

TRANSPORTATION RESEARCH RECORD 762

**Corrosion,
Cathodic Protection,
Aggregate Upgrading,
Concrete Density, and
Pavement Markings**

TRANSPORTATION RESEARCH BOARD

*COMMISSION ON SOCIOTECHNICAL SYSTEMS
NATIONAL RESEARCH COUNCIL*

*NATIONAL ACADEMY OF SCIENCES
WASHINGTON, D.C. 1980*

Transportation Research Record 762

Price \$5.80

Edited for TRB by Mary McLaughlin

mode

1 highway transportation

subject areas

32 cement and concrete

33 construction

34 general materials

35 mineral aggregates

Library of Congress Cataloging in Publication Data

National Research Council. Transportation Research Board.

Corrosion, cathodic protection, aggregate upgrading, concrete density, and pavement markings.

(Transportation research record; 762)

Reports for the 59th annual meeting of the Transportation Research Board.

1. Bridges—Floors—Corrosion—Addresses, essays, lectures.

2. Protective coatings—Addresses, essays, lectures. 3. Aggregates (Building materials)—Addresses, essays, lectures. 4. Road markings—Addresses, essays, lectures. I. Title. II. Series.

TE7.H5 no. 762 [TG325.6] 380.5s

81-405

ISBN 0-309-03104-4

[625.7'61]

AACR1

ISSN 0361-1981

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December 31, 1979.

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Decision Criteria for the Rehabilitation of Concrete Bridge Decks

D. G. MANNING AND J. RYELL

A systematic approach to bridge-deck rehabilitation is presented. Bridge-deck rehabilitation is consuming an increasing proportion of the resources of highway agencies. The nature and extent of deterioration are highly variable so that there is neither a single problem nor a single solution. The requirements for a condition survey are described. The performance of concrete overlays, waterproofing membranes, and cathodic protection applied to existing structures is assessed from field studies and the literature. Decision criteria that can be used to identify the most appropriate method of rehabilitation for any particular structure are given.

An increasing emphasis is being placed on the maintenance and rehabilitation of bridges. Now that fewer changes are being made in the highway network, many structures that were once replaced as part of improvements in highway alignment must be kept in service. Deterioration is especially serious in bridge decks in areas of North America where deicing chemicals are used in winter maintenance operations. The salt penetrates the concrete and initiates corrosion of the steel reinforcement. This in turn causes cracking and rupturing of the concrete and spalling of the concrete surface. In addition, some older bridge decks, built to lower standards of quality than modern structures and constructed without the benefit of efficient waterproofing systems and air entrainment, suffer a more deep-seated distress as the concrete progressively disintegrates from the surface downward under freeze-thaw action.

A major consideration that complicates the development of a policy for bridge-deck rehabilitation is the variability in the condition of the structural concrete, the reinforcing steel, and, more recently, the prestressing systems in structures under the jurisdiction of a single authority. Such variability is inevitable because the condition of the deck slab is affected by many factors, the most important of which are age, the standards in use at the time of construction, the quality of the materials and workmanship, the type of design, and the service environment. Since it is clear that no single problem of bridge-deck durability exists, it is also apparent that there is no single solution for the rehabilitation of all concrete deck slabs. What is needed is a systematic approach that includes the identification of the deterioration on a particular structure and the development of decision criteria that will lead to the selection of the most cost-effective solution for that structure.

CONDITION SURVEYS

A condition survey of a bridge deck may be required for either of two reasons: (a) to establish repair priorities on a statewide scale or (b) to provide details of the nature and extent of concrete deterioration needed for design and execution of the restoration work. The work required to determine repair priorities can be described as a general survey, and the more comprehensive inspection prior to the design of the restoration work is called a detailed condition survey.

The objective of the general survey is to obtain an overall rating of the deck slab so that, where necessary, it can be programmed for future restoration. Deck slabs that have an exposed concrete surface can be rated relatively straightforwardly, and measurements of concrete cover, half-cell potentials

of the reinforcing bars, areas of temporary patches and delaminations, and the presence of significant cracks and scaled areas on a sample area of the concrete surface provide adequate information in most cases.

The presence of an asphalt wearing surface on a bridge deck, which is the case with the majority of structures in Ontario, makes it difficult to determine the overall condition of the concrete slab. Some asphalt-covered deck slabs date back to the 1920s, and their condition is highly variable (1). When deterioration is more advanced, it is sometimes obvious from a visual examination of the deck, and the rehabilitation work can be programmed without a general survey. In most cases, however, it will be necessary to take core samples and remove small sections of the asphalt overlay to determine the condition of the concrete.

Some general criteria can also be established that will assist in determining whether deterioration is likely and whether a general survey of the structure is required. The age of the structure is a key indicator of its condition. Cracking and repairs to the asphalt or leakage, wet spots, or cracks on the underside of the deck slab are useful clues to the condition of the concrete in the deck. Severe deterioration of exposed areas, such as sidewalks, curbs, and handrail posts, may also indicate deterioration in the deck slab.

Detailed condition surveys are carried out when the structure has been programmed for rehabilitation and it is necessary to select the method of repair and prepare the contract documents. Although guidelines and the general scope of the survey can be prepared in advance, the details of the work must be determined by the engineer on site. Existing information on the condition of the deck slab is considered, and additional information is obtained as the work proceeds. For example, if it is clear that the deck must be replaced, as in the case of an older deck that exhibits widespread cracking as the result of alkali-aggregate reactivity, then little or no testing is required. If it is not clear whether the deck slab can be effectively rehabilitated or must be replaced, a very detailed survey may be needed.

In the detailed survey, as in the general survey, asphalt-covered decks are more difficult to evaluate than decks with an exposed concrete surface, and consequently the cost of the detailed survey will be greater and the reliability of the data lower.

In most cases, the detailed condition survey will include the following:

1. If there is a bituminous overlay, its condition and its thickness are determined and significant cracks in the wearing surface are recorded.
2. If there is a waterproofing membrane, its condition is appraised and it is identified by type.
3. Patched and open spalled areas, delaminated concrete, scaled areas, and significant cracks on exposed concrete deck surfaces are recorded and measured.
4. Concrete cover to the top layer of reinforcing steel is surveyed. On asphalt-covered decks, cover can only be measured where sections of

the asphalt overlay are removed.

5. The corrosion activity of the top layer of reinforcing steel is determined by measuring half-cell potentials. Holes are drilled through the bituminous overlay to the deck surface to ensure a good electrical contact.

6. The general condition of the concrete slab is determined by coring and removing sections of asphalt overlay.

7. Tests are performed on concrete core samples to determine chloride content, air void system, and, occasionally, compressive strength.

8. The underside of the deck slab is inspected, and deteriorated concrete, wet areas, efflorescence, significant cracks, corrosion spalling, and other defects are identified.

9. Deck drains are inspected, and their condition, position, and adequacy are determined.

10. Curbs, sidewalks, barrier walls, handrails, and other components of the structure above the riding surface are inspected.

11. The condition, type, and measurement of expansion and fixed-joint assemblies and special features needed for future reconstruction are identified.

12. Other parts of the structure that should be repaired as part of the contract for rehabilitation of the deck are identified.

13. A comprehensive report is produced that documents in detail the condition of the deck slab and its components. The report contains plans, core logs, photographs, tables, and test data.

The test methods, the number of samples required, and the sequence of operations are discussed in detail elsewhere (2).

OPTIONS FOR REPAIR

Many methods of bridge-deck rehabilitation have been proposed and investigated. This paper discusses the application of the three methods of rehabilitation that are currently used in Ontario when major rehabilitation of either an exposed-concrete or asphalt-covered deck slab is required. These three methods are (a) patching, followed by waterproofing and paving; (b) application of a concrete overlay; and (c) cathodic protection. Temporary repairs such as local patching or epoxy injection are not discussed.

It is not our intention to describe in detail the construction procedures involved in each method, since these are well documented elsewhere (2-4). However, the essential processes of each method can be described as follows:

1. The patching, waterproofing, and paving method consists of saw-cutting around areas of delaminated and spalled concrete, removing all unsound concrete, patching [usually with portland cement concrete (PCC)], and applying a waterproofing membrane, a protective layer, and a bituminous wearing course. There are many waterproofing systems available, but those that have proved most satisfactory all require application of a protection board and bituminous concrete (5).

2. In the placing of a concrete overlay, the entire deck surface is scarified, all delaminated and unsound concrete is removed, and a bonding layer is applied. This is followed by an overlay, which may consist of high-quality conventional PCC or a latex-modified concrete.

3. The type of cathodic protection system used in Ontario (4) is installed by placing concrete

patches in exactly the same manner as if the deck were to be waterproofed, placing anodes in the deck surface, and applying an electrically conductive bituminous mixture and a conventional bituminous wearing course. Power in the form of a low-voltage direct current is supplied to the anodes from an alternating-current rectifier.

When the basic problem on a bridge deck is corrosion of the reinforcing bars and shallow surface spalling, one of the most difficult decisions to make is how much concrete must be removed as part of the repair contract. The chloride ion concentration in most bridge decks that are in need of repair exceeds the threshold value sufficiently to initiate corrosion of the reinforcing steel. The threshold value is commonly accepted to be 0.15 percent of the soluble chloride by mass of cement content of the concrete (6). Under these circumstances, unless cathodic protection is to be applied, the only way to ensure that corrosion does not continue is to remove all concrete that contains chlorides in excess of the threshold value and then prevent further applications of deicing salts from gaining access to the reinforcing steel (7,8). Most of the chloride-contaminated concrete is, however, usually physically sound, so that its removal is not only tedious but also very expensive, almost as expensive as replacing the deck. Deck replacement (except when it is required for structural reasons) or the removal of all chloride-contaminated concrete is beyond the financial capability of most highway authorities. Consequently, many authorities believe that it is more economical not to remove chloride-contaminated concrete that is otherwise sound and to accept the resulting uncertainty about the future life of the deck slab. This practice, which is referred to as "experimental cost-effective reconstruction", is common in both the United States (9) and Canada (3).

PERFORMANCE OF REHABILITATION METHODS

The effect of waterproofing or of a concrete overlay on the continuing corrosion activity in a bridge deck is not well documented, yet the ability to predict the future life of the deck slab is an essential ingredient in the technical and financial analysis that must be undertaken to select the most appropriate repair method. Data on which to base investment decisions are lacking, not only because major deck rehabilitation is a relatively new phenomenon but also because many methods have only been in use for a few years and satisfactory tools for measuring the performance of rehabilitation techniques have generally not been available. The use of the half-cell method of measuring the corrosion activity of the reinforcing steel in a concrete bridge deck was first reported in 1973 (10) and formalized as a standard test method in 1977 (11). Only recently have highway agencies initiated systematic surveys to document the condition of decks before repair and to monitor their performance after repair. Few results have yet been published because of the number of years required to establish meaningful trends.

Concrete Overlays

The first low-slump concrete overlay in Ontario was placed in May 1976 and has been monitored regularly. The results of the annual half-cell surveys are shown in Figure 1. The results are presented in the form of cumulative frequency distribution curves, a convenient method of

Figure 1. Corrosion potentials at Interchange 4, Ontario 401.

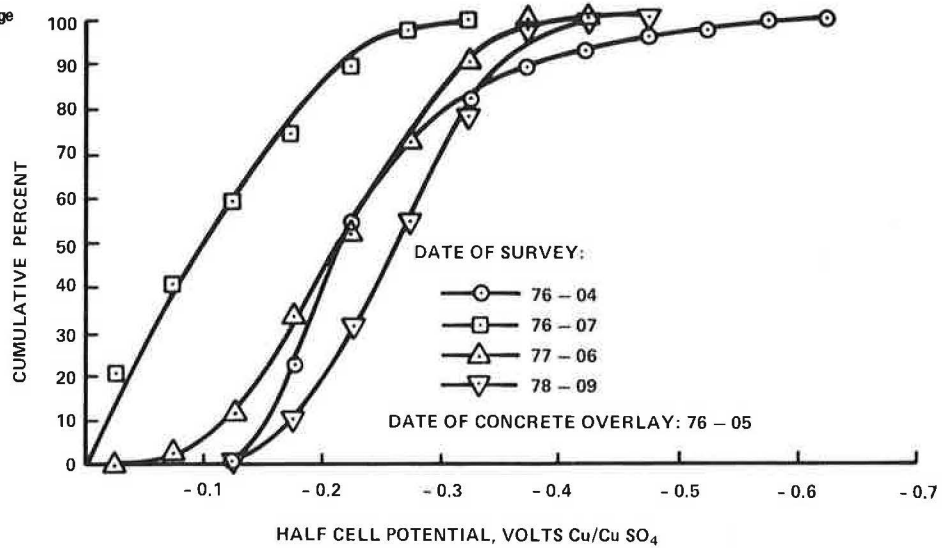
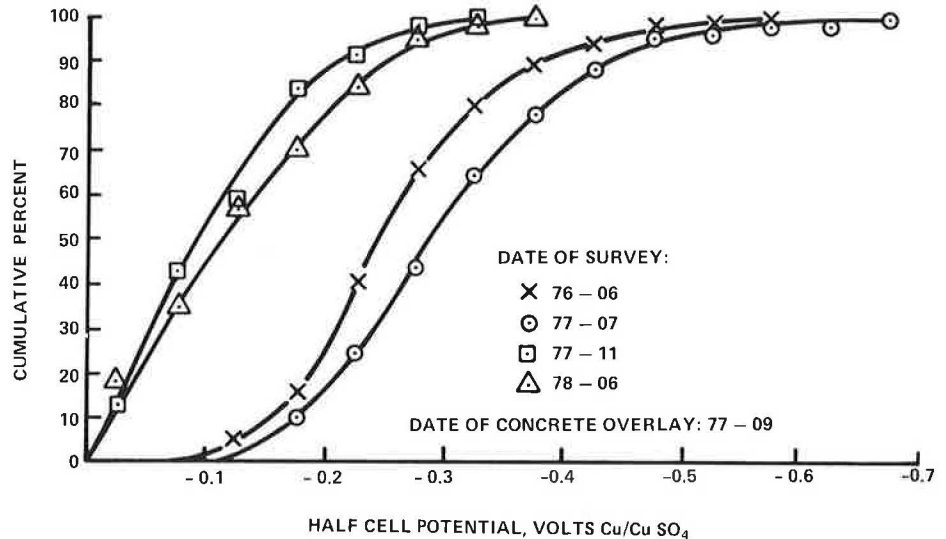


Figure 2. Corrosion potentials at Englehart River Bridge, Ontario 66.



observing changes in corrosion activity between surveys. A shift of the curve to the left indicates a reduction in corrosion activity, and a shift to the right indicates the opposite.

The significant reduction in corrosion activity immediately after application of a concrete overlay has been reported elsewhere (3). This occurs because concrete is removed from around the bars that exhibit the greatest corrosion activity. Delaminations are usually associated with a half-cell potential of more (negative) than -0.50 V and, in the course of removing delaminated concrete, it is often necessary to remove the concrete from around the exposed bars. When the concrete overlay is placed, these exposed bars are covered by the mortar bonding grout and are surrounded by chloride-free concrete so that corrosion activity is suppressed in these areas. Later measurements have shown that the initial reduction in corrosion activity has not been sustained and the overall level of activity has gradually increased.

Although Figure 1 shows the curve for the 1978 survey to be to the right of the curve representing the original deck survey, the area of the deck that exhibits active corrosion (more negative than -0.35

V) is less than it was before the overlay was applied, and the highest potential recorded is much less.

The results of half-cell surveys on a badly deteriorated deck to which a low-slump concrete overlay was applied in 1977 are shown in Figure 2. Two surveys made prior to overlay are shown, and the measurements demonstrate the significant increase in corrosion activity that occurred between 1976 and 1977. Figure 2, like Figure 1, shows a sharp reduction in corrosion activity after construction of the overlay, but in Figure 2 the increase in corrosion in the second year is only slight. Because the deck represented in Figure 2 was considerably more deteriorated than that represented in Figure 1, a greater portion of the reinforcing steel was surrounded by new concrete, and this may account for the slower rate of increase in corrosion activity.

Other decks that have been given either a low-slump or a latex-modified concrete overlay have been monitored and found to exhibit trends similar to those shown in Figures 1 and 2, which can be considered to represent the range of the conditions recorded. Other authorities have reported a similar

initial reduction in corrosion activity beneath low-slump concrete overlays (12,13) and latex-modified concrete overlays (14,15), but this has not always been the case. On some structures, corrosion activity has continued (16) or increased (12) after placement of a latex-modified concrete overlay. The wider variation in the results for latex-modified concrete overlays probably results from the fact that the deck must be wet down before the latex-modified bonding grout is applied. Rapid rust formation takes place when exposed reinforcing bars, which have been sandblasted, are wet down, and this may mask the overall change in the corrosion activity of the reinforcing steel.

Although half-cell potentials are a valid measure of the presence of corrosion activity in a bridge deck, they do not indicate the rate of corrosion, which is determined primarily by the availability of oxygen and moisture at the cathode and the structure and formation of the corrosion products at the anode. It is possible to have a high reading for corrosion potential but a very low rate of corrosion so that physical distress may not occur for many years. In the same way, rapid corrosion rates may be associated with lower, but active, potential readings. Corrosion potentials are thus a useful indication of performance, but the deciding factor will always be actual service life in the field.

Concrete overlays have been used for as long as 15 years in many jurisdictions, including British Columbia (17), and by several state highway departments. A report published by the Iowa State Highway Commission in 1974 (18) indicated good performance of such overlays, which were then from one to nine years old, despite relatively high chloride levels in the concrete in the original deck slab. A more recent study (19) reports the investigation of the condition of the same bridge decks five years after the original investigation. The latest study included a delamination survey and the measurement of corrosion potentials, which were not part of the original survey. The results show that the majority of the decks exhibited delaminations and that most of these were just below the bond line between the overlay and the original deck concrete. Despite the presence of the delaminations, no surface distress was observed and the performance of overlays on chloride-contaminated decks was considered adequate. It should be noted that the overlays were constructed to specifications that required thinner overlays and the removal of less concrete than existing specifications.

A survey of 149 latex-modified concrete overlays in Ohio, West Virginia, Michigan, and Kentucky (14), some of which were more than 15 years old, found that, despite local debonding and cracking, performance was generally satisfactory. Consequently, the trends in the corrosion potentials shown in Figure 1, and the performance of overlays in Iowa and elsewhere, give a better indication of the economic life of a concrete overlay applied to a chloride-contaminated deck than has previously been possible and suggest that 15-20 years is probably a realistic period.

