a small wooden wedge over the wire about every 2 ft. The photograph, Fig. 12, shows the anode being placed in the groove. Once the anode wire was in place, more cement mortar was added to cover the wire, although the slot was not completely filled. After the mortar dried overnight, each groove was filled, and the surface was smoothed with a polymer cement mix.

The contour maps in Fig. 13 are typical of those obtained with the platinum/columbium anode system. These maps show that the top deck area near the midpoint between the platinum/columbium anodes does not receive adequate current to achieve a polarized potential of −770 mV. The bottom measurements give a pattern similar to the top, with potential values several hundreds of millivolts lower than the top values. This demonstrates the problem of attaining uniform current flow in this cathodic-protection application.

During the test, it was noted that the current output with this system was very dependent on the moisture condition in the deck. Also, it was noted that the method of installation of the platinum/columbium anodes was not entirely satisfactory in that considerable oxygen and chlorine gas

FIG. 12—*Installation of the platinum-surfaced niobium wire anode into a groove cut in prototype bridge deck.*

were generated at the anodes. This gas caused some of the polymer cement mortar, and to a lesser extent the cement mortar, to be spalled off.

Thin-Conductive-Overlay Method

The second cathodic-protection design tested on the prototype deck section was an impressed-current system that utilizes a thin (between ⅜ and ⅝ in. thickness) electrically conductive asphaltic-concrete layer placed on the top of the deck. A 2-in.-thick asphaltic-concrete wearing course was then placed over the top of the conductive layer to protect it from traffic abrasion and to distribute wheel loads. The anodes used to introduce the cathodic current into the conductive layer were the platinum-surfaced columbium-wire type described earlier. Also, the spacing of the anodes on the prototype deck was the same (4 ft) as that in the previous test.

The electrically conductive asphaltic concrete for the test was mixed in the laboratory and formulated in accordance with the information developed during a previous coke aggregate/asphalt durability study. The asphaltic-concrete surfaces used during this project were constructed in a manner consistent with usual practice. As the conductive layer was spread

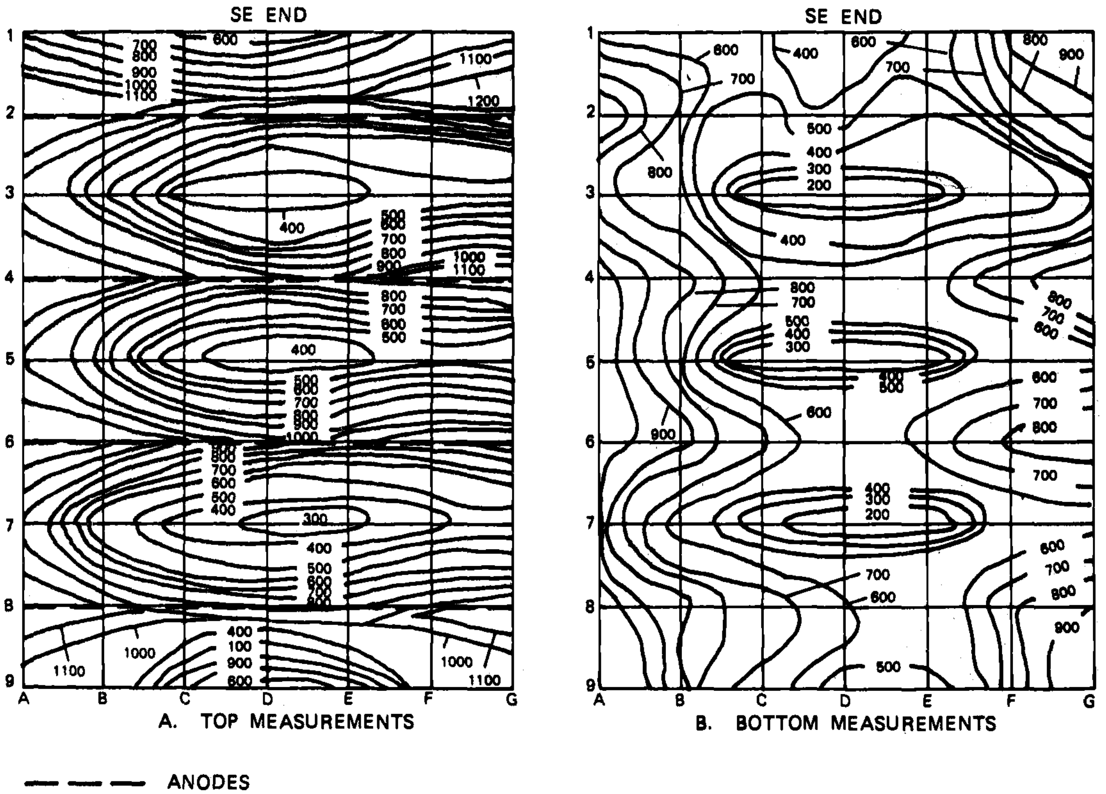


FIG. 13—*Equipotential contour maps of prototype model of bridge deck with cathodic protection applied at 40 V and 6.5 A. Four platinum-surfaced columbium-wire anodes as indicated (4-ft centers) on the 2-ft grid. Contour units are millivolts negative to* $Cu/CuSo_4$ *halfcell.*

over the prototype deck surface, the platinum/columbium wire anodes were placed within this layer of material. The conductive layer was then covered with a 2-in. (compacted thickness) layer of an asphaltic-concrete wearing course.

Overall, the results of tests with the thin-conductive-overlay method were markedly better than those obtained with the previous distributed-anode design. In general, the required current distribution and polarized potential on the deck were achieved with much lower operating voltages and currents than with the previous system. Further, the system's effectiveness did not depend on keeping the deck surface damp. As nearly as could be determined, neither the platinum/columbium wire anodes nor the conductive layer used in this test were adversely affected by the cathodic current voltage.

Conductive-Overlay Method

The third cathodic-protection system tested on the prototype deck is referred to as the conductive-overlay method, which, as has been mentioned, was proposed by R. F. Stratfull. This design is an impressed-current system that utilizes a 2-in. electrically conductive asphaltic-concrete layer placed on the top of the deck. As with the previous system, a 2-in.-thick asphaltic-concrete wearing course is then placed over the top of the conductive layer. To investigate different ways to introduce the cathodic-protection current into the conductive layer, two different anode arrangements were installed in this overlay system. These were (*a*) two platinum/columbium wire-type anodes positioned in the conductive layer, one 4 ft from each end of the deck (8 ft apart), and (*b*) a single 12-in.-diameter by 1½-in.-thick disk-shaped silicon-iron anode located midpoint on the deck top. The photograph, Fig. 14, shows the disk-shaped anode before it was covered with the conductive layer. Figure 15 shows a cross section of the overlay system and the relative thickness of the two asphaltic layers, as well as the position of the platinum/columbium wire anode in the conductive layer.

A series of potential-distribution tests was conducted with the silicon-iron anode located midpoint on the deck. The results, Fig. 16, show that a very uniform potential was achieved and that, based on a criterion of −770 mV, a rectifier setting of 7.2 V and 1.25 A gave complete protection of the deck surface. A second series of tests was conducted with one of the platinum/columbium wire anodes. A comparison of these results with those obtained with a single platinum/columbium wire anode in the ½-in.-thick conductive layer shows that the 2-in.-thick layer gave protection to an area of the deck of at least four times larger. The potential-distribution test was conducted by using both platinum/columbium wire anodes and shows that very uniform protection potential was attained

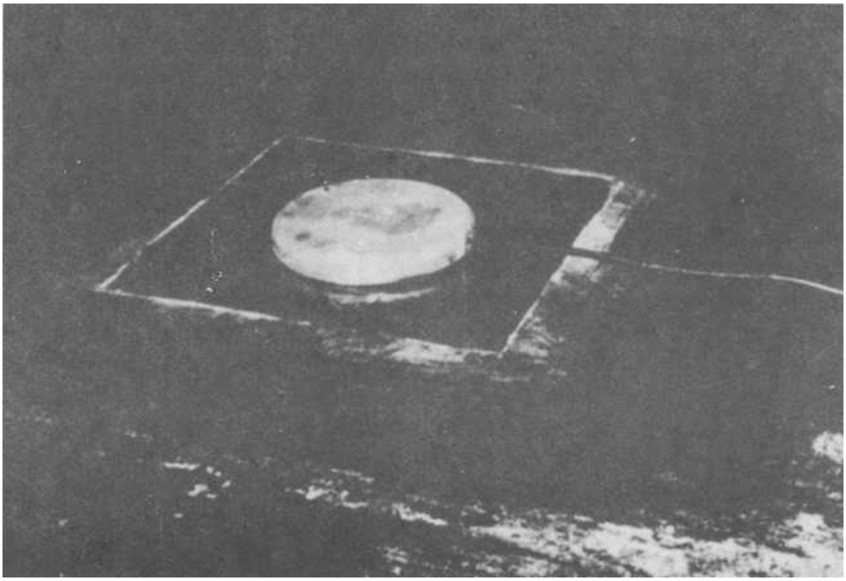


FIG. 14—*Disk-shaped silicon-iron anode and associated epoxy-coated deck surface before installation of conductive overlay.*

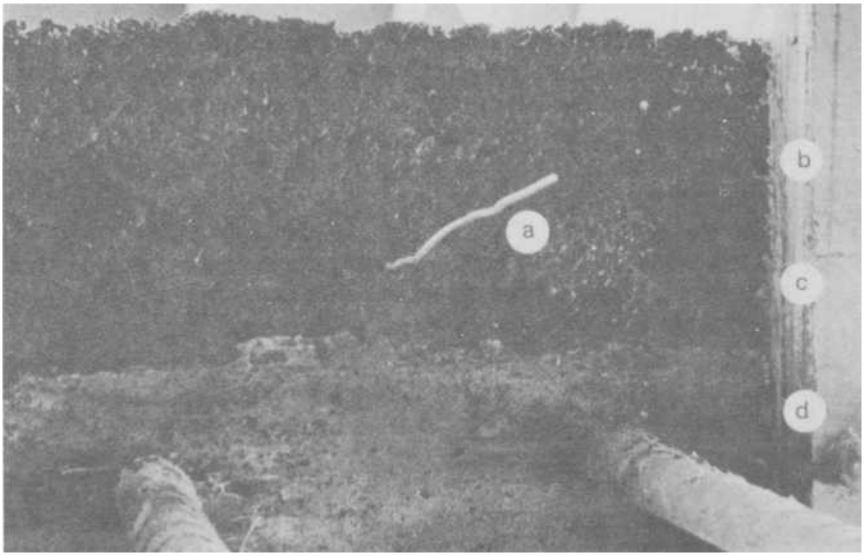
over the deck surface with an applied voltage at 2.5 V and current of 0.6 A.

In general, the 2-in. conductive-overlay method proved to be the most effective of the several impressed-current systems tested. Center-to-center anode distances of more than 20 ft are indicated, and there was no problem associated with gas blockage or deterioration of the anodes or the conductive layer.