Patching, Waterproofing, and Paving

Data on the effect of repairing a chloride-contaminated deck by patching, waterproofing, and paving are even less available than data on concrete overlays. In Ontario, systematic before-and-after surveys have not been made, but in 1978 a study was undertaken to determine the effect of waterproofing on the corrosion activity in a deck slab. Where half-cell readings had been taken on deck slabs that had subsequently been waterproofed, potentials were

measured at the same grid points by drilling through the waterproofing to ensure an electrical contact with the deck surface. For most structures, only part of a deck was included in the original survey, in which case only the same part of the structure was included in the 1978 survey.

The most reliable data were available from two structures that were built in 1959 and 1960 with a bituminous concrete surface but without a waterproofing layer. The concrete decks were exposed in 1975 and resurfaced with a hot-applied rubberized-asphalt waterproofing membrane, a protection board, and 75 mm (3 in) of hot mix as part of the same resurfacing contract. Half-cell measurements, cover, and areas of delamination were recorded on both decks when they were exposed in 1975. The half-cell data, together with those from the 1978 survey, are shown for each structure in Figures 3 and 4.

When the Nith River bridge (Figure 3) was examined in 1975, extensive areas of delamination were recorded. These were associated with areas of inadequate cover to the reinforcing steel. The cover varied between 20 and 60 mm (0.75 and 2.375 in) and averaged 35 mm (1.38 in). Delaminated concrete was removed, and concrete patches were placed in the spalled and delaminated areas. The deck on the Canadian Pacific Railway (CPR) structure (Figure 4) was in good condition in 1975 and free from delamination. The cover varied between two isolated readings of 25 mm (1 in) and a high of 70 mm (2.75 in) and averaged 55 mm (2.2 in).

Of the seven cores taken in 1978 from the Nith River bridge, five were taken at grid points included in the 1975 survey. The half-cell measurements at the time of the two surveys (Cu/CuSO₄ half-cell) and the condition of the cores are given below (NR = not recorded):

Core No.	Half-Cell Potential (V)		Core Delaminated
	1975	1978	
1	-0.49	-0.50	No
2	NR	-0.56	Yes
3	-0.29	-0.30	No
4	-0.37	-0.54	Yes
5	-0.23	-0.23	No
6	-0.42	-0.55	Yes
7	NR	-0.54	No

Of the five cores taken where the potential was -0.50 V or greater, three were found to be delaminated. Although it is possible that not all delaminated areas were removed at the time of repair in 1975, this is unlikely, and the half-cell measurements taken at the locations of cores 4 and 6 in 1975 suggest that the delaminations were not present when the waterproofing was applied. Figure 3 also shows that there has been an increase in overall corrosion activity in the deck between waterproofing and examination three years later.

The opposite effect has occurred on the CPR structure, in which there has been a reduction in corrosion activity since the deck was waterproofed.

An attempt was made to compare the effectiveness of the waterproof membrane on the two decks by using resistivity measurements (20). The resistivity on both decks was low, but this is thought to be the result of moisture in the bituminous concrete shorting the electrical circuit to the reinforcing steel by way of the deck drains. In selected locations on both decks, the waterproofing was examined by removing the bituminous concrete. In all cases, it was found to be well bonded to both the deck and the protection board, of uniform thickness, and in good condition. The reason for

Figure 3. Corrosion potentials at Nith River Bridge, Ontario 401.

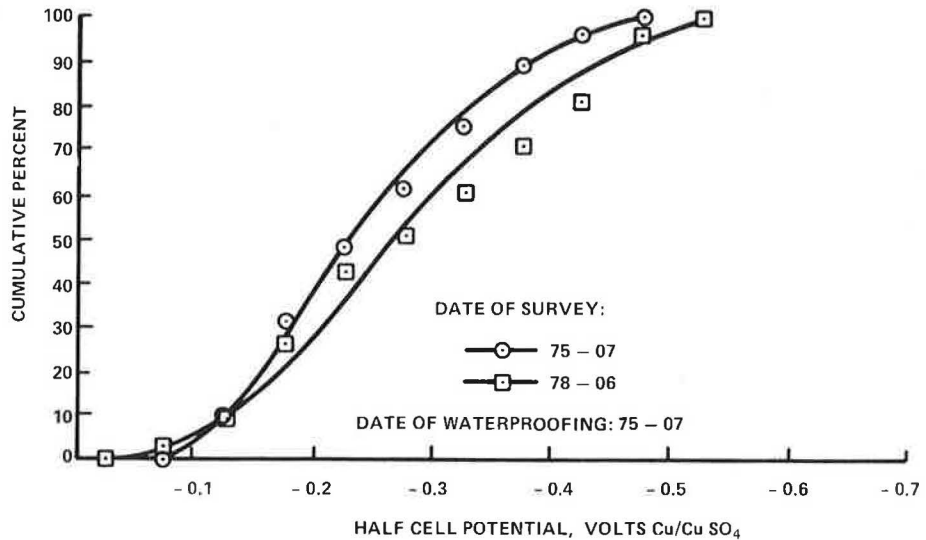
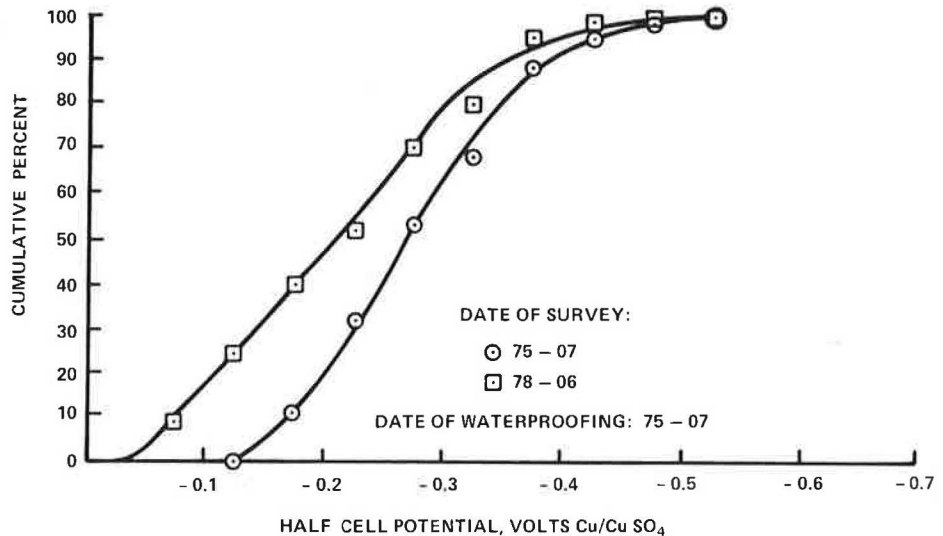


Figure 4. Corrosion potentials at CPR overhead, Ontario 401.



the difference in the performance of the waterproofing membranes is not clear.

Wide variations in the performance of waterproofing membranes applied to chloride-contaminated decks have been the common denominator in other bridge-deck surveys. A survey of 74 structures was undertaken in Ontario in 1974 to evaluate the effectiveness of waterproofing membranes (21), and measurements were made of both electrical resistivity and half-cell potentials. Thirty-seven of the structures tested, including some of the 17 structures that were new at the time of waterproofing, exhibited active corrosion potentials. There was no correlation between resistivity and corrosion measurements so that high resistivity readings, which indicate good membrane performance, were often associated with high potential measurements.

A similar lack of correlation between resistivity and corrosion was found in a survey of 44 decks in New York State (22). All of the decks were waterproofed after being in service for a number of years. Forty percent of the decks indicated active corrosion over more than 25 percent of the deck area. When half-cell measurements were made on the

same deck over a number of years, the number of readings in excess of -0.35 V fluctuated widely from year to year. With few exceptions, the percentage of readings that indicated active corrosion increased with time.

Work in California has shown that repairing delaminations before waterproofing will reduce the percentage of active corrosion potentials in a deck but that corrosion activity will continue at other locations in a chloride-contaminated deck (23). Other surveys in the United States have reported both a reduction in corrosion activity (12) and renewed spalling (24) beneath membranes applied to decks that were previously exposed to deicing salts.

In many ways, the results of the surveys on waterproofed decks are confusing, and the effect of waterproofing on the future service life of the deck slab is unpredictable. Spalls and delaminations have occurred within three years of waterproofing a chloride-contaminated deck. Given the variability in membrane performance, it would seem prudent to seek other methods of rehabilitating such decks until such time as the effect of the waterproofing is better understood.

Cathodic Protection

Cathodic protection differs from the application of a waterproofing membrane or a concrete overlay in that it actively prevents continued corrosion of the reinforcing steel. The low-voltage direct current that is applied over the deck surface by way of the conductive mixture polarizes the reinforcing steel so that corroding anodes on the steel are prevented from discharging ions and become current-receiving cathodes. The effectiveness of the cathodic protection in arresting the corrosion of the reinforcing steel has been demonstrated by embedding corrosometer probes in the deck slab (4).

The first installation of cathodic protection on a bridge deck in Ontario took place in 1974 (25). Only half the deck was protected because the method was considered experimental. The remaining half received a dense bituminous concrete overlay and served as the control for the experiment. All spalls on both sides of the deck were patched with concrete, and the delaminated areas were injected with epoxy.

The bituminous concrete was removed in 1977 so that the concrete deck slab could be examined and a cathodic protection system applied to the entire deck. Many delaminations were located on the unprotected side of the deck. Delaminations were also located on the protected side of the deck, although closer examination showed that these could be divided into two categories--those that had occurred in areas previously injected with epoxy and those that were adjacent to the centerline of the deck. The epoxy used to inject the delaminated areas in 1974 was a dielectric material that prevented the flow of current to the reinforcing steel beneath the injected areas so that further corrosion took place. The delaminations adjacent to the centerline were part of much larger delaminations on the unprotected north side, and the extension of the delaminations into the south side was thought to be the result of corrosion activity in the unprotected side (27).

The examination of the deck surface confirmed the effectiveness of cathodic protection in arresting corrosion in a deck slab but also resulted in a change in repair practices. These now require the removal of all delaminated areas and the placing of concrete patches prior to the application of cathodic protection. Newer installations have also included the recessing of the electrical hardware and the use of a conductive mixture with good stability characteristics (26).

FACTORS THAT AFFECT SELECTION OF A REPAIR METHOD

The selection of the most appropriate method of repair for any particular structure is determined not only by the technical considerations that have already been discussed but also by a number of other factors, some of which are economic and others purely practical. Although the purpose of this paper is to present a systematic approach to bridge-deck rehabilitation, repairs to the deck cannot be separated from an evaluation of the condition and load-carrying capacity of the remainder of the structure. If the structure as a whole is found to be functionally obsolete when current criteria for width, clearances, alignment, and load limits are applied, or if deficiencies are noted in other components of the structure that will limit its service life, then the rehabilitation strategy must be compatible with the life of the structure as a whole. Methods of conducting an economic analysis to determine the costs and benefits of replacing or repairing a structure and

repair priorities between structures have been published (28-30), but the usefulness of such an analysis is limited by the accuracy with which the cost and the life expectancy of the various rehabilitation schemes can be predicted.

The factors that affect the selection of the repair method and the priority of repair can be summarized as follows:

1. The location of the structure and its importance in the highway network;
2. The volume of traffic at the site and the impact of lane closures on traffic flow;
3. The type, size, and geometry of the structure;
4. The nature of the deterioration;
5. The extent of the deterioration;
6. The anticipated service life of the structure;
7. The load-carrying capacity of the structure;
8. The cost of repairs and the availability of funds;
9. The future reconstruction program; and
10. Local experience and contractor expertise.

The importance of a structure is determined by traffic volumes at the site and the availability of alternate routes. Consequently, some freeway structures warrant a greater priority for repairs and a higher standard of maintenance than could be justified for other structures in the highway network. Traffic volumes also affect the choice of the method of repair in that they determine the number of lanes that can be closed at any one time and in some cases may dictate the selection of a rehabilitation scheme that is expedient rather than the one that is the most technically desirable. Greater priority must also be given to the repair of structures in which the deck is part of the main structural member, as in the case of thick-slab structures and some box-and-tee-girder bridges. The size of the structure affects the economics of the various rehabilitation strategies, and unusual deck geometry may eliminate some repair methods. For example, structures with changing superelevation, large skews, or sharp tapers may exclude the use of finishing machines that have transverse oscillating screeds such as those used in placing low-slump concrete overlays. Latex-modified concrete overlays are difficult to place on steep grades and crossfalls, and waterproofing membranes should not be used on grades in excess of 4 percent or in areas that are subject to rapid vehicle acceleration, braking, or turning movements. Cathodic protection cannot be used unless electric power can be supplied to the site, although experimental solar-powered installations are being developed.

The nature and extent of the deterioration have a very significant effect on the selection of the repair method. On severely deteriorated decks, the patching required prior to the installation of a waterproofing membrane or cathodic protection becomes a major item in the repair contract and, when the deterioration has resulted from inadequate cover, the cover is still inadequate after patching. Conversely, concrete overlays are well suited for use on badly deteriorated decks because the areas of concrete removal do not require perimeter saw-cutting and the concrete is replaced in the course of applying the concrete overlay. Furthermore, the concrete overlay acts as a structural component of the deck and, where the load-carrying capacity of the structure is a factor, the additional load from the bituminous overlays used with waterproofing and cathodic protection may be unacceptable. Active cracks in the deck slab

Table 1. Relative merits of rehabilitation methods.

Rehabilitation Method	Advantages	Disadvantages
Concrete overlay (low-slump or latex-modified concrete)	Structural component of deck slab; relatively impermeable; relatively long service life; well suited to repair of badly spalled or scaled decks; many qualified contractors	Not suited to decks with complex geometry; cannot bridge moving cracks; difficult to provide adequate texture on low-slump concrete surface; may not stop active corrosion
Waterproofing membrane with bituminous concrete wearing course	Bridge cracks with small amounts of movement; relatively impermeable; provides good riding surface; applicable to any deck geometry; many qualified contractors	Performance highly variable; will not stop active corrosion; service life limited by wearing course; nonstructural component of deck slab; not suitable for grades greater than 4 percent
Cathodic protection	Stops active corrosion; can be used on decks with moving cracks; provides good riding surface; applicable to any deck geometry	Presence of wearing course will accelerate deterioration of marginal quality concrete; nonstructural component of the deck slab; periodic monitoring of performance required; wearing course requires periodic replacement; specialized contractor and inspection required; electrical power source required

Table 2. Decision matrix indicating which repair methods are excluded for which repair criteria.

Criterion	Concrete Overlay	Waterproofing Membrane and Paving	Cathodic Protection
Delamination and spalls that exceed 5 percent of the deck area		No	No
Corrosion potentials greater than -0.35 V over more than 20 percent of the deck area		No	
Active cracks in the deck slab	No		
Remaining life of structure < 10 years	No		No
Concrete not properly air entrained			No
Complex deck geometry; skew exceeding 50°, curvature exceeding 10°, or changing superelevation	No ^a		
Limited load capacity of structure or span-to-thickness ratio of deck slab > 15		No	No
Electrical power unavailable			No
Epoxy injection repairs previously performed and will not be removed			No

^aRestriction applies only to finishing machines whose axis of screed is transverse to the axis of the roadway.

generally preclude the use of concrete overlays because they are susceptible to reflection cracking, which may limit their service life. The quality of the concrete in the deck slab must also be evaluated and, where the use of cathodic protection is contemplated, the air-void system needs to be measured by using the linear traverse or point-count method to ensure that the concrete will remain durable after application of the cathodic protection system. The bituminous overlay is permeable, increases the severity of the service environment of the deck slab by increasing the degree of saturation of the concrete, and, where the deck surface is previously exposed concrete, increases the number of freeze-thaw cycles. The relative advantages and disadvantages of the various rehabilitation schemes are summarized in Table 1 (2).

The future reconstruction program also affects the time at which repairs are undertaken. It may be more economical to include a structure that does not warrant immediate rehabilitation but is located in close proximity to other structures to be repaired than to award a separate contract at a later date, especially if no other work is planned in that area for several years. Local experience is also an important factor in selecting the repair method. For example, some authorities have reported good success with waterproofing membranes (31), and others have replaced membranes within three years of installation (32). Consideration should also be given to the expertise of local contractors and the available construction equipment.

The discussion above has dealt with the technical and practical considerations involved in selecting the method of bridge deck rehabilitation but has made no mention of costs. Costs have always been of

paramount importance but vary so widely that it is difficult to generalize about them. Actual costs for concrete overlays in Ontario have ranged from \$55 to \$160/m² (\$5-\$15/ft²) of deck area. These costs include all of the work associated with the application of the concrete overlay, such as deck preparation, modification of deck drains and joints, paving of bridge approaches, and traffic protection. Costs for cathodic protection, expressed on the same basis, have ranged from \$85 to \$145/m² (\$8-\$13.50/ft²). The costs for patching, waterproofing, and paving depend heavily on the amount of patching required but tend to be somewhat lower than the costs of either a concrete overlay or cathodic protection (33). The costs for all three repair methods vary not only with such obvious items as the size and location of the structures involved but also with less tangible factors such as scheduling, other types of work included in the contract, and the overall volume of the construction work (2). To be meaningful, a cost analysis of the various technically feasible repair schemes must be made for each repair contract.

SELECTION OF THE REPAIR METHOD

Having completed the survey of the condition of the bridge deck, the engineer must use the survey results to select the most appropriate method of repair for that particular structure. A method used in Ontario has been to quantify the information summarized in Table 1 and to express it in the form of a decision matrix, as in Table 2. This method has proved very useful when the other factors discussed previously have been kept in mind. The criteria used in constructing Table 2 are based on experience

in Ontario to date but are subject to change as more information on performance, costs, and construction experience becomes available.

It should be noted that the decision matrix is constructed so that, by elimination, it leads to the identification of the rehabilitation scheme that is least objectionable rather than the one that is most suitable. This is not accidental; it is an acknowledgment of the fact that there is rarely an ideal solution and that the method of repair will always be a compromise between what is technically feasible and what is economically feasible.

In some cases, the matrix may exclude all of the methods of repair--as, for example, in the case of a deck with active cracks and with spalls and delamination over more than 5 percent of the deck area. In this case, it is useful to work through the matrix again and, on the basis of engineering judgment and cost estimates, to examine the implications of violating each criterion in turn. In the example quoted, the choice may well be between paying the high cost of patching the deck before the application of cathodic protection or accepting the risk of limited service as the result of cracking in a concrete overlay. In extreme cases, deck replacement may be the most economical solution. A possible alternative may be to combine more than one system. In the case cited above, for example, instead of patching the deck it may be possible to extend the service life of the deck by applying a concrete overlay and then either waterproofing and paving or applying cathodic protection.

CONCLUSIONS

Rehabilitating a bridge deck is both complex and challenging. Decisions often have to be made on the basis of inadequate performance data and in the face of serious operational and financial constraints. Sophisticated methods of analysis are not sufficient. Sound engineering judgment, an appreciation of all of the factors involved, and a systematic approach are the key elements in identifying the most appropriate method of rehabilitating any particular structure. Arbitrary selection of a repair method, often without visiting the structure, can no longer be tolerated. The condition and performance of structures are so highly variable that an individually engineered solution is required for every structure. This can only be done by completing a detailed condition survey and assessing the implications of the condition of the structure with regard to the cost of repairs and the future performance of the bridge deck for the various alternative rehabilitation methods.

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Publication of this paper sponsored by Committee on Corrosion.

Successful Application of Cathodic Protection to a Concrete Bridge Deck

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The effectiveness of the cathodic protection treatment of an Ontario concrete bridge deck after three years of service is evaluated. The Duffins Creek bridge was the first Ontario deck to be repaired and treated with cathodic protection. Half of the deck was treated, and the other half was left untreated. Corrosometer probes were placed in the treated half of the deck. These probes showed that cathodic protection was preventing further corrosion. After three years, the deck was stripped and the protected and unprotected sides were compared. It was found that the treated side had much less corrosion than the untreated side. It also became apparent that corrosion had occurred below the rebar, where the epoxy injection technique had been used for repairs. The most current Ontario system of cathodic protection has now been applied to the entire deck.

The Duffins Creek bridge, located on Ontario 7, 8 km east of Ontario 48, was the first of two Ontario bridges to be treated with cathodic protection. The bridge is 28 m long and 11 m wide and was built in 1967. Its concrete deck was left exposed. In 1974, the deck rebars were corroding, and this was causing serious spalling and delaminations to occur in the deck, especially along the centerline. The deck was surveyed for corrosion by using a copper/copper sulfate (Cu/CuSO₄) half-cell (1). It was found that potentials in excess of -0.35 V existed in a good portion of the deck, which indicated an active state of corrosion. The original corrosion voltage survey is discussed elsewhere (2,3).

The delaminations in the deck were repaired by using an epoxy injection, and the spalls were patched with concrete. Since at that time cathodic protection was still an experimental method, it was used only on the eastbound lane. The westbound lane was left unprotected for comparison. It was covered with dense asphaltic concrete to match the level of the protected eastbound lane.

A full description of the original method used for applying cathodic protection to the Duffins Creek bridge is given elsewhere (2,3). In this method, a series of Duriron anodes were applied to the deck and attached to it with epoxy cement. The

connecting wires were run along the deck to the curb, from the curb to the end of the deck, then down through a hole to the control panel. Graphite anodes were also used in order to compare their behavior and stability with those of the Duriron anodes. The entire deck surface was covered with an electrically conductive mixture of coke breeze and asphalt cement. The mixture was then compacted in the usual manner to a thickness of 5.0 cm and covered with 3.8 cm of surface mix, designated HL-1, which distributed the electric power evenly across the deck. Experiments showed that three electrodes down the center of the deck were sufficient to give an even distribution of power in the mix. Electrical resistance probes buried in the deck showed that corrosion stopped as soon as the power was applied.

The deck was under constant current control rather than the constant potential control applied to later decks. The system worked very well and required only two adjustments per year. The reason for this was that during the summer the concrete deck was drier than in the winter and its resistance changed. If the current was set to produce a polarized potential of, say, 1.0 V during the summer, this would produce a lower potential during the winter when the deck resistance was lower because of absorption of deicing salt solutions. In this case, an adjustment would have to be made.

The bridge deck was protected by this system for three years. During the second year, however, several fine random cracks appeared in the asphalt surfacing on the protected half of the bridge while the unprotected half remained uncracked. It was believed that the cracks developed as a result of water saturation in the conductive layer. This conductive layer was 80 percent by weight coke breeze and 20 percent by weight asphalt cement. Although this mix had excellent conductive properties (resistivity = 0.0148 Ω·m), it was low

in stability and high in air voids (17-18 percent by volume). When the bridge was treated with cathodic protection, the conductive layer was applied one day and the wearing course the next. Unfortunately, it rained heavily the morning the wearing course was to be applied, saturating the conductive layer. The wearing course would have helped to retain this moisture. Even if some of the water evaporated, a fresh supply of water could slowly percolate through the surface after each rain.

The high voids and low stability of this mix led to the development of a conductive mix with more suitable properties--i.e., a Marshall stability of 4400 N and voids of less than 5 percent volume (4).

During the summer of 1977, it was decided to strip the entire surfacing of this bridge to compare the protected and unprotected sides. The latest developments in cathodic protection were then to be applied to the entire deck.

PRELIMINARY WORK

The entire operation of stripping the deck, examination, testing, repair, and reapplication of cathodic protection was to be done in five working days. Traffic would be maintained in a single lane during the working day and returned to a two-lane operation at the close of each day.

Before the stripping of the deck, some exploratory work was done to see whether the old anodes could be removed intact for examination. One Duriron and one graphite anode located in the curb lane at the west end of the bridge were bared. When the HL-1 surfacing mix and the conductive mix were removed, water from the surrounding conductive mix flowed into the hole. The conductive mix still contained a lot of water. It was found that the epoxy cement holding the anodes to the deck could be broken by hammering around the edge with a light jackhammer.

Despite the water saturation, the conductive mix was in good condition and still had plenty of adhesion. There was no visible stripping of asphalt from the coke particles.

STRIPPING AND EXAMINATION OF DECK

The westbound lane was stripped first. This lane had no cathodic protection and was paved with 7.5 cm of HL-1 surfacing mix. The material was very dense and almost nonporous. A large backhoe was used to remove the HL-1 surfacing mix. This procedure was difficult because the HL-1 mix had adhered strongly to the deck. In many places where spalling had occurred, the concrete surface came up adhered to the asphalt. In several spots, the deck appeared damp, which indicated some moisture penetration. For the most part, the deck surface was dry. After the lane was completely stripped, the traffic was rerouted and the eastbound lane on the south side was stripped. Great care was taken in stripping this lane to recover all of the anodes. Since the conductive mix was much less stable and less adhesive, it was removed with less difficulty from the deck than was the HL-1 mix on the north side.

When the south side was stripped, none of the concrete deck surface came up with the asphalt conductive mix as had happened on the north side.

The protected lane had been powered only by the four anodes located down the center of the lane (2). When these four anodes were examined, there did not appear to be any metal loss from the three Duriron anodes. The one graphite anode showed no deterioration at all, and the edges were still sharp. Some of the other Duriron anodes, which had not been used since the beginning of the test,

showed a slight stain of rust in some areas. The deck in the southeast corner had a slight rust stain from the anodes. This was caused by the water in the conductive mix. The powered anodes did not show rust stains.

Once each side was stripped, a preliminary Cu/CuSO₄ half-cell survey was run on each side to determine the corrosion potentials, which were found to be much higher than expected. Those read on the south (protected) side were expected to be high because they would be the polarized potentials remaining after the power had been shut off in the morning. These potentials were not plotted on a graph. It was planned to resurvey the deck two days later when the polarized potentials had leaked off and the normal corrosion potential was present.

EXAMINATION AND TESTING OF THE DECK

On the second day, the north side of the deck was chain dragged to enable the delaminations to be outlined and mapped. Many delaminations were found in addition to those in the areas where the concrete surface had come off adhering to the HL-1. The delaminations were removed with a jackhammer and patched with cement.

The south side of the deck was chain dragged, and the delaminations were outlined and plotted. Here, they were much fewer in number and smaller than on the north side. A map of both sides of the bridge deck that indicates the areas of delamination and spalling is shown in Figure 1. These delaminations, too, were removed with a jackhammer and patched with cement.

It was noted that several delaminations on the south side had previously been repaired by use of epoxy injection. It was thought that in these cases a delamination level with the top of the rebar had been repaired. The injected epoxy resin then prevented the flow of current to the bar in this area, and further corrosion took place below the bar. Several other shallow delaminations on the south side of the deck could have been the result of freeze-thaw action, since so much water was trapped in the conductive mix. The delaminations on the south side were, however, less frequent than on the north side.

The delaminations that occurred along the bridge-deck centerline (concrete cover of 19 mm or less) were more serious on the north (unprotected side) but did spread over the south side. This extension into the south side is believed to be a result of the forces generated by the corrosion produced on the rebars in the north side of the centerline.

On Thursday, August 25, 1977, the entire deck was again surveyed for corrosion potentials by using a Cu/CuSO₄ half-cell. Again, very high potentials were found on both sides of the deck. The protective current had been cut off for three days. These potentials are shown in Figure 2.

INSTALLATION OF CATHODIC PROTECTION

The most recent system of cathodic protection used by the Ontario Ministry of Transportation and Communications (MTC) was used on this deck. All anodes and voltage probes were counter sunk in the deck so that their surfaces were bare and were flush with the deck. The half-cells were buried in slits cut in the deck level with the upper rebars and were covered with concrete. All connecting wires were set in saw cuts in the deck and were run to the curb. The wires along the curb were encased in concrete 10 cm wide and 5 cm thick. Thus, all electrical elements were flush with the deck or

Figure 1. Spalling and delaminations in Duffins Creek bridge deck.

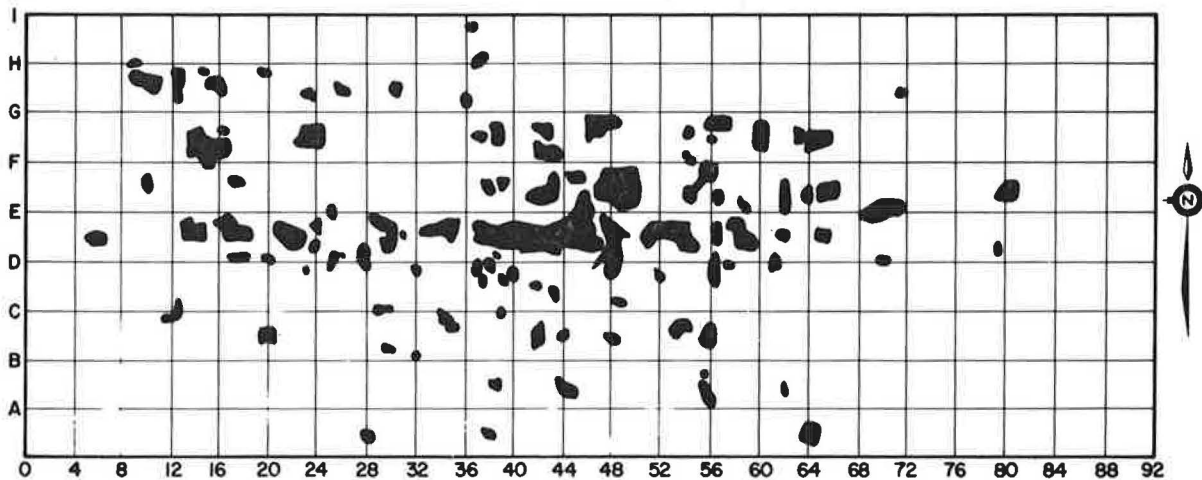
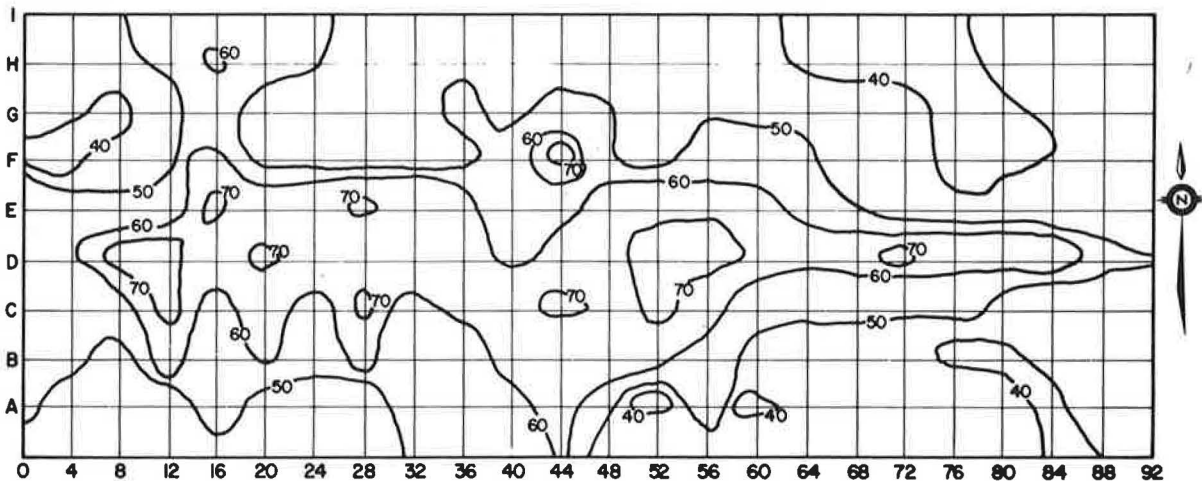


Figure 2. Copper half-cell potentials in Duffins Creek bridge deck.



encased in concrete along the curb. If at any time in the future it is necessary to remove and replace the surfacing of the concrete deck, this can be done without disturbing the electrical elements or the circuitry.

The layout of the anodes, voltage probes, and half-cells is shown in Figure 3. The design called for three anodes down the centerline--anodes 1, 3, and 5 in Figure 3. These are connected in parallel to the main bus, which in turn runs to the power supply. Anodes 2 and 4 are connected to a secondary bus. These were also installed in the event of unforeseen problems. Anodes 2 and 4 can be run separately or in addition to those on the main bus.

Six graphite voltage probes were sunk in the deck so that their upper sides were flush with the top of the deck. These probes were individually connected to the jacks on the control panel.

Six half-cells, three Cu/CuSO₄ and three zinc/zinc sulfate (Zn/ZnSO₄), were buried in the deck and connected by individual wires to the control panel. Four of the half-cells--Cu₁, Cu₃, Zn₁, Zn₂--were located about 2.5 cm below the surface and roughly midway between the rebars. Half-cells Cu₂ and Zn₃ were located within 1 cm of a rebar. This was done to determine whether cell location relative to the rebars would have any bearing on the polarized voltage indicated and also

to select the location best suited for potential control, the system later to be added to the bridge. A wiring diagram is shown in Figure 4 (all wires from probes and half-cells north of the centerline run to the north curb duct, and those from probes and half-cells south of the centerline run to the south curb duct).

At the end of the fourth day, all of the electrical equipment had been installed, the bridge-deck repairs were complete, and the deck was ready for paving. Resistance checks were run on all anodes, voltage probes, and half-cells to make sure that no shorts to ground had occurred. All systems checked out, and no adjustments had to be made.

PAVING OF DECK

On the morning of the fifth day, the surfaces of all anodes and voltage probes were cleaned with wire brushes and abrasives to ensure good electrical contact. A special conductive mix was designed for this project. Its composition and properties are given below:

Item	Value
Mix composition (%)	
Stone retained on 4.75-mm sieve (by weight)	40

Figure 3. Placement of anodes, probes, and half-cells.

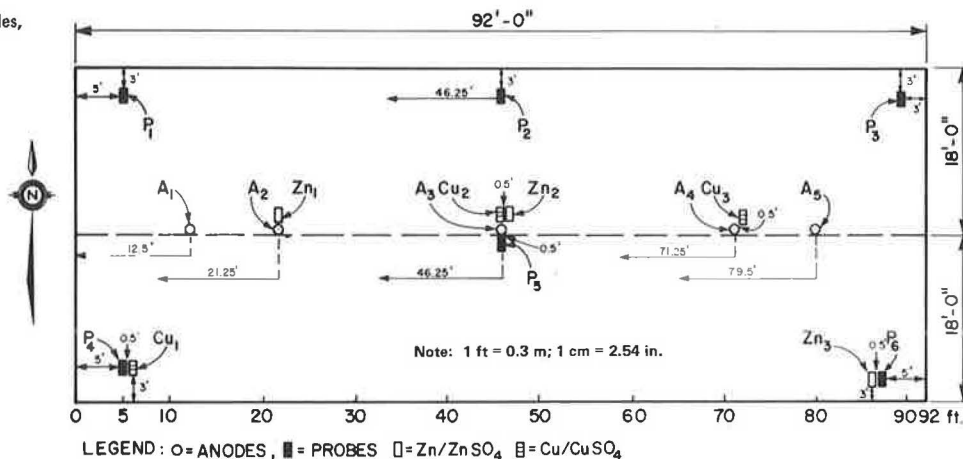
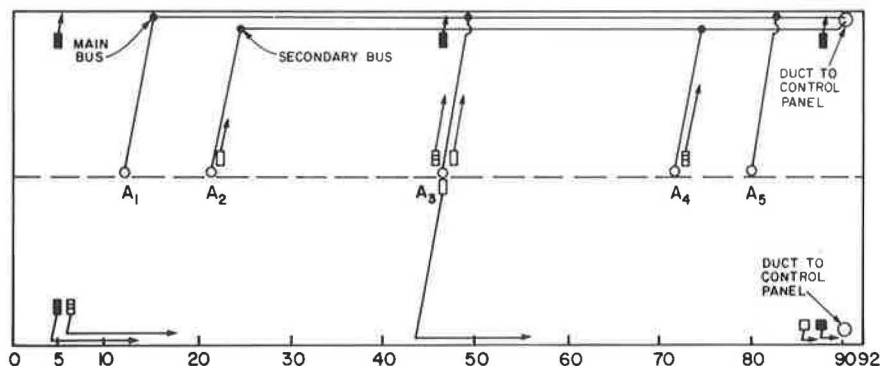


Figure 4. Wiring diagram.