Sacrificial-Anode Method

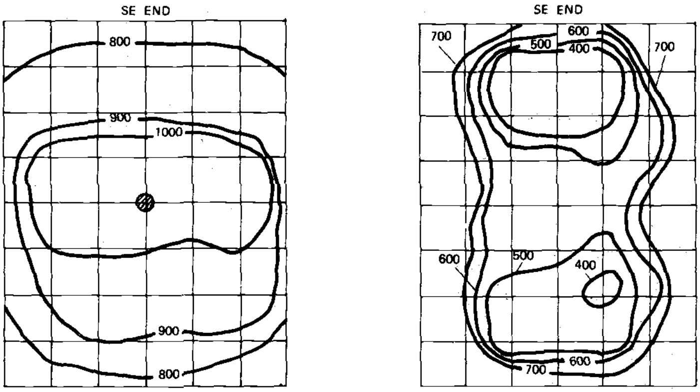
Several possible sacrificial-anode installation methods were evaluated in a prototype test. Because, under certain weather conditions, a limited amount of moisture would be available on a bridge deck, the test was conducted under two conditions: either very wet with chloride-bearing water or dry. Figure 17 illustrates the three installation methods used for sacrificial-anode testing. The ribbon types of anodes used have ben described previously. From Fig. 17, note that two of the methods proposed relate to the use of sacrificial anodes on an existing bridge deck, while the third relates to installation on a new or reconstructed bridge deck.

For the sacrificial-anode test arrangement, an anode spacing of 6 in. was selected. This provides one anode ribbon between each pair of rebars (top row). This spacing was indicated by studies on the analog model. The



(*a*) Wire-shaped anode.
(*b*) Asphaltic wear course Pennsylvania Department of Transportation ID-2A.
(*c*) Coke breeze aggregate-asphalt conductive layer.
(*d*) Concrete deck.

FIG. 15—*Section showing the conductive and wear courses of asphalt material used with conductive-overlay test on prototype deck. Approximately 2/3 actual size.*



A. TOP MEASUREMENTS

B. BOTTOM MEASUREMENTS

FIG. 16—*Equipotential contour maps of prototype model of bridge deck with cathodic protection applied at 7.2 V and 1.25 A. One silicon-iron anode located midpoint on the deck as indicated and in a 2-in.-thick conductive layer. Contour units are millivolts negative to $Cu/CuSO_4$ electrode plotted on a 2-ft grid.*

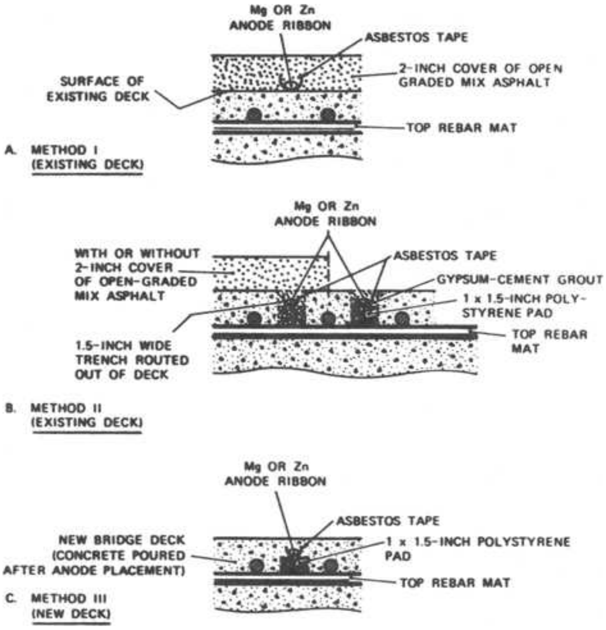


FIG. 17—*Sacrificial-anode-test installation methods.*

test arrangement was duplicated to accommodate testing of both zinc and magnesium anodes, and also to investigate conditions of a wet and a cyclic wet and dry surface on the deck over the anodes. For the wet condition, a 3 percent sodium chloride solution was ponded over the deck ($\frac{1}{16}$ in. pond depth) each working day. Generally, this treatment gave 1 to 3 h time with some free water lying on the deck, followed by 12 to 15 h with a damp deck surface. For the remaining period, the deck surface appeared dry. To simulate the wet and dry conditions, this sodium chloride treatment was made once each week. After four weeks, the surface-treatment procedure was stopped, and the effects of the deck drying out on the test results were observed.

Table 5 summarizes the current output from the anodes and the resultant polarized potentials on the rebars. Overall, the results showed that the 6-in. anode spacing gave about a 10 percent less current output because of mutual interference effects. In most cases, current output from a single anode in a given group compared well with current from other anodes in that group, reflecting a uniform and stable environment of the anodes and the rebar cathode in this test.

The current output from the magnesium anodes was higher than that from the zinc anodes. This was expected because of the higher (four times) driving voltage of magnesium as compared with zinc. It was noted

TABLE 5—*Summary of sacrificial anode test results.*

	Wet and Dry Surface Conditions							
	Mg Ribbon Anode				Zn Ribbon Anode			
	Dry		Wet		Dry		Wet	
Method No.	Avg *I*, mA[a]	Avg *E*, mV[b]	Avg *I*, mA	Avg *E*, mV	Avg *I*, mA	Avg *E*, mV	Avg *I*, mA	Avg *E*, mV
I	21.8	1170	23.3	1170	7.5	810	6.2	910
II[c]	32.1	890	32.1	920	3.0	430	2.7	475
II	13.9	580	14.7	670	0.4	350	0.5	410
III	4.2	405	4.0	485	0.1	310	0.2	390

	Wet Surface Condition			
	Mg Ribbon Anode		Zn Ribbon Anode	
Method No.	Avg *I*, mA	Avg *E*, mV	Avg *I*, mA	Avg *E*, mV
I	25.1	1230	2.6	920
II	26.1	1100	11.1	580
II	28.0	970	5.5	370
III	6.1	400	1.1	260

[a] Current per 5-ft anode ribbon.

[b] Potential measurement made with electrode at center of the rebar and close to rebar surface. Value is negative to $CuSO_4$ half cell.

[c] With asphaltic-concrete overlay.

that the current output (32 to 21 mA) from the magnesium was much greater than that from zinc anodes (6 to 3 mA) in each case with rebars with protective potentials. This self-limiting aspect prevents wasteful current production and gives the zinc anodes a more predictable life.

On the other hand, it can be said that the higher potential (−1800 mV) of magnesium compared with zinc (−1100 mV) is needed when circuit resistance is high. With Method II (embedded anodes), the current produced by magnesium was adequate for rebar protection, whereas the current from zinc was not.

Results obtained with Method III reflect a lack of salt penetration into the concrete. Potential values for the rebars in several cases did not change from their original value, and the others changed only slightly. The results for Method III are inconclusive.

The data show that two weeks of deck drying had little effect on the current input from the anodes and the rebar potentials. At the end of four weeks testing, the application of sodium chloride solution was stopped to investigate the effect of an extended drying time. No significant current or potential change occured during the first two weeks of no deck wetting.

Thereafter, both the current and rebar potentials dropped slowly. These results suggest that, during a prolonged dry spell, the anode current output can drop below the minimum required to provide cathodic protection, but if the deck is wet at least once every two to four weeks, the system should work.

With respect to physical damage, all systems performed well except Method I with magnesium anodes. It was believed that the porous nature of the asphalt would absorb the corrosion products. However, after about three weeks in test, heaving was noticed in the asphalt over the magnesium anodes (Method I). No signs of physical damage were noted with the zinc anodes installed by the Method I procedure, doubtless because the zinc corrosion products are more soluble than the magnesium corrosion products and were absorbed by the overlay.

Summary

Studies were made of factors affecting the cathodic protection of steel reinforcement in concrete and of possible methods of using this approach to control corrosion on highway bridge-deck reinforcement. Both laboratory and model testing were employed.

Laboratory studies of the electrochemical behavior of steel in simulated concrete environments, with and without chloride, demonstrated that corrosion can be controlled by the application of cathodic current to corroding steel. Under all conditions tested, it was found that corrosion was stopped at a steel polarized potential of −770 mV. Other studies showed that, at a potential of −1170 mV, hydrogen gas bubbles form at the steel surface.

Bond studies showed that the application of a cathodic-protection current to rebars in concrete can result in decrease in bond strength between the steel and the concrete. Evaluation of the results in relation to the anticipated conditions of cathodic protection applied to a bridge deck indicates that the decrease in bond strength would be slight and would not give reason to reject the use of cathodic protection on bridge decks. The results show that overprotection of steel in concrete should be avoided.

Two basic approaches to cathodic protection, the impressed-current and sacrificial-anode methods, were thoroughly investigated by using an analog model of a typical reinforced bridge-deck section. This investigation showed that the essential electrical parameter for cathodic protection of a bridge deck in either approach is uniform current flow of appropriate value from a well-distributed anode system located on the deck surface. Exhaustive tests on an impressed-current system using a soffit anode arrangement showed that this method is not economically practical because of the close spacing of anodes needed for adequate current distribution.

The most promising design consists of an inert anode placed in a conductive overlay covering the surface of the deck. The concept for this type of system was originated by the California Highway Department. The system described herein gave complete protection of the top rebar mat with low impressed voltage and total current. Asphaltic concrete made conductive by mixing coke particles within the concrete mix is the practical answer to providing a conductive layer for bridge decking.

A demonstration of the conductive-layer system on a full-scale prototype of a typical deck section was successful. Several different combinations of conductive-layer thickness and types of anodes were used. In general, a 2-in.-thick layer of conductive asphalt worked best. However, reduction in layer thickness was shown to be practical when the anode system is extended to give better distribution of current. A small-diameter platinized-wire anode was found to be suitable for this purpose.

The sacrificial-anode approach was also found to be promising in this study. A mockup demonstration showed that problems associated with low driving voltage and expansion of anode-corrosion products may be overcome by proper design and installation. The requirements for sacrificial-anode type, spacing, surrounding materials, and installation methods are identified.

On the basis of the information developed in this investigation, it is concluded that the cathodic-protection method of corrosion control can be successfully adapted to bridge decking. Both the sacrificial-anode and impressed-current methods show promise.

It is understood that the material in this paper is intended for general information only and should not be used in relation to any specific application without independent examination and verification of its applicability and suitability by professionally qualified personnel. Those making use thereof or relying thereon assume all risk and liability arising from such use of reliance.

Acknowledgments

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References

[*1*] Stratfull, R. F., *Materials Protection,* Vol. 7, No. 3, March 1968, pp. 29–34.

[*2*] Hausmann, D. A., "Steel Corrosion in Concrete—How Does It Occur," *Materials Protection,* Vol. 6, No. 11, 1967, pp. 19–23.

[*3*] Boyd, W. K. and Tripler, A. B., Jr., *Materials Protection,* Vol. 7, No. 10, 1968, pp. 40–47.

[*4*] Vrable, J. B., "Cathodic Protection for Reinforced Concrete Bridge Decks," NCHRP 12-13, National Cooperative Highway Research Program, July 1974.