Item	Value
Sand passing 4.75-mm sieve (by weight)	15
Coke breeze (by weight)	45
Asphalt cement, 85/100 penetration (by weight of total aggregate)	14
Mix property	
Marshall stability (N)	4359
Marshall flow	13.8
Voids in mineral aggregate (percent by volume)	32.5
Voids (percent by volume)	8.2

A better mix would have been one that contained 15 percent asphalt. Such a mix would have had 5 percent air voids and a stability of >1000.

The mix was laid with a regular paver and compacted to a depth of 5 cm with steel and rubber-tired rollers. There were no problems in laying or compacting the mix. The conductivity of the mix was tested behind the steel finishing roller and was found to be acceptable.

The mix was kept away from the expansion joints by laying 5x15-cm planks on the deck against the edges of the steel joints. After the mix was compacted, it was dug away from the curb scuppers for a distance of 15 cm. These actions prevented any short circuits between the conductive mix and the bridge steel.

The eastbound lane was the first to be paved with the conductive mix. After it was compacted, the traffic was routed onto it while the westbound lane was paved. The conductive mix was able to bear the traffic, including trucks, with no problems or dam-

age. Once both lanes were covered with the conductive mix, 3.8 cm of HL-1 wearing course was paved over the deck to complete the job.

POWER DISTRIBUTION ACROSS THE DECK

Once the paving was complete, power was applied to the deck by the original constant current rectifier to provide cathodic protection. Power was applied to anodes 1, 3, and 5 of the main bus. A larger current than necessary (0.8 A) was applied to build up the polarized potential on the steel more rapidly. Three days later, this current was reduced to 0.4 A, and the bridge was allowed to reach electrical equilibrium. At equilibrium, the power distribution was as follows:

Probe Number	Power On (V)	Polarized Potential (V)
1	1.25	1.14
2	1.24	1.14
3	1.18	1.12
4	1.22	1.15
5	1.30	1.16
6	1.20	1.12

Anodes 1, 3, and 5 were sufficient to distribute the power across the deck so that an even, polarized potential was obtained on the steel. Anodes 2 and 4 were not used.

The potential on the rebars was also measured by taking voltage readings between the half-cells in the deck and ground. The technique used is described elsewhere (5). Initially, it was found that the two zinc and copper cells closest to the rebars gave higher readings than the four placed

midway between the rebars. After a few days, the readings between the two sets became similar. It was concluded that a half-cell close to a rebar would give a better, more sensitive type of control than one placed farther away.

A new potential control rectifier has now been obtained, and the bridge is now being controlled by the zinc half-cell close to a rebar.

Cathodic protection is now accepted as a suitable system for bridge-deck repair by MTC. Other accepted methods in Ontario are the use of low-slump concrete and the use of latex-modified concrete overlays.

CONCLUSIONS

1. Cathodic protection is successful in preventing or retarding the corrosion of bridge-deck steel. This was shown on the Duffins Creek bridge through the corrosometer probes and by the fact that there was a much lower degree of damage on the protected side of the bridge deck.

2. Examination of the deck after three years of cathodic protection showed that epoxy injection was not a good method of bridge-deck repair.

3. A conductive mix that has a low void content should be used to prevent water absorption.

4. Cathodic protection is now accepted in Ontario as a method for protecting bridge decks after repair.

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Publication of this paper sponsored by Committee on Corrosion.

New Approach to Cathodic Protection of Bridge Decks and Concrete Structures

J. P. NICHOLSON

Until now, cathodic protection of bridge decks has been accomplished by installing anodes on the concrete surface or by recessing them into the concrete and covering them with a conductive paving layer to spread the current over the entire surface of the structure to be protected. Tests are reported in which wire anode consisting of platinized niobium was installed in a bridge deck in sawed slots with conductive backfill. Tests to date indicate that if the wire anode is carefully spaced a bridge structure can be protected without using a conductive paving layer. This eliminates to a great extent the cost of conductive paving and of other wearing courses required to protect the conductive paving layer.

The deterioration of concrete bridge decks and support structures occurs worldwide, but the deterioration can be accelerated by the use of deicing salts or by salt spray in coastal areas. Since the pH of concrete is generally in the range of 12.5-12.8, one would normally anticipate that steel would not corrode in concrete. In this pH range, steel is usually passive; in the presence of chlorides, however, corrosion of the reinforcing steel can occur quite rapidly. In the United States, the U.S. Environmental Protection Agency estimates that the annual cost of bridge damage caused by deicing salts is about \$0.5 billion/year. No doubt the damage to bridge structures in Canada is high, since deicing salts are used extensively during the winter months, but I have not been able to locate any estimates in the literature of the cost of this damage.

Slater and others (1) have suggested that chloride levels of 0.02 percent or less be considered the threshold value for corrosion of reinforcing steel. In their investigations, the

steel remained passive at levels below this figure and, at chloride concentrations above the 0.02 percent level, corrosion occurred. Slater and others did some investigation on chloride concentration at various depths in concrete bridge structures. As data given in Table 1 show, chloride concentration decreases with the depth of concrete cover. This is no doubt attributable to the permeability of the concrete and the ability for ionic transfer.

Before a bridge deck can be protected, the criteria for protection of steel in concrete must be determined. Many papers suggest that National Association of Corrosion Engineers (NACE) standard RP-01-69 should apply to concrete structures that contain reinforcing. This standard offers three criteria for establishing whether or not a structure is cathodically protected.

To intelligently use these criteria, it is necessary to understand how they were established. Schwertdferger and McDorman (2) did some analytic work on the current and potentials required for the protection of steel in soils. Their investigation indicated that "cathodic protection is the maintenance of a critical potential at the surface of the cathode." This potential, which Schwertdferger and McDorman define by the point of intersection of the potential-pH curve for steel in air-free soils and the potential-pH curve for the hydrogen electrode at atmospheric pressure, was found to be approximately -770 mV referred to the saturated calomel electrode, or -530 mV referred to the standard hydrogen electrode. This converts to -850 mV re-

Table 1. Average chloride content of cores taken from bridge deck before electrochemical treatment.

Depth of Sample from Deck Surface (cm)	Chloride Content					
	Cores Taken in November 1973		Cores Taken in August 1974		Cores Taken in April 1975	
	Amount (kg/m ³)	Percent	Amount (kg/m ³)	Percent	Amount (kg/m ³)	Percent
0.25	6.94	0.31	8.95	0.40	9.2	0.41
3	3.5	0.16	5.16	0.23	4.5	0.20
5.8	1.12	0.05	1.36	0.06	1.36	0.08
8.6	0.23	0.01	0.23	0.01	0.23	0.01
11.4	0.23	0.01	0.23	0.01	-	-

Note: Chloride content based on dry concrete weight of 2242 kg/m³. Data are average values obtained on two cores.

Figure 1. Potential of steel in air-free soils versus soil pH.

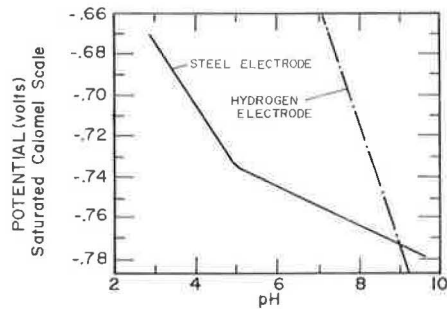


Figure 2. Critical polarization potential for cathodic protection of steel in saturated lime solutions containing sodium chloride.

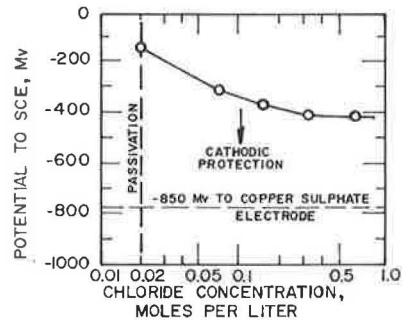
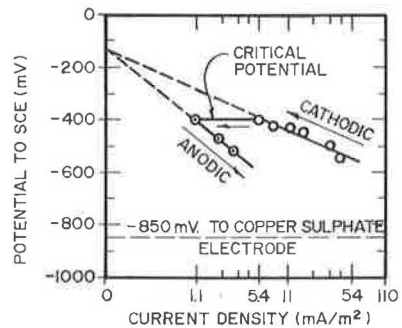


Figure 3. Characteristic change in current during cathodic depolarization of steel in saturated lime solution containing 4.4 percent sodium chloride.



ferred to the copper/copper sulfate (Cu/CuSO₄) electrode (see Figure 1). This does not necessarily mean that the criteria in the NACE standard can be used as an indication of satisfactory protection of steel embedded in concrete in the presence of chloride ions.

In his work, Hausmann (3) determined from analytical procedures that the critical protective po-

tential is a function of chloride concentration for steel in concrete. The critical potential was plotted as a function of chloride ion concentration (see Figure 2). This curve is discontinuous at a chloride concentration of 0.02 molar, the threshold concentration previously determined for passivation of mild steel in saturated lime solutions. At a chloride concentration of 0.64 molar (4.4 percent sodium chloride), the critical polarization potential is -435 mV to a saturated calomel electrode, or approximately -515 mV to a Cu/CuSO₄ electrode. Hausmann found from the current-potential relationship during depolarization that the cathode potential became less negative as a logarithmic function of the current density. This relationship, shown in Figure 3, is consistent with observations made by Tomashov (4).

This relationship held until the cathode was depolarized to its critical polarization potential and corrosion occurred. At initiation of corrosion, the current reversed direction and subsequently increased in magnitude. The potential also appeared to be a logarithmic function of current density. The cathodic and anodic potential curves intersect at the base potential of bare steel and saturated lime solution. The current at the projected intersection is shown as zero in Figure 3 (the current scale is discontinued at about 1.1 mA/m²). At lower current densities, the cathodic depolarization curve is to be considered linear, as suggested by Tomashov (4). In his conclusions, Tomashov states that corrosion of steel can be prevented in concrete exposed in high-chloride-content environments if sufficient current is applied to shift the steel polarization potential to a minimum value of -515 mV to a Cu/CuSO₄ electrode and that corrosion of steel can be arrested in chloride-contaminated concrete if sufficient current is applied to shift the polarization potential to a minimum value of -710 mV to a Cu/CuSO₄ reference electrode.

In some polarization scans, I have found that, in freely corroding reinforcing steel in chloride-contaminated concrete, the minimum potential for cathodic protection was -700 mV to a Cu/CuSO₄ reference electrode, which agrees very well with the values given by Hausmann (3) and Tomashov (4).

The conclusion of many authors is that cathodic protection for reinforcing steel in concrete is practical and can preserve the structure from the degradation resulting from corrosion of the reinforcing steel induced by chlorides. The present state of the art is to use graphite or high-silicon cast-iron anodes on the surface or slightly recessed into the surface of the concrete structure and then to use a conductive paving layer to offer a low-resistance, electronically conductive path for the cathodic protection current to reach remote parts of the structure. This method is both cumbersome and expensive. The conductive paving

Figure 4. Model reinforced-concrete deck.

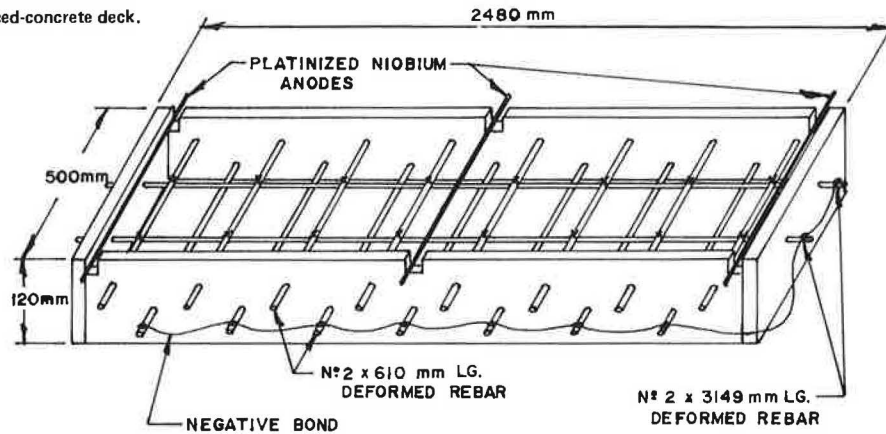


Figure 5. Reference potential locations in experimental deck.

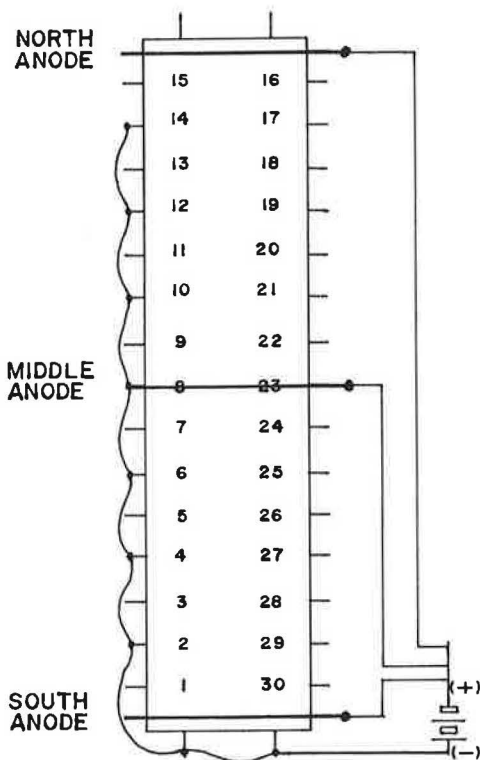
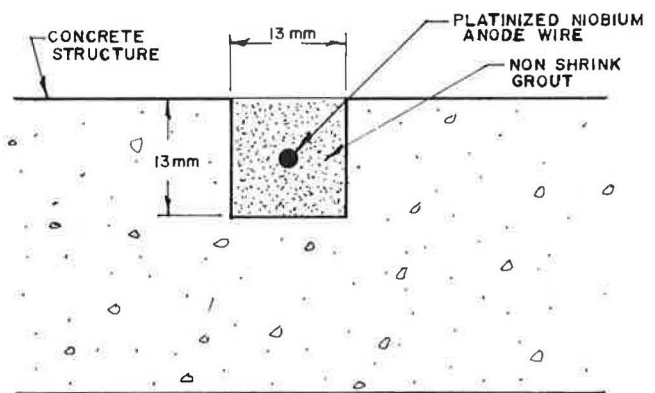


Figure 6. Installation of anode wires in saw slots.



layer is difficult to mix and apply and adds greatly to the weight of the structure.

It has been theorized that, with the new advances in anode materials, cathodic protection of reinforced concrete structures could be simplified and the cost reduced greatly.

It is difficult for a consultant who does not own a bridge structure to experiment with cathodic protection to determine the design parameters needed to protect reinforcing steel in chloride-contaminated concrete. Nevertheless, a small-scale experiment was set up to try to duplicate the conditions that would be experienced on a large, reinforced-concrete structure. Figure 4 shows the model reinforced-concrete deck that was fabricated. The model deck was deliberately fabricated with a thin (120-mm) cross section to accentuate any shielding effects that the upper course of reinforcing bars might have on the lower course. In addition, the use of a thin cross section reduced the effective current path,

thus creating a worst case for current distribution. The deck also had a concrete surface area (1.115 m²) equivalent to the total area of embedded steel, which corresponds to the situation found in actual bridge decks.

The plan view in Figure 5 shows reference potential locations that were used throughout the experiment. Various anode configurations were tried. One platvanized niobium anode was embedded in the concrete when the model bridge deck was poured. Three additional anode wires were installed in 13x13-mm saw slots and grouted in (see Figure 6). A variety of grouting materials that were both electronically and electrolytically conductive were used.

The bottom layer of reinforcing steel in the experimental model was not connected internally to the longitudinal bars but was bonded together with an external copper conductor so that current flow to the individual bars could be monitored during the experiment. This proved impractical, however, because of the low current densities required to achieve satisfactory potentials in the actual experiment.

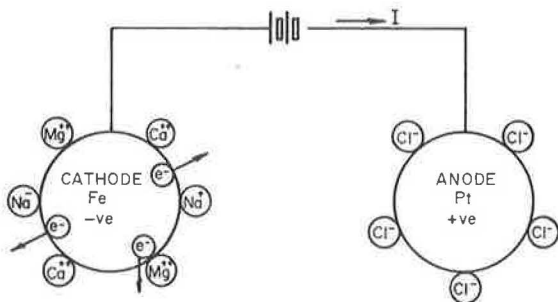
Table 2 gives the potential of the reinforcing steel recorded over a seven-month interval. The first potential measurements were taken on March 10, 1978, approximately 48 h after the concrete was poured. The range of the potential of the reinforcing steel at that time was approximately -400 to -500 mV measured to a Cu/CuSO₄ reference electrode. The potential of the steel varied daily for more than 30 days as the concrete cured. Cathodic protection was then applied to the structure. Potentials recorded in May and June of

Table 2. Experimental results of bridge-deck cathodic protection.

Half-Cell Location	Potential (-mV to Cu/CuSO ₄ half-cell)										
	Current On				Current On			Current Off			
	March 10 ^a	May 23, 27 mA/m ²	June 8, 36 mA/m ²	July 8 ^b	Aug. 18, 20 mA/m ²	Sept. 5, 16.2 mA/m ²	Sept. 12, 24.3 mA/m ² (salt)	Sept. 19, 27 mA/m ² (salt)	Sept. 25, 11 mA/m ² (salt)	Oct. 3, 22 mA/m ²	Oct. 17, 27 mA/m ²
1	445	980	1080	290	1050	1350	1250	1460	650	840	1000
2	440	600	650	300	625	700	820	1310	600	740	830
3	430	520	580	320	525	570	690	1240	570	640	770
4	430	530	570	330	480	515	640	1200	550	650	720
5	435	530	600	360	470	500	650	1190	540	650	710
6	520	550	650	345	470	530	735	1230	570	700	760
7	425	700	850	360	640	650	960	1350	640	840	940
8	420	3750	4200	340	7800	9800	880	1380	580	840	920
9	435	790	880	355	740	790	1040	1400	620	810	940
10	425	570	620	350	490	500	720	1250	570	690	750
11	415	520	540	360	440	450	600	1180	540	630	700
12	480	495	530	350	420	400	610	1180	530	625	690
13	410	510	550	325	470	410	660	1220	540	665	720
14	440	610	680	340	690	550	900	1320	580	750	810
15	415	900	1140	340	1700	1180	1410	1420	620	830	900
16	415	860	980	340	1100	1160	1520	1370	630	780	1010
17	445	585	680	340	660	530	915	1300	620	725	860
18	400	505	530	340	500	410	690	1270	575	675	780
19	460	490	500	345	410	400	620	1210	570	650	760
20	410	510	530	360	430	440	615	1210	575	660	760
21	470	570	620	365	520	530	730	1280	600	725	830
22	440	750	860	355	960	870	1150	1400	630	820	930
23	415	3750	4300	325	7700	9900	880	1420	620	820	1000
24	430	660	780	350	715	670	1060	2330	670	780	990
25	510	540	600	345	480	520	750	1270	600	715	820
26	425	510	550	340	440	460	640	1220	560	670	740
27	440	510	550	340	470	495	630	1210	550	660	720
28	430	520	570	330	515	540	690	1250	560	665	750
29	455	600	640	300	600	640	870	1300	580	700	790
30	450	980	1040	310	1220	1920	1420	1440	580	785	860

^aTwo days after pour.^bStatic readings after current was off for 30 days.