[*5*] Scott, G. N., *Journal of the American Water Works Association,* Vol. 57, No. 8, Aug. 1965, pp. 1038-1052.

[*6*] Hausmann, D. A., *Materials Protection,* Vol. 8, No. 10, Oct. 1969, pp. 23-25.

[*7*] NACE Standard RP-01-69, Recommended Practice, Control of External Corrosion on Underground or Submerged Metallic Piping Systems, *Materials Protection,* Vol. 8, National Association of Corrosion Engineers, 1969.

[*8*] Rose, E. B., McCollum, Burton, and Peters, O. S., "Electrolysis in Concrete," *Bureau of Standards,* Technical Paper, BST 18, 1913.

[*9*] Mole, G., *Engineering,* 5 Nov. 1948, p. 453.

[*10*] Lappin, W. N., Department of the Army, Rock Island District, Corps of Engineers, letter to J. B. Vrable, U.S. Steel Corporation, 5 Dec. 1972.

[*11*] Ewing, S. P., "Effect of Cathodic Current on Bond Strength Between Concrete and Reinforcing Steel," Jersey Research Company, Report WC-IR-60, Jan. 1960.

[*12*] Kemp, E. L., Brezney, F. S., and Unterspan, J. A., *American Concrete Institute Journal,* Vol. 65, No. 9, Sept. 1968, pp. 754–756.

[*13*] Minsk, L. D., "Electrically Conductive Asphaltic Concrete," U.S. Patent 3,573,427, 6 April 1971.

P. M. Ward[1]

Cathodic Protection: A User's Perspective

REFERENCE: Ward, P. M., "**Cathodic Protection: A User's Perspective,**" *Chloride Corrosion of Steel in Concrete, ASTM STP 629,* D. E. Tonini and S. W. Dean, Jr., Eds., American Society for Testing and Materials, 1977, pp. 150–163.

ABSTRACT: This paper discusses the advantages and disadvantages of cathodic protection as a means of extending the life span of reinforced concrete bridge decks. The cost of cathodic protection is compared to the cost of other methods of rehabilitating such structures. Also included is a brief review and discussion of the data collected during the first year of operation of a cathodic protection installation on a reinforced concrete bridge deck with details on mix design of the coke breeze asphaltic concrete, results of resistivity, and voltage drop data taken through the coke breeze asphaltic concrete and the portland cement concrete, as well as actual corrosion criteria versus $E/\log i$ curves.

KEY WORDS: corrosion, concretes, reinforcing steels, cathodic protection

Interest in the area of chloride-induced corrosion of steel imbedded in concrete has been evident for over half a century [*1*].[2] But recent use of large quantities of chloride salts as deicing agents on highways has brought on an outpouring of interest as well as monies in this area of investigation. There are several proposed methods of preventing or minimizing the loss of structural usefulness caused by this corrosion, but this paper will present the advantages and disadvantages of only one of these methods, that being cathodic protection.

This paper will present a basic cost analysis upon which to base future cathodic protection installation decisions. In addition, some early results of a cathodic protection installation that the author designed will be presented. Throughout this paper, the term cathodic protection will apply only to impressed current cathodic protection.

It has been widely accepted that the normal passivation of steel imbedded

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[2] The italic numbers in brackets refer to the list of references appended to this paper.

in concrete can be broken down by a number of ionic substances, and that the chloride salts are among the more aggressive of these nonpassivating chemicals. With the addition of these salts, reductions in pH are followed or accompanied by less noble behavior of the imbedded steel as exemplified by more anodic potential readings and, ultimately, finite amounts of corrosion products formed on the rebar.

The approaches to solving this perplexing problem of chloride-induced corrosion are relatively few, and the proven solutions are even fewer. The areas that have been investigated to solve this problem are:

1. Forced migration of the chloride ion by electropotential gradient. Work in this area has been done by Slater et al at Battelle Institute [*2*] and by Morrison et al [*3*] at the Kansas Department of Transportation.
2. Cathodic protection used by the State Transportation Departments of California [*4*], Oklahoma, Nebraska, Missouri [*5*], Minnesota [*6*], and others.
3. Thin bonded concrete overlays, both modified and unmodified, used for many years with success by the State Departments of Transportation of Iowa [*7*] and Kansas [*8*] and recently by many others.
4. Lastly, attempting to tie up the chloride ion in a compound of low solubility.

Of these procedures, only cathodic protection and the use of thin bonded concrete overlays are at present field worthy.

Cathodic protection offers several advantages that make its use quite attractive.

1. The first of these advantages is that quite a large amount of research material is available on the principle, if not on its direct application to steel imbedded in concrete. After reviewing this research, it is very easy to become convinced that this procedure can indeed be applied to effectively protect the imbedded rebar from corroding, and that this protection would be essentially complete irrespective of the amount of chloride present. (Although this is true of the top reinforcing mat, protection of the lower mat may be much more difficult.)
2. The second advantage is that cathodic protection installations can be installed and maintained relatively inexpensively. The author designed and built a full-size cathodic protection installation at a cost of \$19/yd^2 (\$16/m^2) of bridge deck. The next such installation should cost about \$15/yd^2 (\$12.5/m^2), and, for longer bridges, the price should be even lower. As a yardstick, bid prices in Oklahoma for placement of 1½ in. of thin bonded portland cement concrete runs \$16/yd^2 (\$19/m^2); latex-modified concrete goes down for \$20/yd^2 (\$23/m^2); and complete deck removal and replacement runs \$50 to \$60/yd^2 (\$59 to \$71/m^2). These costs do not include any major patching of the deck; they only reflect the system costs. As can be seen, the cathodic protection installation costs are quite competitive. The

maintenance costs should also be relatively inexpensive. The cathodic protection system in Oklahoma has apparent protection with current densities of 3.0 mA/ft^2 (32.3 mA/m^2), based upon actual areas of rebar; this translates roughly to 6.0 mA/yd^2 (7.2 mA/m^2) of bridge surface for most of our bridge deck designs. For this protection, an anode to steel voltage of approximately 4 V is maintained. This yields an energy consumption of 24.0 mW/yd^2 (28.7 mW/m^2). Assuming 80 percent efficiency from the rectifier, consumption should be approximately 30.0 mW/yd^2 (35.9 mW/m^2) of bridge deck. At the current rate for electrical power in Oklahoma, this works out to be a monthly charge of 0.3¢/yd^2 (0.36¢/m^2) for short bridges and reduces to approximately 0.1¢/yd^2 (0.12¢/m^2) for longer structures. Of course, these are not the only costs; there are labor costs involved in the periodic monitoring of the equipment, and eventually the equipment as well as the overlay will have to be replaced. The average life of an overlay is ten to twelve years. It may be wise to use this figure to design for anode mass and plan to replace anode and overlay coincidently. This figure then could be used to prorate the cost of these consumables.

3. The third advantage is that, with suitable reference electrodes and corrosion indicating probes imbedded in the protected structure, the installations could be remotely monitored with no interruption of traffic flow.

4. The fourth advantage is that, with the use of the sensors just mentioned, a microcomputer could be obtained that would monitor the critical parameters of reinforcing steel potential and current density and then regulate the output of the power supply to provide adequate protective current automatically. This type of control lends itself easily to cyclical power-on and power-off installations that would be even more efficient from the standpoint of electrical power usage.

These are very powerful advantages, but the cathodic protection installations as we know them are certainly not without problem areas. The following problem areas are sites where future research efforts should be concentrated in order for cathodic protection to take its place in the relatively small field of proven solutions to the problem of corroding rebar.

1. The first and most disturbing of these problem areas is the development of a suitable low-resistance medium for distributing the protective current. The coke breeze asphaltic concrete overlays which are widely used have poor stability under traffic, and they bring with them the problem of very possibly accelerating freeze-thaw deterioration of the concrete deck. They also increase dead load and reduce clearance, for example, in truss structures and multilevel interchanges. In addition, in all likelihood, they will accelerate penetration of deicing salts to the bottom mat of steel, thus creating the additional and more difficult problem of protecting this mat.

2. The second problem area is obtaining a source of protection current in remote areas. Silicon solar cells are being used on at least one bridge to recharge the battery that supplies protective current [*9*]. These solar cells

are fairly inexpensive compared to total installation costs, provided that they have an adequate life span. Their cost is presently \$100 to \$200 per continuous watt. Utilization of this type of power source would add \$3 to \$6/yd^2 (\$3.6 to \$7.2/m^2) to the installation costs. Approximately 20 percent of the bridges in Oklahoma could be considered remote from an ac power source, and therefore, would be candidates for such a power source.

These two areas present stumbling blocks in the actual construction of a cathodic protection installation, but the following areas require further study to ensure that future installations are efficient from the manpower as well as electrical power standpoint.

1. Presently, polarized rebar potential is the yardstick that most researchers are using to determine if, in fact, the rebar is fully protected. Various criteria have been presented, but most commonly a polarized potential of 850 mV negative to the copper/copper sulfate half cell (throughout this paper, potential measurements are referenced to the copper/copper sulfate half cell unless so stated), or a 300 mV shift from current off to current on are the criteria used for protection. These criteria are conventions used primarily for steel buried in soil, and there is some research evidence to support the contention that steel imbedded in concrete may not require a polarized potential as negative as −850 mV [*4,10,11*]. More work is required toward developing the relationship that exists between polarized potential versus actual cathodic protection for steel inbedded in salt-laden concrete. This is important, not only for the fact that inordinately large negative values for this potential would make installations and maintenance needlessly expensive, but could also contribute to cation migration and softening of the concrete in the area of the rebar [*12–14*]. The increased concrete permeability found by Morrison et al [*2*] during their forced migration work is another reason to hold potential and current to the minimums required for complete protection.

2. A second area that needs research is the development of a reference half cell that can be imbedded in concrete at the steel level and remain stable under varying moisture and chloride contents while at the same time not altering the ionic content of the concrete surrounding it. This reference cell would permit the remote sensing mentioned previously and is essential for a self-regulated system.

These problem areas do not prevent working cathodic protection installations from being installed and operated, but they markedly reduce the efficiency of the operation.

From a user's perspective, the future of the use of cathodic protection on reinforced concrete structures appears bright indeed. The author designed and constructed such an installation approximately 18 months ago. A detailed discussion of this installation follows.

After a year of literature study, laboratory work, and design, the search began for a suitable structure upon which to place a full-size cathodic protection installation. Because of the desire to leave the coke breeze asphalt overlay exposed for ease of data collection, and armed with other experimenters' reports of poor stability in these overlays [*15*], an effort was made to locate a structure that carried low traffic volumes. Such a structure was found in one of Oklahoma Department of Transportation's field maintenance facilities. The structure was a three-span structure, 120 by 20 ft (36.6 by 6.1 m), of T-beam design that was built in 1927 and removed from the highway system in 1969 due to route relocation. The structure carries only the vehicles used by the maintenance facility, and it was felt that the exposed coke breeze asphalt would stand up under these low traffic volumes.