Figure 7. Cathodic protection system operating in presence of chloride ions.



1978 at current densities of 27 and 36 mA/m², respectively, show the potential of the structure when the rectifier was operating. No allowance was made in these readings for drops in voltage caused by electrolyte resistance. Since very little current spread could be obtained, it was decided to turn the rectifier off for a time to reestablish the static potentials. Static readings were taken on July 8, 1978, after the current was off for approximately 30 days. The static potentials were in the range of -300 to -400 mV. Current was then reapplied to the deck. The controller was set to hold the current density at 27 mA/m². Readings taken in August and September, during a period of extremely dry weather, show very little spread of current away from the anodes, and the controller was unable to maintain the current density required because of voltage limitations.

By mid-September, the model bridge deck was six months old and had fully cured in a weathering

exposure, although July and August had been unusually dry. It was then decided to continue the experiment and apply rock salt to the surface of the deck on a weekly basis at an application rate of 0.047 kg/m² and to allow natural rainfall to wash this salt into the concrete matrix. After the results for the first six months were examined, it was theorized that the addition of chlorides would increase the conductivity of the concrete matrix and thus assist the spread of cathodic protection currents. This proved to be the case after the application of the salt. Measurements taken on September 12, 19, and 26 show the change in current spread in the structure and the potential of the reinforcing steel after the application of salt. On September 19, the lowest potential recorded when the cathodic protection current was interrupted was -1210 mV.

All of the measurements given in this paper are the result of a constant-current cathodic protection system, since the initial data desired were minimum current density and maximum anode spacing. In practice, a potential-controlled rectifier would be used and the potential of the reinforcing steel would be maintained at -1000 mV to Cu/CuSO₄. As the literature suggests, a potential of -700 mV or more will afford protection of reinforcing steel in chloride-contaminated concrete.

After the September 19 measurements were taken, current density was reduced to establish the density required to maintain the structure at approximately -700 mV to a Cu/CuSO₄ reference electrode with the current momentarily off; this produced the lower potentials recorded after September 19.

The initial results obtained in this experiment and the literature review on the protection of reinforcing steel in concrete indicate that a reinforced-concrete structure can be protected by

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Transportation Research Record 762

page 17, column 2, reference 4
Change "1976" to "1966"

Transportation Research Record 778

page 36, column 2, line 20
Change "\$6000" to "\$660 000"

Transportation Research Record 790

page 74, column 1, Equation 1
Change to " $Y = b_0X_0 + b_1X_1 + b_2X_2 + b_3X_1^2 + b_4X_2^2 + b_5X_1X_2$ "

page 75, column 1, Table 1
Change "Variable" column to
"Variable

X_0
 X_1
 X_2
 X_1^2
 X_2^2
 R^2 "

Transportation Research Record 792

page 2, column 2

Insert the following before the last paragraph:

"The speakers present papers that indicate how they have taken steps to reduce such adversary relationships in contractual work, and provide various evaluations of the resulting work. The various papers are placed in proper perspective to provide an overall introductory picture of the subjects to follow this introduction. Three teams of three authors each present viewpoints on three projects, and two authors add their thinking to the seminar."

"This seminar examines three other projects, each of which is addressed by three speakers with three different points of view, namely the owner's, the contractor's, and the engineer's or the Federal Highway Administration's representative. The projects are

1. West Virginia Department of Highways Quality Assurance Program;
2. Eisenhower Memorial Tunnel, Second Bore, in Colorado under Loveland Pass; and
3. Pittsburgh's South Busway.

"In addition we have

1. A paper by two researchers from Virginia.
2. Some thoughts by a service engineer of a large corporation who is constantly out in the field looking at all these problems and thus is in a position to observe what is going on."

Transportation Research Record 797

page ii, price should be \$7.20

Transportation Research Record 816

page 34, Table 6, line 9, column "Realistic Saving"
Footnote b-Change to "0 to 3.8"
Change "Annual liters of fuel saved . . ." to "1000's of liters of fuel saved annually . . ."

page 29, column 1, line 21
Change "millileters" to "milliliters"

Transportation Research Record 834

page ii
Change subject areas to 13, 15, 25
Change mode to 01 only

Preprint Volume for the National Seminar on Portland Cement Concrete Pavement Recycling and Rehabilitation

page 94, column 2, last line
Change "a 0.241-cm (3/4-in.)" to "0.241-cm (0.095-in.) diamond sawblades at 1.9 cm (3/4 in.)"

page 96, column 2, paragraph 2, line 3
Change "(3/15-in.)" to "(3/16-in.)"

page 98, Figure 33, line 3
Change "apepar" to "appear"

page 98, column 2, line 5 below Figure 35
Change "(51,000 sq. yds.)" to "(57,000 sq. yds.)"

NCHRP Report 238

title page, author's name
Change "Shebr" to "Shelar"

NCHRP Synthesis of Highway Practice 66

page 5, caption for Figure 3
Change to ". . . as a type II . . ."

NCHRP Synthesis of Highway Practice 69

Foreword, page iv

Delete paragraph 3

page 13, Table 2, item 2.6
Change formula to $T_t = \sum P_i(t_i + \sqrt{h})$

page 41, column 2
Change formula to $T_t = \sum P_i(t_i + \sqrt{h})$

page 45, Table 15, title
Change to "GUIDELINES FOR SERVICE CHANGES: (Port Authority of Allegheny County)"

page 86, box under Toronto, item 2-6
Change formula to $T_t = \sum P_i(t_i + \sqrt{h})$

NCHRP Synthesis of Highway Practice 76

page 2, line 5
Change "\$50" to "\$25"

page 7, column 2
Change "i = 1" to "l = 1"

i = l to i = 1

page 13, Table 9, under Pennsylvania
Change "10%" to "100%"

page 16, column 1, line 18
Change "\$50" to "\$25"

page 23, column 1, line 29
Change "\$50" to "\$25"

using platinized niobium anode wire without installing a conductive layer on the surface of the structure. The experiment with the model deck will be continued to optimize design criteria for the anode spacing and current densities required to protect reinforcing steel in chloride-contaminated concrete.

A cathodic protection system operating in the presence of chloride ions performs two beneficial functions:

1. It prevents the corrosion of the reinforcing steel by maintaining a surplus of electrons on the steel surface, thus preventing the migration of the iron ions into solution.

2. The chloride ion is attracted to the anode, and this reduces the chloride concentration in the vicinity of the reinforcing steel (cathode) (see Figure 7), thus increasing the pH of the cathode.

The cost of protecting a reinforced-concrete structure by using a conductive paving layer is about \$30/m², whereas platinized niobium wire anodes installed in saw slots would cost about \$12/m². It is anticipated that this cost will decrease further as design parameters are refined and installation methods perfected.

This new concept of cathodic protection was applied to a new 803-m² deck in the fall of 1979. In that application, 600 m of 0.8-mm platinized niobium anode wire was installed in 10x13-mm saw slots cut in the deck at 1-m intervals. The wire anode was grouted in the slot and cathodic protection applied. After approximately 350 h at a current density of 18.5 mA/m², a potential of -770 mV to Cu/CuSO₄ was achieved midway between the anodes, and the

potential-controlled rectifier output was automatically reduced to approximately 9.0 mA/m², the current required to maintain -770 mV after polarization was achieved. During January 1980, the circuit resistance varied between 0.9 and 1.2 Ω between the unfrozen and the frozen condition.

The deck has been given numerous applications of deicing salt and has undergone many freeze-thaw cycles without loss of protection or damage to the anode grout material.

The system is a viable method of cathodically protecting reinforcing steel in concrete in both the vertical and horizontal position without conductive overlays. It lends itself to the protection of lightweight decks, parking garages, and support structures for bridges and docks.

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Publication of this paper sponsored by Committee on Corrosion.

Cathodic Protection for Continuously Reinforced Concrete Pavement in Minnesota

R. G. TRACY

The corrosion of steel in concrete can be suppressed by the use of cathodic protection, which involves applying a low-voltage direct current to the steel from a remote anode so that corrosion is transferred to the remote anode and the steel becomes a protected cathode. The results of the application of cathodic protection to continuously reinforced concrete pavement (CRCP) in Minnesota are presented and discussed. Several segments of CRCP are undergoing rapid, premature deterioration that is directly related to corrosion of the embedded mesh reinforcement. Pavement testing revealed that salt concentration at the reinforcement is high, and copper/copper sulfate half-cell potentials indicated widespread corrosion activity. Essential elements from pipeline and bridge-deck applications of cathodic protection were integrated, and a prototype system was installed along a 1000-ft section of CRCP. Two methods of power (current) application were examined: (a) burying anodes in a trench filled with a conductive aggregate and (b) burying anodes in individual postholes along the pavement shoulder. Both installations were connected to a central rectifier controller, which was interfaced with an automatic device for monitoring and recording the data. An initial data evaluation, expected by late summer of 1980, will provide information on the performance and effectiveness of the system.

During the past three years, an increasing number of continuously reinforced concrete pavements in Minnesota have been exhibiting a spalling type of

deterioration. The frequency and extent of this deterioration have progressed from isolated and random in 1975 to widespread and concentrated on certain pavement designs in 1978. The pavements that show severe and moderate delamination and spalling are of the two-course construction type designed with a steel-to-concrete ratio of 0.6 percent. The reinforcement used was deformed wire mesh with specified clear cover of 2-4 in. In most cases, steel was at the minimum cover of 2 in.

BACKGROUND

Construction of continuously reinforced concrete pavement (CRCP) in Minnesota began in 1963 with the placement of a rather extensive test section on I-35W near Faribault. It was hoped that this trial would provide some specifics on construction techniques, design adequacy, and short-term performance. Variable ratios of steel to concrete—0.5, 0.6, and 0.7 percent—were used, and different combinations of base-course thicknesses

and end-anchor types were tried. The study was completed in 1968, and a project report (1) was prepared.

Study results indicated that a minimum steel-to-concrete ratio of 0.6 percent was needed for proper pavement performance. It was also apparent that smooth wire mesh is not acceptable as reinforcement for CRCP. Pavement design was modified to reflect the study findings, and construction of CRCP as standard practice began in 1967.

Three CRCP sections were completed in the Minneapolis-St. Paul (Twin Cities) area during 1967. Other sections were designed and built at various locations in Minnesota between 1967 and 1970. By 1970, however, CRCP was dropped as a pavement design, partly because of the continued evaluation of the original test section at Faribault and evidence of rupture (tension failure) on some of the first sections in the Twin Cities metropolitan area. A 1970 report that documents the field performance of CRCP reveals evidence of corrosion at transverse cracks (2).

Isolated cases of shallow potholing began to show up on the oldest sections of CRCP early in 1974. I-94 from Cedar Avenue to Riverside Avenue, and again from the east end of the Dartmouth Bridge to MN-280, exhibited what appeared to be random and minor spalling. A second area of this type of deterioration also appeared on several miles of I-35W near Arden Hills. Initial spalling on I-94 appeared to be somewhat concentrated immediately beneath overpasses. Spalling of the I-35W section, however, was random. A distinguishing feature common to both sections was that spalling generally occurred in the wheel-path zone. The spalling condition continued to grow more severe, and during 1975 open holes frequently had to be patched.

Maintenance patching of the two sections mentioned above had reached a significant level by 1976. On the 4000-ft section of I-94 between Cedar and Riverside Avenues, a five-member maintenance crew spent nearly one month chipping and patching holes. Similar operations were occurring at the I-35W location. Spalls were now also exposing deformed wire mesh on ramps near the deteriorating pavement sections. Other sections of pavement not previously reported as showing surface potholing also required patching in 1976.

In August 1976, research personnel from the central office of the Minnesota Department of Transportation (DOT), while investigating bridge-deck repair and protection systems, conducted a brief survey of the deteriorating I-94 CRCP section. Tests performed during this survey included cover measurement, delamination detection, and half-cell potentials for corrosion detection. A visual survey was also performed. Strong evidence was found to support the theory that (chloride) corrosion-induced spalling was now occurring on CRCP. A discussion with the maintenance foreman and crew revealed that all patched spalls extended to the welded-wire-fabric reinforcement. Massive pack rust was evident on both the bars and the underside of concrete removed from delaminated areas.

Results from the survey of half-cell potentials indicated that all readings were at or well into the corrosion range of greater than 0.35 V to a copper/copper sulfate (Cu/CuSO_4) half-cell (CSE). The delamination survey revealed that 13 percent of the pavement tested was delaminated. Further discussion with the foreman provided additional information. His estimate was that, for each open spall patched, three or four hollow (delaminated) areas were being left untreated. At the time of the survey (August 1976), almost all of the noticeable

spalling type of deteriorations had been confined to the oldest sections of CRCP in the metropolitan area.

It is interesting to note that two-course construction was used on the distressed pavement sections. The continuous-reinforcement steel used was a welded wire mesh fabric that consisted of deformed 0.45-in-diameter wires on 3-in centers. The ratio of steel to concrete was 0.6 percent for both the I-94 and I-35W sections that showed distress. Specified cover for the bar mats was 2-4 in; however, most of the steel was closer to the minimum cover.

Joint survey efforts by district and central office personnel were initiated to collect information, first for the two rapidly deteriorating sections of pavement (I-94 and I-35W) and then, on a much broader scope, for all sections of CRCP in Minnesota. The results of the survey of the Twin Cities metropolitan area showed the following:

1. Three sections (3.75 miles) of Interstate are in a rapidly deteriorating, or critical, condition. Delaminated areas and open spalls constitute 20-30 percent of the roadway area.
2. Classified as being in fair condition is 13.75 miles of Interstate that is showing some evidence of the beginnings of the spall type of deterioration.
3. On 39.10 miles of Interstate classified as good, there is currently no evidence of spalling.

The first assessment of this situation, based on miles assigned to each category, may lead one to believe that problem pavement sections are only a small fraction of the total pavement in place and so as such are not indicative of CRCP performance. Although this may be true, there are other factors that deserve consideration.

Two of the three sections currently classified as critical are among the highest-volume Interstate sections in Minnesota, carrying more than 100 000 vehicles/day. The third section carries 25 000 vehicles/day. The six sections (13.75 miles) classified as fair are also broken down according to daily traffic volumes: One carries more than 110 000 vehicles, four carry from 55 000 to 80 000 vehicles, and one carries 13 000 vehicles. The remaining six sections, those classified as good, carry from 11 000 to 32 000 vehicles/day.

There is reason to believe that deterioration of the type encountered will continue on the critical sections of pavement. It is highly probable that, in time, it will advance into those sections currently classified as fair. A case in point is a section of I-94 between Snelling Avenue and MN-280 in St. Paul. During 1975 and 1976, there were perhaps 10-15 spalls on the entire 2-mile, six-lane section of CRCP. By the summer of 1977, the number of spalls had exceeded 100. In 1978, spalls were occurring with a frequency that made counting and patching futile.

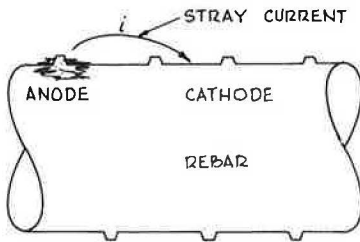
EXPERIMENTAL SYSTEM

In late 1978 and early 1979, an experimental cathodic protection system was installed along a 1000-ft section of two-lane CRCP near Arden Hills, Minnesota. Two methods of anode installation were tried to determine which would be the easiest and most effective.

Design Concept

One of the few systems available today for stopping stray-current corrosion of steel is cathodic protection. It has been successfully implemented to

Figure 1. Electrical aspect of the corrosion process.



protect buried pipelines for more than 30 years (3). During the past 4 years, it has proved an effective means of arresting the corrosion of rebars in concrete bridge decks (4,5).

Corrosion of steel is an electrical as well as a chemical process. In its basic form, it is caused by stray current discharging from one area of the steel (the anode) and returning to another (the cathode) (see Figure 1). Corrosion, or oxidation, occurs at the anode. As corrosion continues, its byproduct, pack rust, accumulates at the anode while no harmful side effect is experienced at the current-receiving cathode. When all steel is placed in a current-receiving mode, discharge is stopped and corrosion ceases.

The concept of integrating the essential components of bridge-deck and pipeline cathodic systems to protect rapidly spalling pavements in Minnesota is being examined in a laboratory and field test program. There would be several advantages in using such a system on pavements if it proves to be effective at controlling corrosion:

1. All essential component parts would be located off the traveled roadway.
2. The cost of installing and operating such a system would be small in relation to the combined costs of maintenance or pavement replacement, to say nothing of the social impact caused by extensive traffic delay and disruption.
3. The effectiveness of the cathodic system can be determined over a short period of time.

Buried pipelines are protected by placing anodes in a conductive backfill material, usually near the in-place pipeline (3). A ground connection is attached to the pipeline and, when the system is energized, an electrical potential field is set up between the pipe and the anode. Current flows through the field to the pipe, which is the cathode of the system. When the pipe is in a current-receiving mode, stray current discharge cannot occur and corrosion is stopped (see Figure 2).

Reinforcing bars in concrete bridge decks have also been protected by using impressed current. This type of protection requires a somewhat more specialized system. To conduct the current, a coke-modified asphalt overlay and special anodes are placed on the deck surface. The special asphalt creates the electrical field needed to distribute current to protect the rebars (see Figure 3).

There is a possibility that cathodic protection of the steel in CRCP can be provided by integrating essential elements from the pipeline and bridge-deck systems (see Figure 4). The concept involves burying anodes in a trench or in postholes backfilled with conductive coke aggregate. The trench or postholes could be located along either the median or the right shoulder on a four-lane system and would run parallel to the pavement. A ground connection would be attached to the continuous reinforcing at several locations to

complete the circuit. When energized, the anodes would create a potential field between themselves and the steel within the CRCP. This field would conduct current to the rebars in a way similar to that in which the soil conducts current to the pipeline. It is believed that corrosion can be stopped when the pavement-reinforcing steel is in a current-receiving mode and that the forces necessary to generate the pressure that causes cracks and, eventually, potholes in the pavement would then be eliminated.

Design Criteria

Two electrical parameters are commonly used in designing a cathodic system: half-cell potentials and current density.

Potential

Perhaps the oldest and most frequently referenced criterion is the 0.85 V CSE polarized potential criterion used by the pipeline industry (3) (voltages in the negative according to the standard method). This criterion is based on the recognition that the most anodic potential of steel in soil is on the order of 0.80 V CSE. Protecting the structure requires establishing a slightly higher cathodic potential and having some allowance for a safety factor--thus, the 0.85 V CSE value. For protecting reinforced-concrete pipe and reinforced-concrete bridge decks, the additional consideration of preventing hydrogen over voltage and possible debonding of the steel from the concrete leads to imposing an upper limit of 1.10 V CSE on the acceptable polarized potential (6).

More recent research has shown that, although 0.85 V CSE may be realistic as a criterion for steel in soil, it is not necessarily accurate for steel in a concrete environment (7). Bridge-deck testing with the Cu/CuSO₄ half-cell shows that measured potentials in excess of 600 mV are rather uncommon (8). Work by Hausmann also supports the position that lower polarized potentials may protect steel in concrete (7).

Other potential criteria suggested for cathodic protection relate to the shift in potentials during the polarization process rather than a fixed range of 0.85-1.10 V. The National Association of Corrosion Engineers (NACE) suggests that for pipelines a potential shift of 300 mV is indicative of achieving cathodic protection. For steel embedded in concrete, however, it is loosely held that a lesser shift in potential is required. Kubit (9) has suggested that polarization and depolarization curves provide a very reliable basis for determining cathodic protection needs and that depolarization shifts of 100 mV or less may signify achievement of cathodic protection.

Current

Current-density requirements are essential to the proper design of any cathodic system. Current density is associated with protection in two ways: (a) It is a function of the polarized potential, and (b) it may be viewed as a fixed range, perhaps similar to the 0.85-1.10 V CSE criterion. When Stratfull (4) protected the first bridge deck, he found that the current-density requirements were 0.7 mA/ft² of steel surface. At that time, it was assumed that he was protecting the top mat of rebars only (he may have protected the bottom mat as well). Fromm (5) has reported that current densities to achieve cathodic protection range from 0.15 to 0.50 mA/ft² of bridge-deck surface in

Figure 2. Pipeline cathodic protection system.

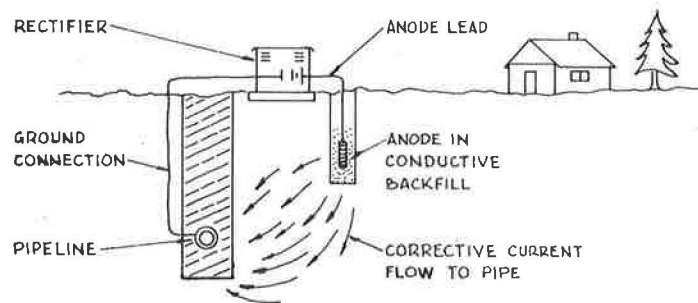


Figure 3. Bridge-deck cathodic protection system.

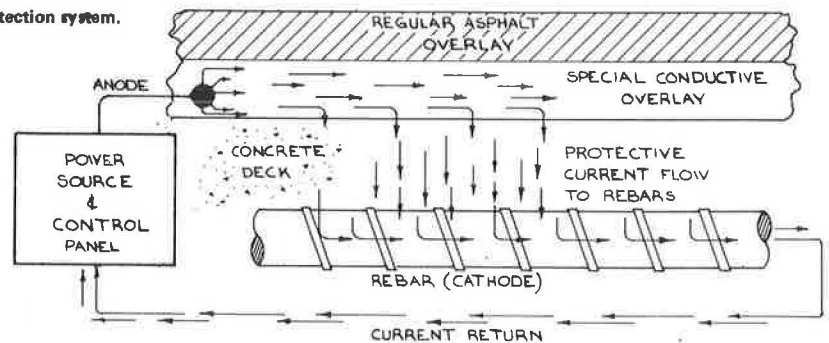
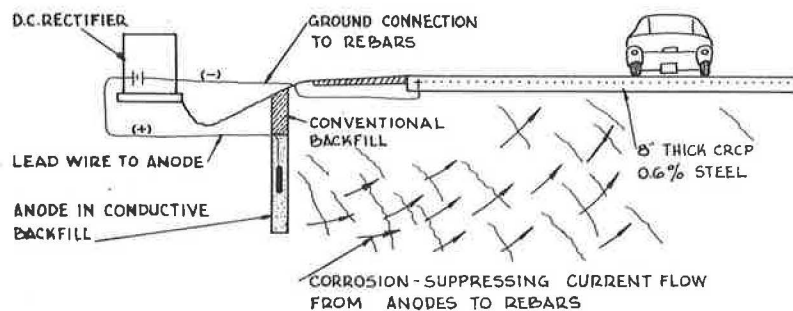


Figure 4. Cathodic protection installation for CRCP.



Canada. Vrable (10) notes that others have related current-density requirements to the condition of the concrete. Neither Fromm nor Vrable specify whether the current densities are associated with square feet of deck surface or the steel surface within a square foot of deck. It is also important whether protection is assumed to be applied to both mats of steel or only one, since there tends to be a difference. It was Vrable's assessment that current-density requirements normally do not exceed 1.0 mA/ft^2 of surface (10). I have assumed here that the surface-area designation in question is associated with the steel surface and not with the pavement surface.

System Design and Control

The first and major consideration in this effort, as with any experimental system, was designing it so that the probability of its successful operation would be high. A prerequisite to achieving effective cathodic protection is uniform current distribution. If distribution is nonuniform, hot spots (overprotection) will occur at some locations and cool spots (underprotection) will occur at others.

Several factors have an effect on the characteristics of current distribution; some can be controlled, others cannot. These factors are

1. Resistivities in the medium separating the anode and cathode,
2. The continuity of the steel in the structure being protected,
3. The uniformity of the applied potential, and
4. The proximity of the source (anode) to the protected structure (pavement).

The two most commonly used methods for applying impressed-current cathodic protection to pipelines is the distributed anode ground bed and the remote anode ground bed (3). Experience with these two methods seems to indicate that the distributed ground bed offers the most uniform applied potential and is the most responsive to control efforts.

In general, the design concept of the distributed anode ground bed involves placing an array of anodes along the structure requiring protection. The anodes are usually placed in a continuous trench backfilled with a conductive aggregate or in postholes as prepackaged assemblies, each of which contains the anode and an appropriate amount of conductive fill material. In both cases, the anode leads are connected to a main line from the controller. It is common practice to provide separate circuits or staggered anode wiring arrangements so that, if part of the system goes out, the domino effect is avoided and protection can be maintained. When the review of all available

Figure 5. Trench method of cathodic protection for CRCP.

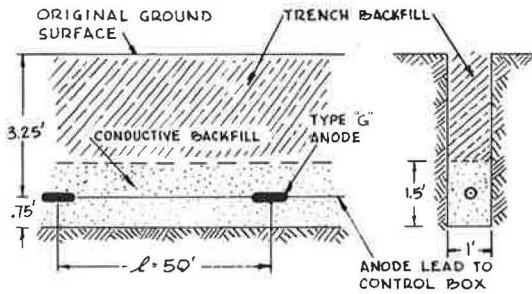
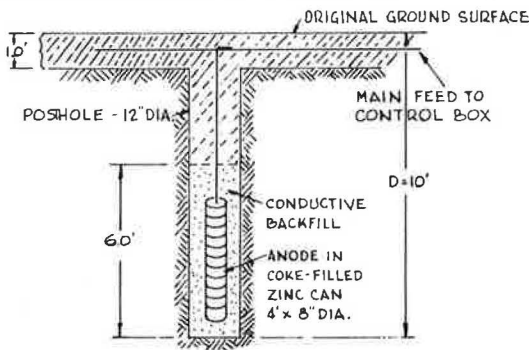


Figure 6. Posthole method of cathodic protection for CRCP.



information was complete and all options had been considered, a decision was reached to use the distributed anode ground bed for our trial system. It was felt that this approach would minimize interference problems, maximize system control capabilities, and provide the most uniform applied potential and current density, thus improving the chances for successful operation. It was also decided that a rectifier with constant current output would be used in the initial trial.

Anode Installation Schemes

The two schemes for anode installation, the continuous trench and the posthole approach, were both used in this trial. The test area was divided into two major sections, each 500 ft long. The first 500-ft section would be protected by using a continuous trench and the second 500-ft section by using the posthole method. Each test area was further subdivided into five zones, and each zone can be independently controlled at the rectifier.

Continuous Trench

One method of achieving uniform current distribution is to provide an array of anodes at some predetermined uniform interval that run parallel to the structure being protected. An anode's ability to discharge current to the surrounding soil is limited by (a) the resistivity of the soil, (b) the applied potential, and (c) the surface area of the anode.

Although the soil resistivity cannot be altered, the "effective" surface area of the anode can. Current discharge from an anode to the surrounding soil can be enhanced by placing a conductive (low-resistance) material around the anode. This can increase the effective anode surface area many times. The conductive material, in effect, becomes a secondary anode. By using a continuous trench, as shown in Figure 5, and spacing the anodes at uniform

intervals, a very efficient current-distribution network can be created. Current will radiate out from the conductive backfill along the entire length of the trench.

Posthole (Vertical Anode) System

The posthole approach also uses the concept of increased effective anode surface area. It does so, however, in a different way. Holes 10-12 in in diameter are drilled along the structure that is being protected. Conductive backfill is placed and tamped in the hole around an anode. This column of conductive material (usually 6-8 ft high) then becomes the effective anode (see Figure 6).

The anodes placed in both systems will be controlled from a rectifier located halfway along the system. The present thinking is to use a current-controlled rectifier with reference cells for monitoring installed in the CRCP slab. Each test section will be divided into five zones, each roughly 100 ft in length. The anodes supplying power to a zone will be regulated at the rectifier according to the potential in that zone.

Several bases for assessing system performance will be considered. The widespread and accepted method is polarization testing. Here, Cu/CuSO₄ or some other suitable reference electrode is used to measure potentials. The electrode is placed in close proximity to the steel being protected, and readings are taken. The current state of the art from both laboratory investigations and field experience supports the criterion of 0.85-1.10 V relative to the Cu/CuSO₄ electrode as being representative of protective polarization.

Other available, but less frequently used, criteria are the 100-mV shift for instantaneous-off potential measurements or a 300-mV shift between the system-off and total polarization status. The final technical criterion that will be considered is current-density requirements. There is considerable evidence that current densities on the order of 0.3-0.7 mA/ft² of steel are realistic values in achieving cathodic protection.

Laboratory Evaluation

As an aid in determining performance--or, more correctly, working parameters--some preliminary tests are being scheduled. A potentially controlled rectifier to power and regulate the system is also being reviewed but has not been tested. It is not known just how reliable this device will be in the field. Information on the operational characteristics of the rectifier controller is needed. To obtain this information, a laboratory simulation of the system is being conducted. The areas identified for study are (a) the resistivities in the various materials between the anode and the rebar, (b) the time required to achieve polarization, (c) the reliability of the zinc half-cell as a potential sensor, and (d) current densities associated with polarization.

As stated earlier, the field evaluation of this system will involve both internal and external monitoring. Internal monitoring will be restricted to observing and documenting applied potentials and current output to each of the anode circuits. Each circuit will consist of two anodes and two reference cells cast into the pavement slab. The reference cell in the passing lane, which is farthest from the anodes, will be used in setting the power output from the anodes, and the reference cell in the driving lane will be used to monitor applied potentials. In view of the fact that uncertainties still exist about reference-cell reliability, an

optional plan to monitor potentials with an internal carbon probe will also be examined.

It is probable that extensive use will be made of the Cu/CuSO₄ half-cell as a means of external evaluation. Horizontal potential gradients are anticipated at the structure from the trench system because of the trench's relatively shallow burial. There could well be a tendency for the steel in the pavement near the trench to polarize first and for that farther away to polarize later. This being the case, as polarization occurs, a back electromotive force would develop in the closer steel and redistribution of current would result. A less noticeable horizontal gradient is expected with the posthole method. Because the anodes are buried deeper, they "see" the structure from a better angle, and potentials applied to the structure should be more uniform. Cu/CuSO₄ surface testing will again be used to measure applied potentials (absolute) and to check for possible gradients.

It should be obvious by now that there are still many unknowns in relation to the functional aspects of this system. If the situation had permitted, a more comprehensive approach to evaluating cathodic protection for application to CRCP would have been pursued. No doubt this would have consisted of conducting first a laboratory simulation, then a limited field trial, and finally a full-scale experimental test installation. Such an effort would have required 24-36 months to complete. Our present approach is to use the fast-track method, which is not unlike the approach used at the advent of the bridge-deck-spalling repair programs, a technique called "research by crisis".

CONCLUSIONS

In the past three years, more than \$100 000/year has gone to patching or other ways of trying to maintain approximately 4 miles of four-lane Interstate highway that is now 10 years old. The distressed pavement is an 8-in slab reinforced with deformed wire mesh that was built by using the two-course construction technique. Since the corrosion

phenomenon is for the most part irreversible, cathodic protection is now being examined as one possible solution to serious and rapidly advancing pavement deterioration problems.

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Publication of this paper sponsored by Committee on Corrosion.

Study of Adhesive-Bonded Composite Concrete-Metal Deck Slabs

DANA J. MCKEE AND JOHN P. COOK

The results of a study conducted to determine the effectiveness of an epoxy resin as a shear connector in composite systems are presented. Composite concrete-metal deck slabs were constructed by using an epoxy resin to bond the concrete to the metal deck. Three composite specimens and three noncomposite control specimens were used in the test program. The concrete was plant mixed and trucked to the site by a local concrete supplier. No special additives were used in the concrete. All specimens were loaded to failure on a simply supported span of 3.66 m (12 ft). A four-point loading system was used. The loads were applied slowly, and impact loading was not considered. The noncomposite control specimens showed a fairly high percentage of partial composite action. Two of the three composite specimens failed by excessive deflection without reaching a definitive value of ultimate load. The adhesive-bonded composite specimens, based on serviceability criteria, carried more than twice the load carried by the noncomposite control specimens. The test results indicated that the epoxy bond performed well as a shear connector and allowed the composite concrete-metal deck

slabs to achieve full composite action. Additional studies are required to extend the results to both other composite systems and other types of loading.

There is a considerable attraction to be found in the use of adhesives as shear connectors for composite beams. Mechanical fasteners, while quite effective, furnish a horizontal shear connection only at a set of discrete points. There are also high local stress concentrations in the shear connectors and in the surrounding concrete.

On the other hand, the adhesive furnishes a continuous bonding plane at the point where the two dissimilar materials meet. Several references in the literature (1-5) show the feasibility of the

Figure 1. Specimen cross section.

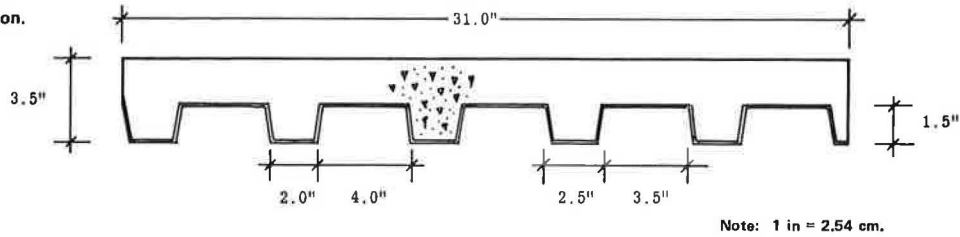


Figure 2. Location of strain gages.

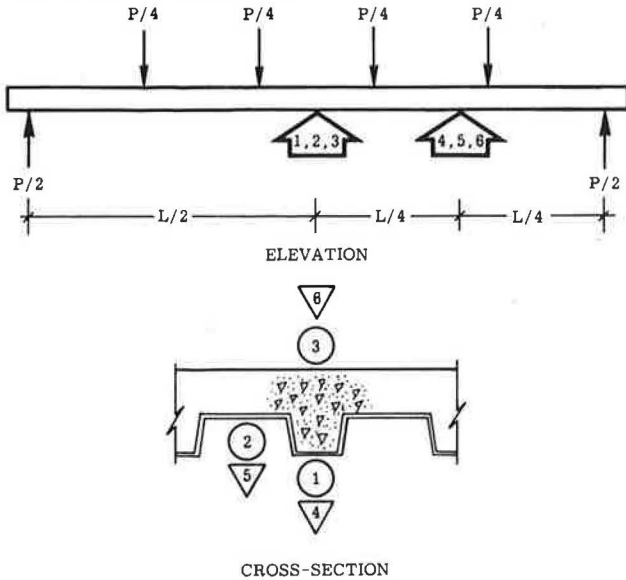
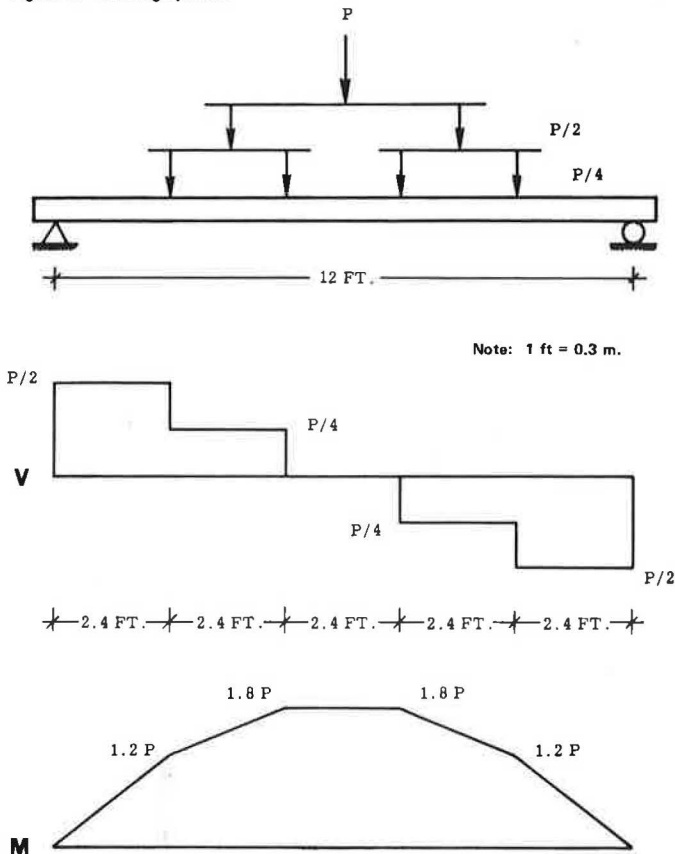


Table 1. Summary of testing program.