The type of cathodic protection installation chosen was of the conventional design utilizing 12 by 1½ in. (30.5 by 3.81 cm) high-silicon cast iron anodes cemented to the bridge deck with current supplied through a constantly charged wet cell and distributed across the deck with a coke breeze asphaltic concrete.

Procedure

Prior to placement of the coke breeze overlay, the deck was routinely surveyed to determine half cell potential of the reinforcing steel, chloride content of the concrete, and concrete cover over the top steel mat. At the same time, the structure was visually surveyed to locate any exposed steel that might act as a short to the coke breeze asphaltic concrete.

The half cell survey pointed out only four readings in excess of 300 mV negative to the copper/copper sulfate electrode. Three of these four were along the curb line; the fourth was adjacent to a transverse crack. The chloride content survey indicated only one area out of the eight areas sampled that contained chloride ion in excess of 2 lb/yd^3 (1.179 kg/m^3) of concrete which is considered by many to be the chloride content threshold for corrosion. This area is an area where half cell readings were more negative than −300 mV. Concrete cover averaged 2⅝ in. (6.67 cm) with rebar depths ranging from 2½ to 4½ in. (6.35 to 10.32 cm).

Several areas along the curb face had exposed reinforcing, and all drains were exposed cast iron that were grounded to the rebar mat. The exposed steel in the curb face was coated with a general-purpose epoxy, and the areas near the drains were excavated of conductive asphaltic concrete after the laydown operation and backfilled with ordinary nonconductive asphaltic concrete.

The anodes were epoxied along the bridge's centerline at a 20-ft (6.096-m) spacing, beginning 10 ft (3.048 m) into the bridge. Holes were cored in the deck to the level of the rebar mat in four locations, and electrical resistance corrosion probes were placed in these holes and then backfilled with portland cement concrete to which 5 percent sodium chloride was added based on the cement content. The wiring from these anodes and probes was routed underneath the bridge through holes drilled adjacent to the respective anode or probe, and from there they were routed to a control box near the south end of the structure. Figure 1 presents a diagram of the location of the cathodic protection appurtenances.

The coke breeze asphaltic concrete was made up of approximately 60 percent bituminous coke breeze, 40 percent mineral aggregate, and 13 percent asphalt cement. Table 1 contains the gradation and mix design. The stability as determined by the Hveem method (ASTM Tests for Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus (D 1560-76)) was 33, and uncompacted thickness was 4 in. Batching and placement of the overlay went relatively uneventfully, but difficulty was encountered during the compaction. Severe rutting of the overlay occurred when rolling was attempted before the overlay temperature dropped to 150°F (65.5°C). If rolling commenced at asphalt tempera-

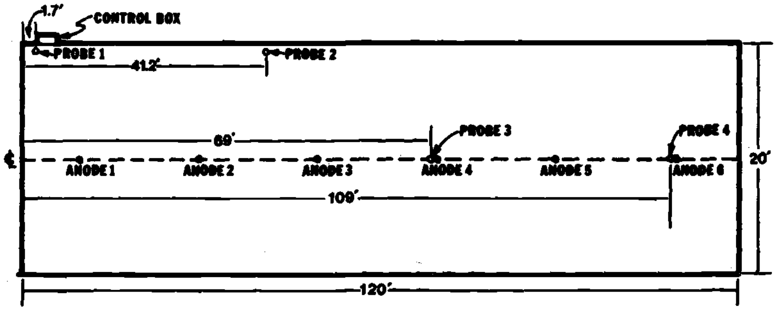

FIG. 1—*General bridge layout.*

TABLE 1—*Coke breeze asphaltic concrete mix design.*

Sieve No.	Aggregate Percent Passing
½ in.	100
⅜ in.	99
No. 4	79
No. 10	49
No. 40	17
No. 80	6
No. 200	1.7
A-C3, %	12.8
Bulk impregnated specific gravity	1.854
Hveem stability value	33

ture of 150 to 160°F (65.5 to 71°C) and followed the normal sequence of initial rolling with a steel wheel roller followed by pneumatic rolling and a final rolling by a steel wheel, then the compaction problems were greatly alleviated. A large amount of our bituminous coke breeze was wasted while calibrating the asphalt plant, and we obtained, as backup, material from a local supplier of petroleum coke breeze. This material was much more porous than the original material, and the one load of asphaltic concrete utilizing this material proved to be much too low in asphalt cement to provide stability. In comparing this mix design with that of Spellman's [*16*], it appears the design would have benefited from a 2 to 3 percent increase in the amount of asphalt cement. Eventually, an adequate surface was obtained over all the structure except where the petroleum coke breeze had been used; this area remained unstable.

Several days after the placement of the overlay, 1¼-in. (3.75-cm)-diameter holes were drilled through the overlay and the concrete to the level of the previously buried corrosion probes. These holes dead ended approximately ¼ in. to the south of each probe and at the same level as the probe. A polyvinyl chloride (PVC) pipe was epoxy cemented into this hole. Adjacent to this hole, another was drilled, but this one was drilled only through the coke breeze overlay. Another PVC pipe was epoxied here. These pipes allowed entry of the copper/copper sulfate reference cell in order that potential measurements could be taken at the level of the reinforcing steel in the near vicinity of the electrical resistance corrosion probe and also upon the concrete bridge deck. These four corrosion probe sites are referred to as probe locations in the body of this paper and only as probes in the figures. Close-fitting caps were placed on these pipes to prevent intrusion of debris or rainwater.

Protection current is supplied through a constantly charged 12-V wet cell with power potentiometers to control total applied voltage and individual anode voltage. The current is essentially dc with a 50-mV full-wave ripple imposed on it from the charger. Circuitry is provided to measure total applied voltage and current, individual anode voltage and current, corrosion probe readings and current impressed on these probes. Data on all these functions as well as half cell potentials at the four probe sites have been taken weekly since July 1975.

Results

Coke Breeze Conducting Layer

The in-place resistivity of the coke breeze asphalt is 3 to 6 Ω-cm, as measured by the Wenner four-pin method. Initially, only one anode was driven in order that the relationship concerning voltage drop versus distance from the anode could be developed. Figure 2 is a plot of the data

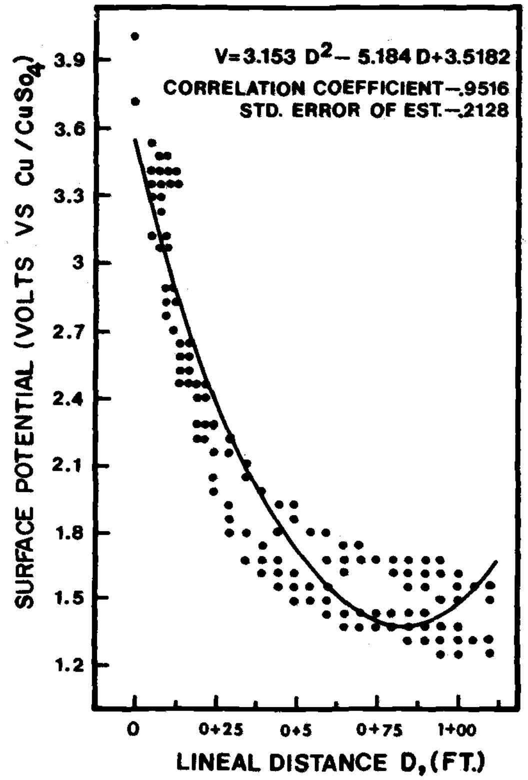

FIG. 2—*Surface potential profile. One anode driven.*

collected on two half cell surface surveys taken approximately six weeks apart and consists of 350 data points. The parabolic curve fits the aggregate data adequately at distances from the anode of 60 to 70 ft (18.3 to 21.3 m) or less. This distance should be greater than the practical radius of protection for this type of installation.

It is not the intent of this paper to suggest that this curve represents the underlying relationship. It is simply the one that fits best out of the several we have tried.

The equation for this best fitting curve is

$$V = 3.153D^2 - 5.184D + 3.182$$

where V = potential (volts versus copper/copper sulfate) and D = distance from the anode, ft.

More recently, three anodes have been driven, giving rise to a surface potential versus distance along the structure curve as shown in Fig. 3. Ob-

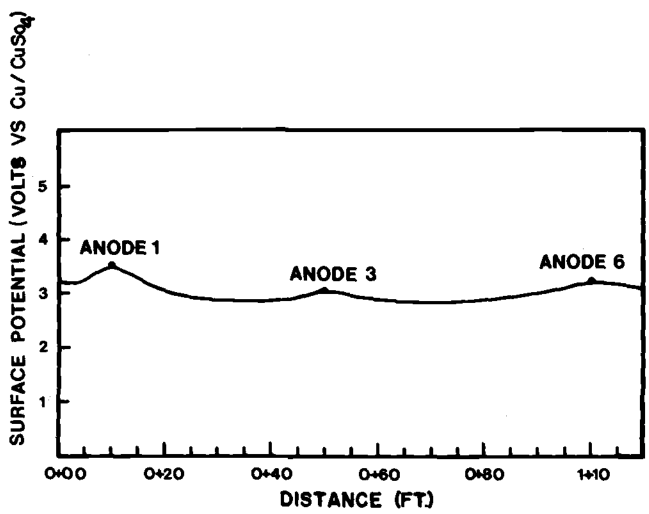

FIG. 3—*Surface potential profile. Three anodes driven.*

viously, driving three anodes gives a much better distribution of potential along the bridge surface.

At three of the four probe locations, voltage drops were measured vertically through the coke breeze overlay and through the concrete to the level of the upper mat of steel. These voltage drops were determined by taking potential measurements on the overlay surface, on the concrete deck surface through the previously mentioned drilled hole, and in the concrete deck at the level of the top mat of reinforcing, through an adjacent drill hole. The results of the voltage drop measurements through the coke breeze are shown in Fig. 4. The total voltage drop was divided by the overlay

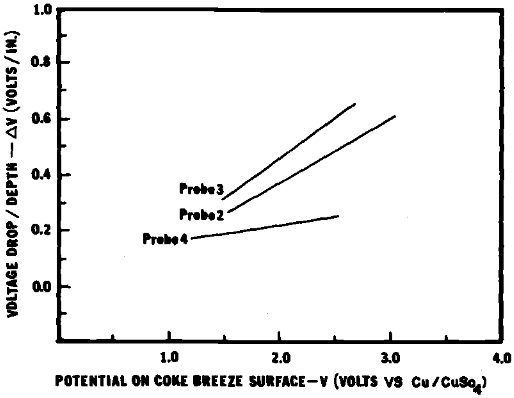

FIG. 4—*Voltage drop through coke breeze asphaltic overlay.*

thickness to reduce the voltage drop to a per inch basis. This was required because of the variability of the overlay thickness between probe locations.

The upper section of Table 2 contains the equations of the lines and also the correlation coefficient and standard errors of estimate for these lines.

From the information in Fig. 4 and Table 2, it appears that the voltage drops at Probe Locations 2 and 3 agree quite well in the statistical data, both having good correlation coefficients with small standard errors of estimate. They also have slopes that are very similar and fall out of agreement only in their relative displacement along the y-axis. The data from Probe Location 4 have poor agreement with the data taken from these other two areas. The line itself has a much shallower slope, although its y-axis displacement is not unlike that of the other two locations. The statistical data are greatly different, having a poor correlation coefficient and a relatively large standard error of estimate. The poor statistical data are rather a perplexing disparity, particularly in the light of the overall voltage drops that will be presented later in this paper.

TABLE 2—*Probe location voltage drop equations.*

Voltage Drop Through	Probe	Equation	Standard Error of Estimate	Correlation Coefficient
Coke breeze	2	$\Delta V = 0.224V - 0.0769$	0.0211	0.9612
	3	$\Delta V = 0.248V - 0.1125$	0.0189	0.9722
	4	$\Delta V = 0.063V - 0.0972$	0.0383	0.5968
Concrete	2	$\Delta V = 0.260V - 0.1622$	0.0092	0.9493
	3	$\Delta V = 0.242V - 0.1206$	0.0294	0.5412
	4	$\Delta V = 0.207V - 0.0921$	0.0082	0.9945
Coke breeze and concrete	1	$\Delta V = 0.271V - 0.2080$	0.0155	0.9891
	2	$\Delta V = 0.149V - 0.0751$	0.0039	0.9968
	3	$\Delta V = 0.147V - 0.0693$	0.0138	0.9588
	4	$\Delta V = 0.115V - 0.0400$	0.0043	0.9964

Voltage Drop Through the Concrete

At Probe Locations 2, 3, and 4, voltage drops through the concrete to the level of the upper mat of reinforcing were taken in a like manner to the coke breeze voltage drops just presented. The curves for these data are presented in Fig. 5 with the equations for these curves and statistical data listed in the middle section of Table 2. It is evident that the voltage drop through the concrete is much more uniform from location to location than the voltage drop through the coke breeze. The high correlation coefficients and small standard errors reflect good predictability at Probes 2 and 4. Probe 3 data, though not as predictable as Probes 2 and 4, were slightly improved from the data presented here because of a data collecting error

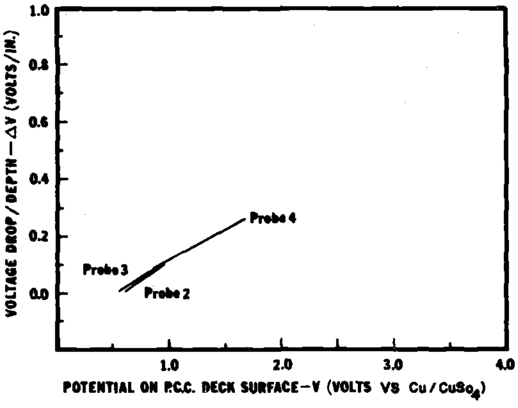

FIG. 5—*Voltage drop through concrete.*

found too late to be reflected in these tables. The relatively good predictability with widely varying conditions of temperature and moisture content of the concrete is a bit unusual in light of work done by Locke [*17*], which indicated changes in resistivity of dry to wet concrete of approximately one order of magnitude.

Voltage Drop Through Overlay and Concrete Cover

At all four probe locations, data were taken for total voltage drop through coke breeze and concrete. The curves resulting from the collection are presented in Fig. 6. The equations and statistical data are presented in the bottom section of Table 2. These curves, with the possible exception of the one for Probe 1, contain few surprises and could have been predicted

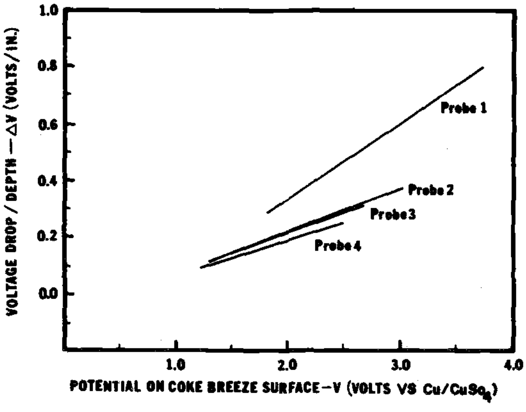

FIG. 6—*Voltage drop through overlay and concrete cover.*

from the curves in the two previous sections since they are essentially the combination of these. What is unusual is the relatively poor statistical data associated with one curve in each of the two previous sections are not reflected here.

Potential and Current Density at Steel Level

Current density on the corrosometer probes was measured with a zero resistance ammeter patterned after the one described by Henry and Wilde [*18*]. This current density was plotted against potential at the probe level to obtain the familiar *E*/log *i* curves shown in Fig. 7. The data range along the log *i* axis was not great enough on the individual probes to obtain individual plots; therefore, the data from all probes were utilized to develop this plot. The scatter is considerable, but a general trend toward linearity appears to begin in the area of 600 to 700 mV and current densities of 1 to 10 mA/ft^2 (10.8 to 108 mA/m^2). This is in fair agreement with data collected from the electrical resistance changes of the imbedded probes. These data indicate corrosion on the probes halted at potentials of 700 mV with current densities of 3 mA/ft^2 (32.3 mA/m^2).

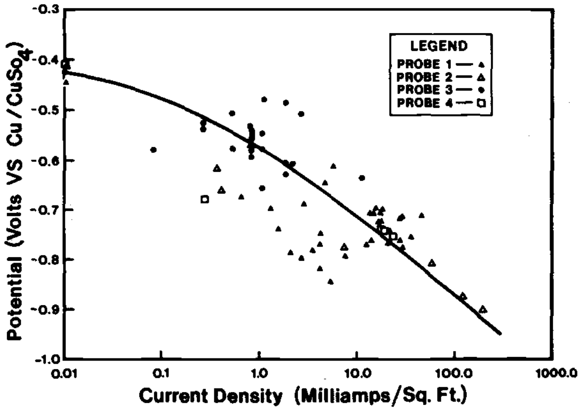

FIG. 7—*Potential and current density at steel level.*

Discussion

Predictions of potential at the rebar level from surface potentials are not possible primarily because of the poor site to site correlation of voltage drop through the coke breeze asphaltic concrete. This is in marked contrast to the uniform site-to-site voltage drops through the portland cement concrete. From the voltage drops versus distance data, the bulk resistivity of the coke breeze appears to be relatively uniform. The solution of this apparent disparity may be inconsistencies at the coke breeze concrete interface. At any rate, accurate predictions of steel level potential can be made from the concrete surface as long as the thickness of concrete cover is fairly uniform.

None of the potential data taken during the course of this study indicate that protected potential values need be higher than 700 to 750 mV negative to copper/copper sulfate. However, it must be borne in mind that these data, although interesting and worthy in their own right, should be approached with trepidation if analogies are to be drawn to the whole of this or any other structure. Much of these data were obtained from an artifical environment (that of the corrosion probe and backfill portland cement concrete) that may not hold absolutely true for the body of this or other structures, particularly older ones where differences in resistivity and aggressiveness of electrolyte are to be expected. Certainly, more work needs to be done to determine how closely these data, obtained artifically, correlate to a variety of bridge decks.

Conclusions

1. Voltage drop with distance from the anode is approximated by a parabolic function and can be predicted with good accuracy to a distance of 70 ft (21.3 m) for coke breeze asphalts with this range of resistivities.
2. Rebar level potentials will be difficult to predict from coke breeze surface potentials primarily because of the poor predictability of voltage drops through the coke breeze overlay. Additional work in the area of imbeddable reference electrodes is needed to provide the vital information in this area.
3. Measurements of protected potential presented in this paper support the contention that 850 mV may provide overprotection for steel imbedded in chloride contaminated concrete.

References

[*1*] Wig, R. S. and Ferguson, L. R., *Engineering News Record*, 1917, pp. 79, 689.

[*2*] Slater, J. E., Lankard, D. R., and Moreland, F. J., "Electrochemical Removal of Chlorides from Concrete Bridge Decks," Battelle Columbus Laboratories, 1975.

[*3*] Morrison, G. L., Virani, Y. P., Stratton, F. W., and Gilliland, W. J., "Chloride Removal and Monomer Impregnation of Bridge Deck by Electro-Osomis," Interim Report, Kansas Department of Transportation, 1976.

[*4*] Stratfull, R. F., "Experimental Cathodic Protection of a Bridge Deck," California Department of Transportation, 1974.

[*5*] Girard, R. S., "Missouri's Experience with Cathodic Protection," paper presented at the first meeting of Cathodic Protection for Reinforced Concrete Bridge Decks, Kansas City, Mo., 1976.

[*6*] Tracy, R., "Construction of a Cathodic Protection System," paper presented at the first meeting on Cathodic Protection for Reinforced Concrete Bridge Decks, Kansas City, Mo., 1976.

[*7*] Iowa Method Bridge Deck Repair and Membrane Systems Demonstration, July 1974.
[*8*] Stingley, W. M., personal communication, 1974.
[*9*] Corum, D., "Design Criteria and Electrical Equipment for a Cathodic Protection System," paper presented at the first meeting on Cathodic Protection for Reinforced Concrete Bridge Decks, Kansas City, Mo., 1975.
[*10*] Locke, C. E., "Specialized Studies Cathodic Protection of Rebars in Bridge Decks," final report, in cooperation with the Oklahoma Department of Highways, 1975.
[*11*] Hausmann, D. A., "Criteria for Cathodic Protection of Steel in Concrete Structures," *Materials Protection*, Oct. 1969.
[*12*] Gent, W. C. and Mole, G., "Electrolysis and the Bond Strength of Reinforced Concrete," British Electrical and Applied Industrial Research Association, Technical Report, 1947.
[*13*] Mole, G., "Electrolysis and the Bond Strength of Reinforced Concrete: The Effect of Direct Current," British Electrical and Applied Industrial Research Association, Technical Report, 1947.
[*14*] Casad, B. M., "The Effect of Cathodic Current on Bond Strength Between Concrete and Reinforcing Steel," thesis submitted to Oklahoma State University, Aug. 1957.
[*15*] Wilson, C. P., Ontario Ministry of Transportation and Communications, personal communication.
[*16*] Spellman, S., "Characteristics and Design of Asphaltic Coke Breeze," paper presented at the first meeting on Cathodic Protection for Reinforced Concrete Bridge Decks, Kansas City, Mo., 1976.
[*17*] Locke, C. E., personal communication, 1976.
[*18*] Henry, W. D. and Wilde, B. E., "An Electronic Zero Resistance Ammeter with Instantaneous Null Characteristics," *Corrosion NACE*, National Association of Corrosion Engineers, Vol. 27, No. 11, Nov. 1971.