Test	Type of Specimen	Method of Failure	Load (kN)	
			Failure	Ultimate
1	Noncomposite control	Horizontal shear	11.1	23.1
2	Noncomposite control	Horizontal shear	11.6	26.7
3	Noncomposite control	Horizontal shear	10.7	23.1
4	Adhesive-bonded composite	Yield failure in metal deck, no loss of bond between slab and deck	26.7	Not reached
5	Adhesive-bonded composite	Horizontal shear	26.7	28.9
6	Adhesive-bonded composite	Yield failure in metal deck, no loss of bond between slab and deck	28.9	Not reached

Note: 1 kN = 224.8 lbf.

Figure 3. Loading system.



adhesive as a shear connector.

Epoxy bonding compounds are now widely accepted as a construction material in heavy construction, although their use in composite construction has been quite limited. Up to this time, adhesives have been used in composite bridge construction in connection with the use of externally bonded thin steel plates to strengthen existing reinforced concrete bridges. In lighter construction, such as residential and light commercial buildings, elastomeric adhesives have been used successfully in composite members. Concrete slab on metal deck would appear to be the ideal type of construction for adhesive bonding because of the large bond area available between the metal deck and the slab.

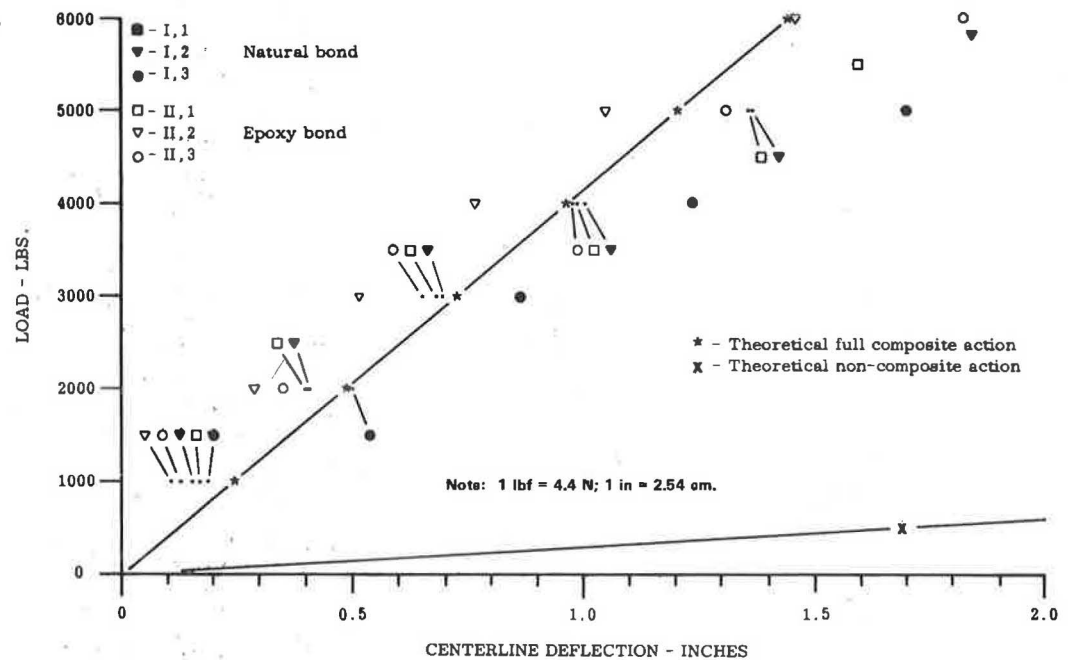
Steel deck composite beams are usually associated with building construction, where the steel decking can provide raceways for electrical conduit and telephone services. However, "stay-in-place" metal forms are gaining wider acceptance in bridge construction. If these metal forms can act to form a composite slab, considerable economy can be achieved in construction.

TESTING PROGRAM

The specimens used in the testing program were 3.96 m (13 ft) long and 0.76 m (2.5 ft) wide. The specimens were formed on 22-gage metal deck. Concrete was placed to a depth of 51 mm (2 in) above the high flutes of the deck. Figure 1 shows a typical cross section of a specimen. Non-air-entrained concrete with a strength of 34.5 MPa (5000 lbf/in²) was used in the project; it was plant mixed and trucked to the site by a local concrete supplier.

The metal deck sections were sandblasted clean. Then a brush coat of a water-compatible epoxy was applied to the deck. The epoxy bonding agent consisted of an epoxy resin and an epoxy hardener; the two were blended together, mixed with water, and applied to the surface of the metal deck at a rate of 4.9 m²/L (200 ft²/gal). Because the length

Figure 4. Load-deflection curves.



of time from mixing to initial set of the glue (pot life) was only 1.75 h, its application before the concrete was cast had to be carefully timed. The epoxy used met the requirements of ASTM C882-78. Finally, the concrete was cast on the freshly applied epoxy coat. The control specimens were constructed by casting the concrete directly on the cleaned metal deck sections.

The specimens were loaded as simply supported beams with a span of 3.66 m (12 ft). The loading was applied as four equal line-load concentrations, equally spaced along the span. This loading simulated a uniform load condition and provided a very small shear-free region at midspan.

Figure 2 shows how sets of strain gages were mounted at the quarter point and centerline of the 3.66-m span. At both locations, three gages were applied: two on the bottom of the metal deck on the upper and lower flutes and one directly above on the top of the slab. Dial gages to measure deflection were placed under the deck at midspan.

Loading was applied to the specimens in 890-N (200-lbf) increments at a slow, uniform rate of approximately 45-60 s. No impact loading was used. Figure 3 shows the loading system applied to one of the slabs.

The strength of the noncomposite control specimens was calculated based on the strength of the concrete and the metal deck acting independently. No natural bond was assumed between the slab and the deck. No interior reinforcing was used in any of the slabs.

TEST RESULTS

The types of specimens used, their methods of failure, and their failure loadings are summarized in Table 1.

The specimen in test 1 carried much more than its design load, which indicates the presence of enough natural bond to give partial composite action. No load-deflection curve is available for specimen 1 because the gage was damaged in the course of the test. The first cracking and local bond failure occurred at a very low load--3.3 kN (750 lbf). At 11.1 kN (2500 lbf), horizontal shear failure was

noted at both ends of the specimen. The specimen continued to carry load with increased cracking, up to ultimate load, when there was a sudden collapse of the specimen.

The specimen in test 2 also showed the presence of some natural bond and partial composite action. The first cracks that appeared in the specimen appeared at 6.7 kN (1500 lbf) and were horizontal cracks in the region of high flexure and zero shear. Local bond failure occurred near midspan at 11.6 kN (2600 lbf). Specimen 2 showed behavior similar to that of specimen 1 up to ultimate load.

In test 3, the first cracks appeared at 8.9 kN (2000 lbf). Horizontal shear cracks were noted at both ends at 10.7 kN (2400 lbf). The specimen continued to carry load, with increased cracking, up to ultimate load.

In test 4, the first crack in the specimen was a vertical crack near midspan, which formed at 22.2 kN (5000 lbf). As a result of continued deflection, there was local crushing adjacent to this crack. At 26.7 kN (6000 lbf), significant yield in the metal deck at midspan was noted. The specimen continued to deflect excessively, with only minor increase in load, up to 31.1 kN (7000 lbf). The bond remained intact between the metal deck and the concrete. At this point, the specimen had deflected 152 mm (6 in) and exceeded the stroke of the testing machine.

In test 5, the first vertical crack appeared at the centerline at 17.8 kN (4000 lbf). Under continued loading, the specimen showed horizontal shear failure at 26.7 kN (6000 lbf). At 28.9 kN (6500 lbf), the specimen would accept no more load.

In test 6, the first vertical crack appeared at one interior load point at 13.3 kN (3000 lbf). Another vertical crack near the interior load point appeared at 20 kN (4500 lbf). The specimen continued to carry load with no loss of bond. At the failure load, the deck yielded significantly but continued to carry load with no other distress until it exceeded the stroke of the testing machine.

Theoretically, it is impossible to attain 100 percent composite action because any amount of elasticity in the shear connection causes at least a negligible amount of slip. It is also theoretically impossible to obtain absolute noncomposite action

because of friction and the presence of some natural bond between the components of the member.

However, a member may be considered to have full composite action if one of the components of the member fails without rupturing the shear connection. This type of failure was achieved in two of the three composite specimens. The third composite specimen did show a horizontal shear failure but only after the load equaled the failure load of the other composite specimens.

INTERPRETATION OF RESULTS

The expected mode of failure in flexural members without adhesive bonding is a horizontal shear failure at the interface between the two dissimilar materials. All of the noncomposite specimens did fail in horizontal shear between the concrete slab and the metal deck. The dimensions of the slab unit were selected so that each specimen could resist a uniform loading of 5 kPa (100 lbf/ft²). No natural bond between the concrete and metal deck was assumed for these calculations. However, the unbonded members all carried load well in excess of their computed capacity, which indicated the presence of a relatively high percentage of partial composite action.

In this program, failure was considered as a lack of serviceability of the unit. In the case of the noncomposite control specimens, this lack of serviceability was defined by horizontal shear and separation of the decking from the slab. In two of the three composite specimens, no horizontal shear failure occurred and lack of serviceability was defined by excessive deflection. The load-deflection curves (see Figure 4) show that the adhesive-bonded composite specimens deflected less than the noncomposite control specimens by an average of 15 percent.

CONCLUSIONS AND RECOMMENDATIONS

As defined by serviceability criteria, the composite specimens carried more than twice the load of their unbonded counterparts. These results demonstrate that the adhesive-bonded concrete-metal deck member can achieve full composite action (3,6).

It is difficult to form a comparison based on ultimate load because two of the three composite specimens never actually reached an ultimate load but continued to deflect with increasing load. These tests were terminated because the deflection exceeded the stroke of the testing machine. The one

composite specimen that demonstrated horizontal shear failure showed only an 8 percent increase in ultimate load capacity compared with its unbonded counterparts.

Some aspects of adhesive bonding that might be recommended for study include the following:

1. Long-term creep effects on the effectiveness of the adhesive shear connection,
2. Determination of the amount of natural bond that exists between the slab and the metal deck,
3. The effects of cyclic and impact loading on the adhesive shear connection, and
4. Definition of the amount of permissible slip between the slab and the metal deck, which should be correlated with a study of the properties of various adhesives.

The results presented here are not expected to have any immediate impact on current practice, but they are one more contribution to the growing history of adhesive-bonded composite members.

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Publication of this paper sponsored by Committee on Adhesives, Bonding Agents, and Their Uses.

Evaluation of the Durability of Metal Drainage Pipe

RICHARD W. KINCHEN

Preliminary results are presented of a 10-year field study undertaken in Louisiana to determine the ability of aluminum and galvanized-steel culverts to resist corrosion in moderate, acidic, and low-electrical-resistivity environments. In 1973, 10 types of aluminum and galvanized-steel culverts were installed, generally as side drains, at 10 test locations. One pair of each type of culvert was installed at each site. Every two years, investigators are removing one designated culvert of each of the pairs and subjectively rating the condition of the metal and protective coating. Field samples of the culverts are evaluated in the laboratory, and the test culverts are installed again after each inspection.

The undisturbed mate of each pair remains buried for the duration of the study to analyze the impact of the periodic inspections. After 6 years of field exposure, the asbestos-bonded, bituminous-coated, galvanized-steel culvert is resisting corrosion quite well even in brackish water, an environment characterized by low electrical resistivity. Several aluminum and coated galvanized-steel culverts appear to be well suited for the normal range of acidic environments encountered in Louisiana. All of the test culverts appear to provide satisfactory service life in the test environment designated as moderate.

Figure 1. Location of test sites.

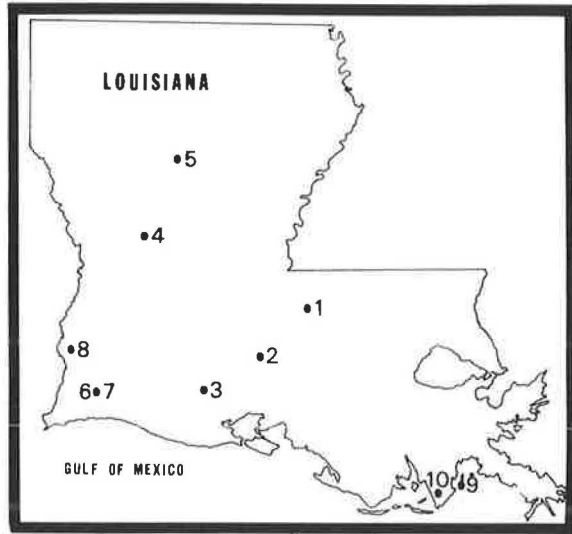


Table 1. Soil and effluent characteristics at test sites.

Site	Soil		Effluent		
	Type	Electrical Resistivity (Ω/cm^3)	pH	Electrical Resistivity (Ω/cm^3)	pH
1	Clay	1 038	6.4	11 750	6.4
2	Silty clay	812	7.5	4 280	7.3
3	Silty clay	1 268	6.9	5 460	7.0
4	Silty sand	11 323	5.3	20 667	5.6
5	Sand	3 479	6.6	2 333	6.6
6	Sandy clay	314	8.2	135	7.0
7	Sandy silt	456	8.1	133	7.1
8	Silty clay	3 437	5.5	16 200	6.8
9	Sand	971	8.4	338	7.7
10	Silty clay	254	8.2	121	7.3

The wetlands state of Louisiana receives approximately 152 cm (60 in) of rainfall each year. Road design engineers of the Louisiana Department of Transportation and Development (LADOTD) must assign cross slopes and texture to highways to rid them of this water. Hydraulics design engineers must then apply appurtenances such as drainage pipes to remove the water from highway rights-of-way. LADOTD hydraulics engineers can generally choose either reinforced concrete or corrugated metal pipe in their designs. Concrete pipe is very durable (1) and, under stable bedding conditions, can normally serve effectively for the life of a highway.

LADOTD recognizes that metal pipe has its place in the field of hydraulics and maintains an interest in innovations in metal pipe. Metal pipe is relatively lightweight, an advantage that gains significance as the size of the pipe increases. Because metal pipe is also relatively flexible, it deflects rather than breaking under heavy loads or when underlying support is lost. The major drawback of metal pipe is its tendency to corrode in the presence of moisture, oxygen, and salt. Additional information is needed on the rates at which aluminum or galvanized-steel pipe corrodes in the variety of environments found in Louisiana.

The Federal Highway Administration (FHWA) has identified the pH and electrical resistivity of the environment as major factors in metal culvert

corrosion (2). This concept is particularly applicable in Louisiana. Previous research by LADOTD has related corrosion rates for metal culvert with general geological soil areas, which in turn have been associated with pH and electrical resistivity levels (1).

Metal culvert corrosion in Louisiana can be analyzed in terms of three general environments. The geographic heart of the state can be termed a moderate environment. The pH values of soil and effluent in this area typically range from 6 to 8, and electrical resistivity values are usually greater than 2000 Ω/cm^3 . In the Gulf of Mexico region, pH values range from 6 to 9 and electrical resistivity values are less than 2000 Ω/cm^3 because of the inland intrusion of salt water. The hills of northwest and west-central Louisiana plus a portion of the eastern flatwoods represent a third type of environment in which electrical resistivity values exceed 2000 Ω/cm^3 but pH values approach 5 on the acid side of the hydrogen ion scale.

In 1972, the state found itself with a continuing need for drainage, a diverse set of environments, and a wide array of remedies offered by the metal culvert industry. The state responded with a major 10-year field study to determine the ability of available aluminum and galvanized-steel culverts to resist corrosion in moderate, acidic, and low-electrical-resistivity environments. A limited laboratory study parallels the field evaluations.

METHOD OF PROCEDURE

Site Selection

Research engineers initially selected 10 locations, spread across the state, as test sites (see Figure 1). These sites represented the spectrum of moderate, acidic, and low-electrical-resistivity (R) environments, as follows:

Minimum Soil Resistivity

Soil pH	<2000 Ω/cm^3	>2000 Ω/cm^3
5.0-6.0		Sites 4 and 8 (acidic)
6.1-7.0	Sites 1 and 3 (low R)	Site 5 (moderate)
7.1-8.0	Site 2 (low R)	
8.1-9.0	Sites 6, 7, 9, and 10 (low R)	

Table 1 gives the characteristics of the soil and effluent at the test sites. Sites 6 and 7 are ditch and canal installations, respectively, and are located on opposing sides of the highway at that location.

Materials Tested

Eleven varieties of coated and uncoated aluminum and galvanized-steel culverts were selected for evaluation. The corrugated test culverts were 1.2 m (4 ft) long and 46 cm (18 in) in diameter, except for the aluminum plate arch, which was approximately 1.4 m (4.5 ft) square.