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Development of a Cold-Poured Bridge Deck Membrane System

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ABSTRACT: A two-component asphalt-extended urethane membrane was developed which satisfies the requirements demanded of a waterproofing membrane for protecting reinforcing steel in concrete bridge decks from corrosion. A technique for application of the membrane is described which essentially eliminates blister formation. Blisters caused by thermal expansion of gases in the concrete have been a constant problem with liquid-applied membranes. The technique consists of: applying a black prime coat to the deck, allowing the black-primed deck to heat up by absorbing radiation from the sun, and applying the membrane after the maximum average deck temperature has been reached. The black prime increases the absorptivity of the deck, causing it to heat up more than an unprimed deck. It thus avoids the sudden increase in temperature caused when a dark membrane is applied over a white deck on a sunny day. Then, by applying the membrane after the sun has passed its zenith and the deck temperature has started to fall, blister formation caused by the outgassing of the concrete deck is avoided. This technique works equally well on damp or dry decks. The system is completed by applying a tack coat of hot asphalt and a 2-in. overlay of asphalt concrete.

KEY WORDS: corrosion, concretes, reinforcing steels, membranes, waterproofing, bridge decks, corrosion prevention, urethanes, blistering

The need for protecting reinforcing bars in concrete bridge decks from corrosion by the action of deicing salts has been discussed in numerous papers [*1–4*].[4] Several approaches are being used; but currently the most widely used method for protecting bridge decks is the application of a

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[4] The italic numbers in brackets refer to the list of references appended to this paper.

waterproofing membrane, either preformed or poured in place, followed by a protective overlay of asphalt concrete [*5*].

Advantages of a poured-in-place system include: seamless installation, no gaps or fishmouths, no cutting to fit irregular shapes, simple installation on curved decks, and no adhesive bonding required. In addition, a poured-in-place system can provide corrosion protection for metal drain systems.

The Waterproofing Membrane

The membrane we developed is a cold-applied, two-component asphalt-extended urethane. The two components, mixed in a 9/1 volume ratio, have the proper viscosity to be self-leveling and yet not sag on a 5 percent grade. Working time (pot life) of ½ h at 77°F (25°C) and 20 min at 90°F (32.2°C) is adequate for mixing and application when good techniques are used. The membrane can be walked on in 2 to 4 h, depending on the temperature, and can be paved over with hot asphalt concrete in 12 h. A tack coat of hot AC-20 (AR 4000 or 85/100 penetration) grade paving asphalt is used over the membrane to ensure good adhesion to the overlay.

Typical physical properties of the membrane developed for this use are given in Table 1. Physical properties compare favorably with the best of the materials tested in the recent study by Van Til, Carr, and Vallerga [*4*] covering more than 90 membranes. Our membrane was designed to have good flexibility at the lowest temperatures normally encountered and to resist flow at 300°F (149°C) during application of the asphalt concrete overlay. For example, the high extensibility and good low-temperature flexibility of the membrane permit it to pass the crack opening test at 0°F (−18°C) described by Van Til, Carr, and Vallerga [*4*].

The membrane can be applied either by squeegee or two-component spray equipment. After application, the membrane can easily be tested for effectiveness by use of electrical resistance measurements developed in California [*6*] and in Colorado [*7*].

The Blister Problem with Poured-in-Place Membranes

In spite of a number of advantages inherent in liquid-applied membranes, their widespread adoption has been retarded by the persistent problem of blister formation when applied to concrete in the sunshine. These blisters can take several forms but most frequently appear as hemispheres, often reaching the diameter of golf balls. Sometimes they show up as numerous small pimples about ¼-in. (6.4 mm) in diameter. Other times, strings of bubbles appear, linked together, floating along the top of the membrane, and usually culminating in a blister. Figure 1 gives a good

TABLE 1—*Typical properties of two-component urethane membrane.*

Liquid components	
viscosity, cP, 77°F	
premix	7000
activator	30
mix ratio by volume	
premix/activator	9/1
pot life (time to 100 000 cP)	
min (at 77°F)	32
set time, h (at 77°F)	4
flash point	
premix, ASTM Test for Flash Point by Tag Closed Tester (D 56-70) (TCC), °F	190
activator, ASTM Test for Flash and Fire Points by Cleveland Open Cup (D 92-72), (COC), °F	425
Cured membrane	
peel strength from concrete, 90-deg peel, 2 in./min, lb/in. (Federal Specification TTS-00227)	15+
hardness, Shore A, ASTM Test for Rubber Property—Durometer Hardness (D 2240-75)	55
tensile strength, psi } ASTM Test for Tensile Properties of Plastics (D 638-76)	375
extension to break, %	450
split tear strength, 20 in./min	40
crack bridging at 0°F on concrete block	pass crack opening to 0.25 in. without splitting
high-temperature flow test	no flow at 300°F for 24 h in a vertical position

example of this type of defect. All of these types of blisters show up either during application of the membrane or during the curing stage.

Bridge deck concrete is purposely made with about 6 percent air entrainment to reduce surface scaling and freeze-thaw damage. Consequently, it contains thousands of narrow passageways through which air slowly moves in and out as the temperature of the deck changes. Any dark-colored, liquid, film-forming material applied over such a surface in sunshine or in a rising temperature will form blisters. These blisters in fresh poured-in-place membranes are caused by outgassing of the concrete deck, primarily due to thermal expansion of entrained air. Two factors can cause this expansion: (*a*) a rising ambient temperature or (*b*) the increase in deck temperature caused by increased absorption of solar radiation upon application of a black or dark-colored membrane.

The pressures developed underneath a dark-colored membrane applied over a cold deck could be appreciable. For example, a concrete bridge deck could easily cool to 50°F (10°C) in the early morning hours. A black membrane applied about 8:00 a.m. could warm the concrete deck to 140°F (60°C) in the sunshine, a rise of 90°F (50°C). If the air trapped in the deck could not escape, it would develop an excess pressure of $(273 + 60)/(273 + 10) - 1 = 0.177$ atm (17 935 N/m^2).

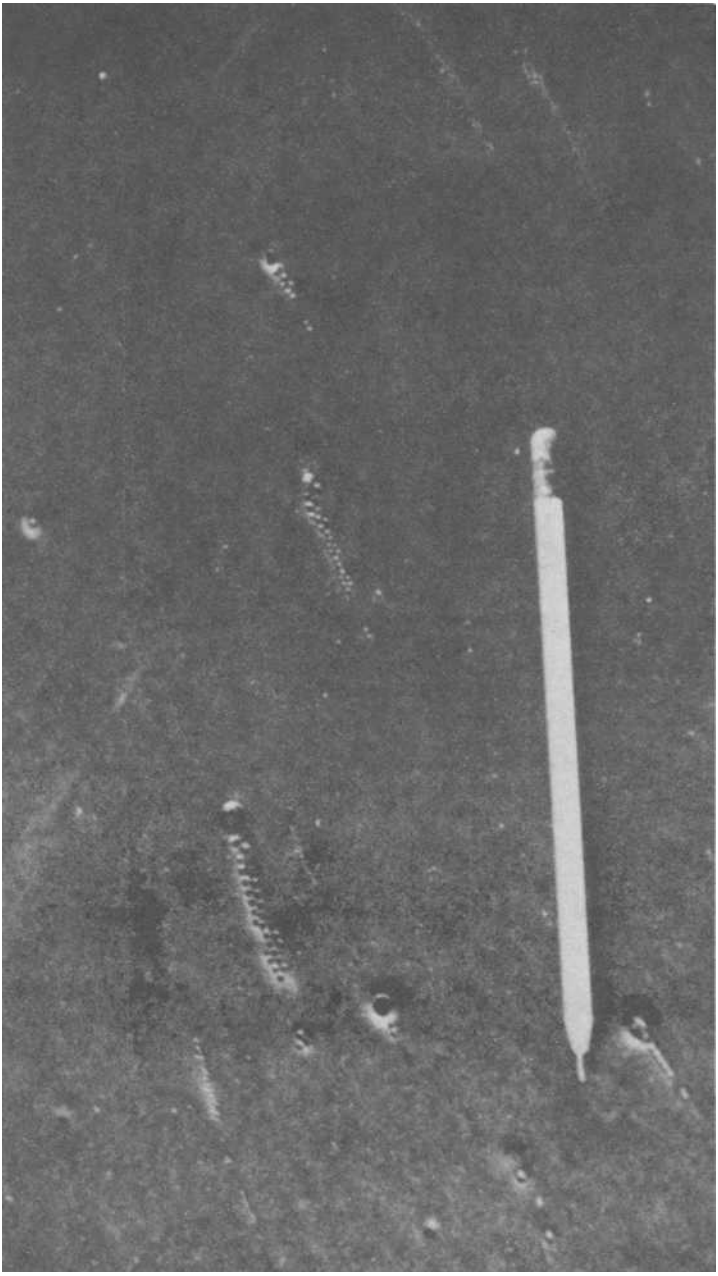

FIG. 1—*Strings of bubbles occurring in membrane applied over a damp deck.*

In an actual bridge deck, air can escape by diffusing in all directions through the concrete. However, because the greatest temperature increase, and thus the greatest air expansion, will be near the top surface, most of the air will escape here. As long as the membrane is liquid, it will offer little resistance to the expanding air.

Each square foot (929 cm^2) of a bridge deck 8 in. (20.3 cm) thick, constructed of concrete with 6 percent air voids, contains 69.11 $in.^3$ (1133 cm^3) of free volume. When the temperature of this section of deck is raised by 90°F (50°C) as just described, the air will expand, and 12.2 $in.^3$ (200 cm^3) will escape. If the membrane is still fluid enough to heal itself, no holes will be formed. As the membrane cures, it becomes more viscous, and the rising air bubbles can no longer escape, but become entrapped and remain as pimples or blisters. The size and stability of the blisters can increase if a skin forms on the membrane surface. When the membrane is fully cured, it has strength enough to resist the pressure of the expanding air, and no further blistering takes place, provided it maintains good adhesion to the deck.

Water or water vapor is not a necessary requirement for blister formation but can be a strong contributing factor. Consider the same small section of bridge deck with 6 percent voids but now containing 4 percent by weight of water (4 lb; 1816 g). A temperature rise of 90°F (50°C) will again produce a severe blowing action; but now instead of only 12.2 $in.^3$ (200 cm^3) of air, there will be a mixture of water vapor and air. If as little as 0.04 percent water evaporates at a temperature of 140°F (60°C), it will produce 1623 $in.^3$ (26 590 cm^3) of water vapor $[((18.16\ g)/18\ g/mole) \times (22\,400\ cm^3/1\ mole) \times (333\ K/283\ K)]$.

Blisters Caused by Poor Adhesion

Blisters of a different sort can form if the membrane (either preformed or poured in place) has poor adhesion to the deck. These blisters are much larger than those formed during application of a liquid membrane. They can form beneath an uncovered membrane during placement of the asphalt concrete overlay or later during hot sunny days. Beijers [*8*] of the Delft University of Technology, Netherlands, has described these blisters, created them artifically in the laboratory, and analyzed quantitatively the forces which cause them. His findings can be summarized by saying that blistering can occur if the pressure of the expanding gases in the concrete exceeds the combined forces of adhesion to the deck and the weight per unit area of the total waterproofing system. The nature and extent of the blisters will be influenced by the uniformity of adhesion and by the stiffness and viscoelastic properties of the membrane and overlay.

As we pointed out previously, significant pressures can easily be developed merely from the heat of the sun under membranes applied to a dry deck. Consequently, if adhesion of the membrane is poor, there is a good chance that blisters of this type will be caused by solar heating. During application of hot asphalt concrete at 300°F (149°C), even greater pressures can be developed. Fortunately, the asphalt concrete usually cools rapidly. Nevertheless, the potential danger at this stage of construction emphasizes the need for excellent adhesion of membrane to the deck.

Since our membrane does have excellent adhesion to the deck, we have concentrated on the problem of blister formation during application. Our attempts to prevent blistering have followed the following routes: (*a*) sealing the pores in the deck completely, and (*b*) applying the membrane in a falling temperature situation.

Sealing the Deck to Prevent Blisters

If the pores in the concrete were completely sealed so that air could not escape, no blisters would form when a membrane is applied. Over completely nonporous substrates, such as steel or glass, the membrane does not blister. However, sealing the pores on a concrete bridge deck is difficult.

A laboratory test procedure was developed to determine the effectiveness with which various types of materials seal concrete pores. These included urethanes, asphalt emulsions, cutback asphalts, rubber latexes, and a number of polymers in organic solvents. Sealers were applied by brush, squeegee, or roller to blocks of bridge deck formula concrete about 2 ½ in. (6.4 cm) thick and allowed to cure overnight. The two-component urethane membrane was then applied at the rate of 0.56 gal/yd^2 (2.54 litres/m^2), about 100 mils (2.5 mm) thick, and heated for 1 h with an infrared lamp held 15 in. (38 cm) above the surface to simulate heating by the sun. Failure to seal the surface was immediately evident as air in the concrete expanded, came up through the pores, and blew bubbles in the membrane. In the early stages when the membrane was still fluid, the bubbles escaped and the membrane rehealed itself. In about 30 min, the membrane had cured enough so that the air no longer escaped and started blowing blisters instead. In the instances where the concrete surface had been completely sealed by the prime coat, no blisters were formed.

Although many types of sealers were tried, few gave good results. Latexes alone were ineffective at preventing blisters, and all gave poor adhesion to the membrane.

Asphalt emulsions and cutbacks reduced the amount of blistering but gave very poor adhesion, especially when hot.

Various solutions of polymers in organic solvents failed to prevent blisters and usually gave poor adhesion to the membrane. A two-component epoxy resin gave excellent adhesion but did not prevent blisters.

The most effective sealers in laboratory tests were paints. These all contain pigments and fillers as well as a film-forming polymer. A number of white and black paints seemed to eliminate blister formation.

We then went to an outdoor test procedure using slabs of concrete 18 in. (46 cm) or 1 yd (92 cm) square and about 3 in. (7.6 cm) thick. In these outdoors tests, sealers were applied in the afternoon and the membrane on the following morning, which is the most severe time for blister formation. We found that this test was more severe than the laboratory test, and practically nothing was effective in eliminating blisters. Furthermore, we found that sealing almost all the pores (even 99 percent of them) was worse than not sealing the concrete at all because the remaining pores acted as blow holes to blow even larger blisters than normal. On an actual bridge, these blisters would still be so numerous that individual hand repair of each blister would be impractical. We were able to get a complete seal in the outdoor tests with a gray enamel by using four coats; but the cost of labor, time, and material for such a procedure would be prohibitive. It appeared that it was possible, but not practical, to avoid blister formation in the membrane by sealing the deck.

Use of Falling Temperatures to Prevent Blisters

We then turned our attention to applying the membrane in a falling temperature situation. Three methods where one can take advantage of a falling temperature to apply a membrane are: (*a*) preheating the deck, (*b*) applying the membrane late in the day or at night, and (*c*) using a black prime coat to help the sun heat the deck.

Preheating the Deck

A falling temperature situation can be created artificially by heating the deck immediately before application of the membrane. In this way, the deck is cooling off while the membrane is curing; and if the membrane is sufficiently cured before the deck temperature starts to rise again, blister formation is avoided. This procedure was used successfully on a number of bridge jobs in California, using propane-fired heaters mounted on a trailer or on the tailgate of a truck. The use of spray equipment is almost imperative in order to apply the membrane closely behind the heater. The membrane should be applied when the deck temperature is still significantly higher than the temperature of the adjacent unheated deck. Experience has shown that a temperature differential of 30°F (16.7°C) at the time of membrane application is usually sufficient to avoid blister formation on a dry deck during the morning hours. Later in the afternoon, a smaller differential is required to achieve the same effect. These temperature differentials give time for the membrane to cure enough to withstand the force of the expanding air before the temperature starts to rise again. About 1 ½ h are required before the membrane is strong enough to withstand the pressure developed without blowing blisters.

Figure 2 shows results obtained with the heat-and-spray technique on a California bridge during the morning hours. A control section purposely left unheated shows extensive blistering while the remainder of the membrane is smooth and blister free.

FIG. 2—*Comparison of membrane sprayed over heated and unheated deck. Time: 9 to 9:30 a.m.*

Frequently, concrete bridge decks are damp or wet at the time it is desired to apply a membrane. When these wet decks are heated with an infrared heater, the presence of water (because of its high latent heat of vaporization) causes the temperature differential after heating to be smaller than on a dry deck. In addition, the heater converts liquid water to water vapor, which intensifies and prolongs the blowing action and gives rise to long strings of bubbles in the membrane as described previously (Fig. 1).

On a dry deck, the expansion produced by the heater stops as soon as the heating is terminated. Consequently, when the membrane is applied (usually 2 to 5 min after heating), the outgassing has been completed. On a wet deck, however, the situation is more complex. In this case, the surface which heats up instantaneously may be relatively dry. After the heater passes and the membrane is applied, the wetter locations several inches below the surface may still be rising in temperature and thus vaporizing water. Consequently, the heater actually causes bubbles such as those shown in Fig. 1.

The only way that we found to avoid these strings of bubbles with moist concrete was to predry the deck with the infrared heater, then use the heat-and-spray technique to apply the membrane. The persistent and frequent occurrence of these strings of bubbles on wet decks caused us to forego the heat-and-spray technique as the preferred method of application.

Applying Membrane Late in the Day or at Night

In most areas, the temperature usually drops in the afternoon and continues declining until the following morning. This provides plenty of opportunity for applying membrane in a falling temperature situation but has several disadvantages. First of all, a black membrane applied to a white concrete deck in the sun will raise the deck temperature by as much as 30°F (16.7°C) and cause outgassing. In order to avoid this problem, it is necessary to wait until the sun is at a very low angle so that changing the color of the deck from white to black will not cause a significant increase in deck temperature. This eliminates most of the working time in daylight hours.

A blister-free membrane can be applied at night in a falling temperature, but nightwork requires lighting equipment and special traffic control and can be dangerous on heavily traveled routes. Low nighttime temperatures can slow down the rate of cure of membrane, leaving it open to blister formation the following morning. If the temperature drops below the dewpoint, work must stop to avoid reaction of the urethane ingredients with moisture.

Applying a Black Prime Coat

In order to overcome the problems just described, we decided to apply a black prime coat to enable the sun to preheat the deck, and thereby eliminate the sudden rise in temperature caused by the black membrane. Since the black prime coat does not completely seal the surface, the outgassing it causes will continue as long as the average temperature of the concrete is rising and will cease when the average temperature stabilizes or begins to fall. The amount of radiant energy being absorbed by a black primed deck in the sunshine depends primarily on the angle of the sun. Maximum radiation occurs when the sun is at its zenith, but the deck continues to increase in temperature until the rate of heat loss exceeds the rate of heat absorption. Therefore, the maximum average temperature of the deck will occur sometime after the sun has reached its zenith.

It is important to note that temperature of the deck surface alone does not always indicate the average temperature throughout the deck. As we will show later, in the afternoon, the surface temperature of the deck starts to drop before the interior temperature cools off; and the time when gas expansion stops is dependent on the temperature change of an average cross section, not just the surface temperature.

Figure 3 shows typical temperature measurements on a 3-in. (7.6-cm)-thick portland cement concrete slab outdoors on a sunny day. A black prime was applied to the concrete deck at 9:00 a.m. (daylight savings time) and allowed to warm in the sun until 1:00 p.m. when a 100-mil (2.5-mm) thickness of membrane was applied. Temperatures of ambient air, the bare concrete surface, the primed concrete surface, and the membrane surface were measured each hour throughout the day. Maximum temperature on the black primed surface was reached at 2:00 p.m., and, even though the membrane was applied an hour earlier, it was free of blisters. Membrane applied at the same time over an adjacent section without the black prime developed numerous blisters.

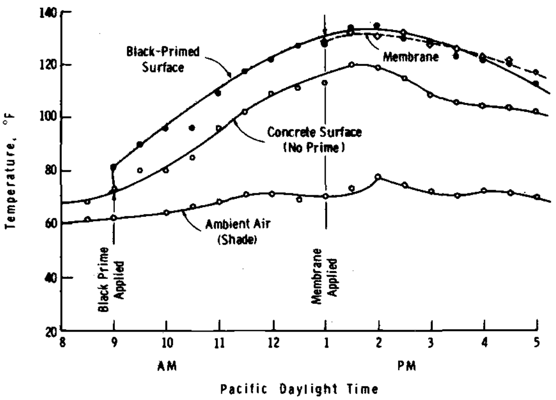

FIG. 3—*Temperature measurements on a black-primed and a membrane-coated concrete surface during a sunny day.*

On bridges near Portland, Oregon, Weed, California, and Salt Lake City, Utah, the temperature maximum on a black deck occurred a bit later, between 2:30 and 3:30 p.m. (daylight savings time). In the small-scale experiments, the concrete slabs were lying on the ground, which acted as a heat sink. On the actual bridges, the concrete slabs were not in direct contact with the ground so that heat was not drained away as fast as it was being absorbed from the sun. Therefore, the temperature continued to rise after the sun had reached its zenith. Other factors such as ambient air temperature, wind velocity, and amount of moisture in the deck can affect the rate of heat buildup, the maximum temperature achieved, and the time the maximum is reached.

Figure 4 shows the rate of heat buildup on a black-primed slab as a function of depth. On this particular day, the morning was overcast, and the sun came out at 11:00 a.m. The intial heat buildup was delayed, and the maximum temperature was lower than on a sunny day during the same week. Temperatures 0.5, 1.0, and 2.0 in (1.27, 2.54, and 5.08 cm) below the concrete surface continued to rise even after the surface temperature started to fall. In such a situation, membrane applied before the average temperature of the concrete starts to fall could form blisters.

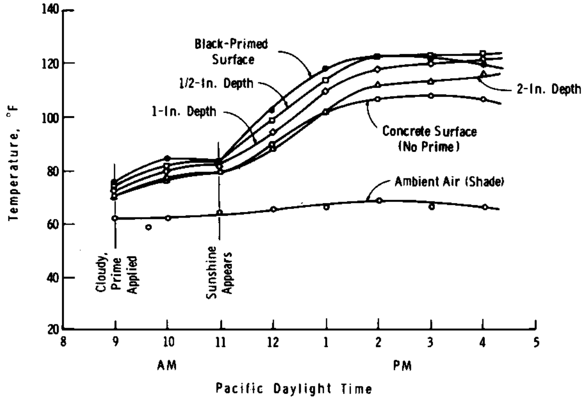

FIG. 4—*Heat buildup in black-primed concrete as a function of depth.*

The effectiveness of a black prime has been verified by more than 100 experiments in the laboratory and outdoors. Even with very wet concrete, no "strings of pearls" which characteristically appear in membrane applied over slightly damp concrete were formed.

Figure 5 shows membrane applied over a black-primed bridge deck in Salt Lake City. The foreground shows the black-primed surface before application of the membrane. This 2000 yd^2 (1672 m^2) bridge was coated with membrane in 5 h in late-afternoon sunlight.

Characteristics of the Black Prime Coat

The prime coat should be black, have a viscosity suitable for spray or

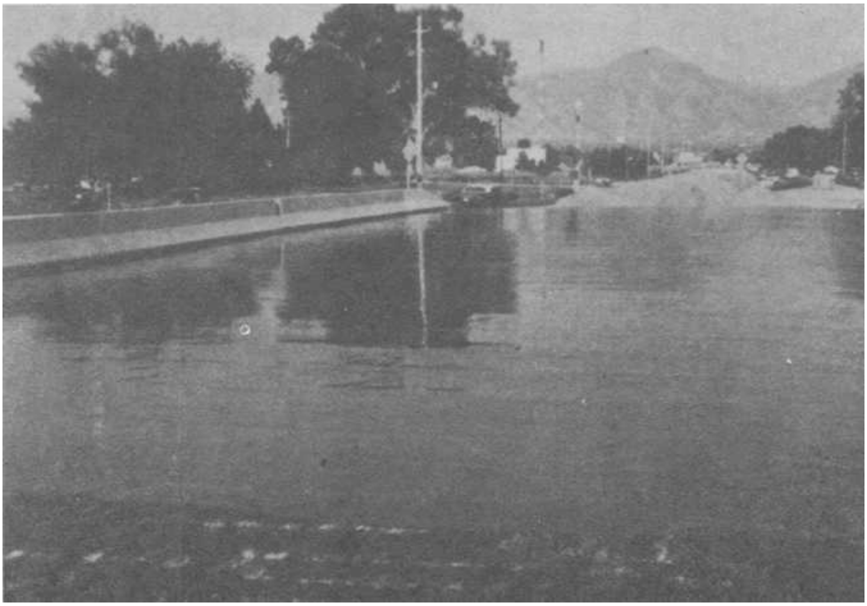

FIG. 5—*Membrane squeegee applied over black prime coat. Time: 4 p.m.*

roller application, and should dry thoroughly within 2 h. It must adhere strongly to concrete and provide good adhesion under extended wet and dry conditions. It is not necessary or even desirable that the primer seal the concrete. In fact, a thin application with no attempt to cover 100 percent of the concrete will suffice.

A test procedure was developed to measure the ability of the membrane to maintain its adhesion to concrete on water immersion. In these tests, our membrane was applied in afternoon hours to a large concrete slab outdoors with and without a black prime and allowed to cure for 48 h. The membrane and the upper part of the slab were then slit into 1-in.-wide strips. After initial peel strength tests were run on the dry block, a dam was built around the membrane and filled with water. The water penetrated the slits in the membrane and saturated the concrete. Additional peel tests were then run after one, two, and three weeks' continuous immersion in water. Based on these adhesion tests, we developed a black prime coat, which gave a high dry peel strength of 15 lb/in. (2.67 kg/cm) and which maintained this high peel strength even after three weeks' soaking in water.

Advantages of the Black Prime Coat

The use of a black prime coat increases the number of daylight hours which can be used to install a blister-free membrane. The black prime can be applied in the late morning and the membrane application started by 2 to 4 p.m. (as soon as the temperature of the black-primed deck is decreasing).

Damp bridge decks do not cause bubble formation such as the strings of bubbles formed when a heater is used on dry concrete.

Our black prime coat also aids wetting and spreading of the membrane. The prime coat itself is hydrophobic in nature, and is, in turn, easily wetted by the membrane. Air trapped at the surface on the concrete deck thus is displaced by the membrane.

With the black prime technique, timing of the membrane application is not as critical as it is with an infrared heater, where the membrane must be applied within 1 to 5 min behind the heater in order to maintain the required temperature differential. This can only be done with continuous spray equipment, which is costly and complicated. The technique of using a black prime coat, allowing the required 2-h heat-up period, and applying the membrane in a falling temperature allows greater flexibility in timing. Consequently, batch mixing and squeegeeing can be used as a simple, rapid, and low-cost procedure for applying the membrane. We have adopted this procedure as the preferred one for use on bridge decks.

Summary

A poured-in-placed waterproofing membrane and a technique for applying it to concrete decks have been developed. This system gives a defect-free, tough membrane. It essentially eliminates the formation of blisters, a major problem in membrane construction. Our bridge deck membrane system consists of:

1. A black prime coat to convert the color of the deck from white to black, not to seal the deck. The prime coat adheres strongly to the concrete and to the membrane applied over it.
2. A warmup period after application of the prime coat, followed by application of membrane in a falling temperature situation.
3. A two-component asphalt-extended urethane membrane which adheres strongly to the primed deck and satisfies the requirements demanded of a waterproofing membrane for bridges.
4. A tack coat of AC-20 (AR 4000 or 85/100 penetration) grade asphalt applied evenly over the membrane from a distributor truck at the rate of 0.1 to 0.15 gal/yd^2 (0.45 to 0.68 litre/m^2).
5. A minimum of 2 in. (5.1 cm) of asphalt concrete overlay to provide a riding surface and to protect the membrane from wear.

This cold-poured bridge deck membrane system was developed in the laboratory and field tested throughout the country. It has been used successfully during various stages of development on bridges in California, Oregon, Utah, New Mexico, Vermont, Ohio, Tennessee, and Virginia.

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References

[*1*] Stewart, C. F., "Deterioration in Deicing Salted Bridge Decks," paper presented at the Highway Research Board Western Summer Meeting, Aug. 1970.

[*2*] Boulware, R. L. and Elliott, A. L., *Civil Engineering*, American Society of Civil Engineers, Oct. 1971, pp. 42-44.

[*3*] Dallaire, Gene, *Civil Engineering*, American Society of Civil Engineers, Aug. 1973, pp. 43-48, and Oct. 1973, pp. 81-86.

[*4*] Van Til, C. J., Carr, B. J., and Vallerga, B. A., "Waterproof Membranes for Protection of Concrete Bridge Decks," NCHRP Project 12-11 Final Report, National Cooperative Highway Research Program, May 1976.

[*5*] Hay, R. E., *Public Roads*, Vol. 39, No. 4, March 1976, pp. 142-147.

[*6*] Boulware, R. L. and Stewart, C. F., "Field Electrical Measurements for Bridge Deck Membrane Permeability and Reinforcing Steel Corrosion," technical report, California Division of Highways, June 1973.

[*7*] Brakey, B. A. and Whalin, W. V., "Bridge Deck Membranes Evaluation and Use in Colorado," paper presented at the Annual Meeting of American Association of State Highway Officials, Los Angeles, Nov. 1973.

[*8*] Beijers, G. M. H., "Research to the Formation of Blisters Which Occur by Application of Waterproofing Layers on Concrete Bridges," paper prepared for presentation at the Annual Meeting of the Association of Asphalt Paving Technologists, New Orleans, 16-18 Feb. 1976.

Summary

The papers by Cady, Mehta, and H. K. Cook and McCoy deal with a definition of the nature and scope of the problem of chloride-induced corrosion of steel in concrete, the effects of cement composition, and the influence of chlorides in reinforced concrete. There is an important call by Professor Cady for a major, centralized effort to collect, evaluate, and synthesize the work which has been carried out on methods and techniques to control this corrosion problem. Peterson's paper presents the problem as seen through the eyes of a highway bridge engineer from a major northeastern state. His remarks and observations bring the problem down to many of the basic issues faced by a public official pursuing his responsibilities to control bridge deck corrosion.

The protective barrier technique papers present a state-of-the-art discussion of various bar coating methods, bar corrosion inhibitors, and barrier membranes. The Baker, Money, and Sanborn; A. R. Cook and Radtke; and Kilareski papers on nickel, galvanized, and epoxy bar coatings suggest that each of these offer a substantial improvement over the performance obtained from unprotected bars. However, much more reliable measures of performance enhancement are needed for full-scale, dynamically loaded structures.

It appears that the performance criteria adopted by Baker, Money, and Sanborn in the design and evaluation of their test results deserve careful consideration and more widespread adoption by others contemplating corrosion performance experiments on nonferrous coatings. The authors are to be commended for the particular attention which was paid to isolating the differences between the degree of cracking of specimens and failure of specimens due to spalling. These data provide valuable support to the argument that performance criteria based on cracking or time to crack must be handled with great care.

The Rosenberg, Gaidis, Kossivas, and Previte paper on the use of a calcium nitrite inhibitor to protect the reinforcing steel is a novel approach to the problem. Whether this should be classified as a matrix modification technique or as a barrier coating is debatable. While the data presented in the paper look promising, it should be borne in mind that they were collected under accelerated test conditions. A final judgement on the utility of this approach must be deferred pending a longer-term study and evaluation under conditions more representative of field service.

The Meader, Schmitz, and Henry paper, along with the Frascoia paper, discusses many of the important considerations in using sheet and cold-poured membrane systems. Although membranes are one of the oldest systems to be employed in bridge rebar protection, their effectiveness has been actively debated in highway bridge circles. Meader et al and Frascoia present information which shows that membranes continue to hold considerable promise when they are selectively chosen, intelligently specified, and carefully applied.

Kukacka's paper on use of concrete polymer materials, Steele and Judy's paper on polymer modified concrete, and O'Connor's paper on the "Iowa Method" for partial-depth resurfacing show that so-called matrix modification methods are viable techniques deserving serious consideration by the specifying engineer. As is true with most other materials discussed in this volume, these must be carefully applied by skilled and knowledgeable workers.

Impressed current cathodic protection represents one of the most common and effective techniques for corrosion control in general use today. Adapting this technology to the bridge deck environment has not been a simple undertaking. Considerable innovation has been necessary to produce a system which is suited for reinforced concrete. Vrable's paper is an excellent reference for any worker interested in the use of this technique. Ward's paper provides a valuable review of the practical considerations which must be studied before undertaking field applications of impressed current protection systems for bridge decks.

It is the hope of both the organizers and participants in the symposium that this publication will find good use as a basic reference document of protection systems for countering the corrosive effects of chlorides in reinforced concrete structures.

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