The following types of culverts are commonly known:

1. Uncoated, 0.15-cm (16-gauge) galvanized-steel pipe (AASHTO M36);
2. A 0.15-cm galvanized-steel pipe, coated with asphalt of typical thicknesses of 0.14 cm (55 mils) on the interior and 0.12 cm (48 mils) on the exterior of the pipe (AASHTO M190);
3. A 0.19-cm (14-gauge) galvanized-steel pipe,

coated with asbestos-bonded asphalt of typical thicknesses of 0.14 cm (55 mils) on the interior and 0.16 cm (64 mils) on the exterior (LADOTD 1977 Standard Specifications for Roads and Bridges);

4. Uncoated, 0.15-cm clad aluminum alloy pipe (AASHTO M196);

5. A 0.15-cm clad aluminum alloy pipe, coated with asphalt of typical thickness of 0.13 cm (50 mils) on the interior and on the exterior (AASHTO M190, type A); and

6. A 0.27-cm (12-gauge) aluminum alloy structural plate arch (AASHTO M196).

The following five types of test culverts are new products:

1. A 0.15-cm galvanized-steel pipe with a 0.03-cm (12-mil), coal-tar-based laminate applied to the interior and a 0.008-mm (0.3-mil) modified-epoxy coating on the exterior (AASHTO M246);

2. A 0.15-cm galvanized-steel pipe with a 0.05-cm (20-mil), coal-tar-based laminate applied to the interior and a 0.008-mm (0.3-mil) modified-epoxy coating on the exterior (AASHTO M246);

3. A 0.15-cm galvanized-steel pipe with a 0.02-cm (10-mil) interior and 0.08-mm (3-mil) exterior polyethylene coating (AASHTO M246);

4. A 0.15-cm galvanized-steel pipe with a 0.03-cm (12-mil) interior and 0.01-cm (5-mil) exterior polyethylene coating (AASHTO M246); and

5. A 0.15-cm galvanized-steel pipe with a 0.02-cm (10-mil) interior and 0.08-mm (3-mil) exterior plasticized-vinyl-resin coating (AASHTO M246).

Field Installation

In 1973, LADOTD research and maintenance personnel installed 20 sections of culvert at each of the 10 selected locations. Two sections of each type of culvert were buried in all locations, one section to be removed periodically for evaluation and reinstallation and the other to remain undisturbed for the duration of the 10-year study.

A "Grade-All" was used to remove all grass and debris from the ditches at the test sites for approximately 61 m (200 ft) to facilitate the installation. Next, the top 0.6 m (2 ft) of in-place soil was removed, and the pipes were lowered into the ditch by hand and spaced approximately 1.8 m (6 ft) apart. The removed soil was then used to cover the individual pipe sections to provide a minimum cover of 0.3 m (1 ft). A similar installation procedure was used at the two water sites, where the drainage pipes were installed along the side of drainage canals parallel to state highways that run through the coastal marshes. The pipe sections were installed perpendicular to the roadway, half covered with soil and half extending out into the brackish water.

In 1975, research and maintenance personnel used the installation procedures described above to install one pair of an eleventh type of test pipe at each of the 10 sites. This was the galvanized-steel culvert with a plasticized-vinyl-resin polymer coating.

Field Inspection

In 1975, 1977, and 1979, a panel of research engineers and technicians inspected the culverts after two, four, and six years of field exposure, respectively. Maintenance personnel removed one designated pipe of each type at each site. The 1.2-m (4-ft) sections were then washed clean to remove as much of the soil as possible without removing the coatings. After the pipes were

cleaned, photographs were taken from several angles to document their condition. Then the inspection panel, which consisted of two highway research engineers and three highway research engineering technicians, visually rated the pipes and coatings by using an evaluation report form.

The rate of corrosion is being studied in terms of the following criteria:

<u>Rating</u>	<u>Category</u>	<u>Condition Under Visual Observation</u>
0-1.0	Excellent	No signs of deterioration
1.1-2.0	Good	Very slight signs of deterioration and pitting
2.1-3.0	Fair	Moderate signs of deterioration and pitting
3.1-4.0	Poor	Extreme signs of deterioration and pitting
4.1-5.0	Very poor	Signs of complete deterioration; pipe no longer useful as a drainage tool

Coatings were also rated on a scale from zero to five, and notes were made of blisters, delamination, and removal from the metal.

The pipes were then sampled for laboratory examination. The sampling consisted of cutting an 8-cm (3-in) band off the end of each section removed. To provide protection between evaluations, an asphalt film was brushed on the metal edges that were exposed during the cutting process. When the field evaluation was completed, the pipes were returned to the ditch, oriented in their original positions, and covered with in-place soil.

Laboratory Analyses of Soil, Water, and Unexposed Culverts

Soil and water samples were initially collected from each installation site on a semiannual basis. Since the results from the semiannual samples showed relatively little change in pH and resistivity, the investigators have changed to sampling annually. These samples have been tested for pH and minimum resistivity in accordance with LADOTD laboratory procedures, which require the use of a pH meter and a resistivity meter as the basis of measurement. The soil samples were identified by field classification techniques.

The culvert testing program initially dealt with determination of the physical characteristics of the various metals and their protective coatings as manufactured. The amount of zinc coating was determined by weight loss as the coating was dissolved in an acid solution (ASTM A90). The thicknesses of the bituminous, asbestos, and various organic coatings were measured with a micrometer. The composition of steel and aluminum used in the culverts was determined by X-ray fluorescence, a process that provides a quantitative analysis of each element present in the metal alloys. Thickness and composition data are given elsewhere (3).

The durability of the culvert materials as manufactured has been evaluated in the laboratory by the use of three methods: salt-fog exposure, Weather-Ometer exposure, and accelerated corrosion by induced voltage.

Salt-fog exposure (ASTM B117) and Weather-Ometer exposure (ASTM D609) are described in LADOTD procedure TR1011-74. In the salt-fog test, culvert samples are subjected to a fog with 18 percent salt concentration, at approximately 57°C (135°F), for four weeks. In the Weather-Ometer test, culvert samples are subjected to ultraviolet light; an intermittent, pressure-controlled water spray; a temperature of approximately 63°C (145°F); and

relative humidity of approximately 85 percent. Evaluations of the salt-fog and Weather-Ometer exposures are subjective and take the form of satisfactory or unsatisfactory ratings for the number of hours exposed. Initial test results are reported in the appendix of the report by Kinchen and others (3).

In the method of accelerated corrosion by induced voltage, technicians cut metal culvert samples that measure approximately 7.6x15.2 cm (3x6 in) and coat the exposed edges with epoxy. The samples are individually suspended in a series of wooden boxes that contain 0.1 percent salt solution. A galvanic cell is created by means of wires that interconnect the samples. An electrical potential difference of 12 V is induced across each sample by means of a battery charger. The samples are removed from the galvanic cell as signs of severe corrosion appear. The culverts are examined by X-ray to detect the presence of perforations in the metal. Radiographic analysis thus allows the samples to be checked for perforation without removal of the asphaltic or polymeric coatings. The results from this induced-voltage testing are not yet available.

It is hoped that a minimum relative life can be determined from these three laboratory test methods and, ultimately, that culvert life in the laboratory and in the field can be correlated.

RESULTS

General Evaluation

The basis of field performance is the ability of the culverts to withstand metallic corrosion. The culvert that provides the best performance after six years of field exposure is the asbestos-bonded, asphalt-coated, galvanized-steel pipe.

Panel ratings of the test culverts after two, four, and six years of field exposure are given in Table 2. Minor inconsistencies in time-successive ratings for a given culvert reflect the independence of the evaluations. Subsequent discussion will consider the performance of the test culverts in the three types of environments given in Table 2.

The aluminum-alloy and galvanized-steel culverts are experiencing the fastest corrosion rates at locations where the effluent and/or the soil exhibits low electrical resistivity. There are conditions of low electrical resistivity at four sites near the Gulf of Mexico (sites 6, 7, 9, and 10), where resistivity values for the effluent are 350 Ω/cm or less, which is truly corrosive.

Low pH values are having less dramatic but still notable corrosive effects on the galvanized-steel culverts and no significant effects on the aluminum-alloy culverts. The pH values (5.3-5.5) referred to are low for Louisiana, where such values are normally above 5.5. The test environment designated as moderate is inducing corrosion at the slowest rate for the culverts as a group. Resistivity values of soil and effluent are greater than 2000 Ω/cm , and pH values center around 6.6 at this site.

Evaluation of Culverts by Type

Galvanized-Steel Culvert

Table 2 and Figures 2-4 reflect the rates of corrosion of the galvanized-steel test culverts and the relative corrosive nature of the three types of environments under consideration. As Table 2 indicates, corrosion of this type of test culvert was well under way after two years at a number of sites that are characterized by low electrical

resistivity. An increase in the rate of corrosion can also be noted in Table 2 as the character of the environment changes from moderate to acidic and then to low electrical resistivity. Figure 2 shows the advanced level of corrosion in the galvanized-steel test culvert after six years in the low-resistivity (brackish-water) environment. Figure 3, a photograph taken after six years, shows that corrosion of this type of culvert is progressing at a slow but steady pace in an acidic environment. The soil-side corrosion shown in Figure 3 indicates that the nature of the soil as well as the nature of the effluent should be considered in specifying and designing metal culvert for durability. In Figure 4, the galvanized-steel test culvert is shown to be resisting corrosion quite well after six years in a moderate environment.

Coal-Tar-Based, Polymer-Coated, Galvanized-Steel Culvert

The panel's two-, four-, and six-year evaluation ratings of the coal-tar-based, polymer-coated, galvanized-steel test culverts are given in Table 2. Six-year ratings of the coatings per test site and per environment are given in Table 3.

Comparison of six-year panel ratings in Table 2 for the galvanized-steel test culverts with and without this particular polymeric coating reveals that the coating is providing a small to moderate measure of corrosion protection for the culverts in the acidic and low-resistivity environments. The data given in Table 2 also show that the resistance to corrosion offered by these polymer-coated culverts decreases significantly as electrical resistivity decreases. The panel assigned ratings of good at six years to the polymer-coated culverts at sites 1, 2, and 3 where soil resistivity is at the 1000- Ω/cm level. However, the panel noted greater deterioration of these culverts at sites 6, 7, 9, and 10, where effluent resistivity is 350 Ω/cm and less and thus gave these culverts ratings of fair to very poor. Minor corrosion of the rivets was noted at the acidic sites.

The panel noted that the 0.51-mm (20-mil) thick coal-tar-based polymer coating tends to separate from the metal in all of the various types of environments (Table 3). The 0.31-mm (12-mil) thick coating tends to so separate in an environment that exhibits extremely low resistivity. Polymeric coatings of both thicknesses were susceptible to the formation of pockets of air and moisture (i.e., blisters) in the low-resistivity environments.

Polyethylene-Coated, Galvanized-Steel Culvert

Comparison of panel ratings of galvanized-steel test culverts with and without the polyethylene coating (Table 2) indicates that this coating has generally provided substantial protection to the metal in the various test environments. The panel did note perforation of this type of culvert at site 6 after four years and at site 7 after six years. The effluent at these two sites is brackish. In addition, minor corrosion of the rivets and seams was common in the acidic and low-resistivity environments.

The outlook for continued protection is not so promising. This polymeric coating is blistering in all of the various types of test environments and separating from the metal in the moderate and low-resistivity environments (Table 3).

Table 2. Panel ratings of test culverts at two, four, and six years.

Type of Culvert	Years Exposed	Moderate Environment, Site 5	Acidic Environment		Low-Electrical-Resistivity Environment						
			Site 4	Site 8	Site 1	Site 2	Site 3	Site 6	Site 7	Site 9	Site 10
Uncoated galvanized steel	2	1.3	1.9	1.7	1.4	1.2	1.2	2.9	3.7	1.5	2.9
	4	2.0	2.0	2.4	1.4	1.2	2.2	2.8	5.0 ^a	4.0 ^a	5.0 ^a
	6	1.8	2.4	2.2	2.2	2.0	3.0	4.4	5.0	4.5	5.0
Asphalt-coated galvanized steel	2	1.2	1.2	1.2	1.8	1.4	1.2	1.4	2.1	1.3	2.0
	4	1.6	1.2	1.8	1.4	1.8	1.8	2.2	4.4 ^a	2.0	3.2 ^a
	6	1.4	1.4	1.6	1.8	2.0	2.0	2.0	4.6	3.2	4.5
Asbestos-bonded asphalt-coated galvanized steel	2	1.2	1.2	1.2	1.8	1.2	1.2	1.2	1.4	1.3	1.5
	4	1.2	1.0	1.2	2.0	1.6	1.2	1.2	1.4	1.2	1.5
	6	1.0	1.0	1.4	2.2	2.0	1.8	1.2	1.4	2.0	2.0
Clad aluminum alloy (round)	2	1.6	1.5	1.4	1.2	1.8	1.3	3.2	3.2	2.7	3.1
	4	2.0	1.4	2.0	1.0	1.8	1.6	2.2	2.6	3.0	2.5
	6	2.0	1.6	1.8	1.4	2.0	2.4	4.2 ^a	2.4	2.8	2.5
Asphalt-coated clad aluminum alloy (round)	2	1.2	1.2	1.2	1.8	1.4	1.2	1.4	2.1	1.3	2.0
	4	1.6	1.2	1.8	1.4	1.8	1.8	2.2	4.4	2.0	3.2
	6	1.6	1.4	2.0	1.6	2.0	2.0	2.6	2.0	2.5	2.5
Aluminum alloy structural plate	2	1.4	1.3	1.5	1.5	1.4	1.2	3.3	3.0	2.7	2.6
	4	1.8	1.0	1.8	1.0	1.2	1.2	2.8	2.8 ^a	3.2	3.2
	6	2.2	2.0	1.8	2.0	2.0	2.0	2.6	2.6	3.8	3.5 ^a
12-mil, coal-tar-based, polymer-coated galvanized steel	2	1.2	1.2	1.2	2.0	1.2	1.2	2.0	2.6	1.2	3.1
	4	1.8	2.0	2.2	1.0	1.4	1.4	2.6	4.0 ^a	2.5	3.5
	6	1.8	2.0	2.0	1.6	2.0	2.0	3.2	4.0	4.0	4.5 ^a
20-mil, coal-tar-based, polymer-coated galvanized steel	2	1.2	1.2	1.2	1.7	1.2	1.2	1.8	2.6	1.2	3.1
	4	1.8	1.6	2.0	1.2	2.0	1.8	2.4	4.0	2.5	3.5
	6	1.8	2.0	2.0	1.8	2.0	2.0	3.2	4.0	4.0	4.5 ^a
Polyethylene-coated galvanized steel, 10-mil interior, 3-mil exterior	2	1.0	1.5	1.2	1.3	1.2	1.0	1.5	1.4	1.7	1.5
	4	1.8	2.0	1.8	1.8	2.0	1.6	2.2	2.4	3.0	2.2
	6	1.2	2.0	1.8	1.8	2.2	2.0	2.4	3.2 ^a	2.2	2.0
Polyethylene-coated galvanized steel, 12-mil interior, 5-mil exterior	2	1.0	1.4	1.2	1.2	1.0	1.0	1.5	1.4	1.7	1.5
	4	2.0	2.0	1.4	2.0	1.8	1.8	4.4 ^a	2.2	2.2	2.0
	6	1.0	2.0	1.6	1.8	2.0	2.0	4.0	2.4	2.8	2.0
Vinyl-coated galvanized steel, 10-mil interior, 3-mil exterior	2	1.0	1.0	1.0	1.0	1.0	1.0	2.2	1.8	2.0	2.0
	4	1.0	1.2	1.0	1.0	1.2	1.2	1.6	2.0	1.8	1.8
	6			1.1				1.5			

Note: 1 mil = 0.025 mm.

^aCulvert perforation.

^bNo rating available; test culvert missing.

Figure 2. Condition of galvanized-steel culvert in low-electrical-resistivity environment (site 10) after six years.



Figure 3. Condition of galvanized-steel culvert in acidic environment (site 4) after six years.



Vinyl-Coated, Galvanized-Steel Culvert

Two- and four-year panel ratings for the vinyl-coated, galvanized-steel culvert are given in Table 2, and the four-year ratings for this coating are given in Table 3. These culverts were installed two years after the other culverts under discussion here. The polymeric coating has thus far protected most of these culverts from significant corrosion. However, at the four brackish-water sites, the coating is blistering and separating from the metal culvert (Table 3). Additional time is required to properly evaluate the durability of this type of polymer-coated culvert, especially in moderate and acidic environments.

Asphalt-Coated, Galvanized-Steel Culvert

Comparison of ratings in Table 2 for the galvanized-steel culvert with and without the bituminous coating reveals that the coating is generally providing a moderate amount of corrosion protection. However, this protection is inadequate for the test environments with low resistivity, and

Figure 4. Condition of galvanized-steel culvert in moderate environment (site 5) after six years.



at two brackish-water sites corrosion has rendered the culverts unfit as drainage tools.

The bituminous coatings being evaluated in this study are not adhering sufficiently to the steel culverts in any of the environments under consideration (Table 3). Research personnel noted a lack of adhesion in 1973 before the culverts were installed. The bituminous coating, softened under high summer temperatures, incurred minor scrapes during preinstallation transporting and handling. In subsequent years, the panel has also observed asphalt coating clinging to the soil after the culvert has been removed from the ditch line for inspection. For each bituminous-coated culvert so inspected, there is a mate that is being left undisturbed for the duration of the study. It will be interesting to observe the condition of these undisturbed pipes at the conclusion of the 10-year study.

Asbestos-Bonded, Asphalt-Coated, Galvanized-Steel Culvert

Minor corrosion has been noted on the rivets of the asbestos-bonded, asphalt-coated, galvanized-steel test culverts at three of the low-resistivity sites and on the wall of one of these culverts (site 10) where the asphalt coating has been removed. However, this type of culvert is performing the best of all of the types under evaluation. The panel assigned ratings of 1.0-2.2 to test culverts of this type (Table 2). It is significant that this type of culvert is withstanding corrosion so well at sites 6, 7, 9, and 10, where the effluent is brackish water with resistivity values of 350 Ω /cm and less.

Overall, the coating is deteriorating in direct proportion to the harshness of the environment, from moderate to acidic to low resistivity (Table 3). Coating ratings heavily reflect the extent to which the asphalt coating is missing from the inside and outside of the culvert.

Aluminum-Alloy Culvert

Panel ratings in Table 2 indicate that a general trend could be developing in which aluminum alloy experiences initial oxidation followed by a leveling off of this process. An obvious exception to this trend is the aluminum-alloy culvert at site 6, which has experienced extreme pitting and a perforation after six years.

The Federal Highway Administration has suggested that uncoated aluminum-alloy culverts be allowed in environments where pH values range from 4 to 9 and electrical resistivity values are greater than 500 Ω -cm (2). At test sites 4 and 8, pH values are

Table 3. Panel ratings of coatings at six years.

Coating	Moderate Environment, Site 5	Acidic Environment			Low-Electrical-Resistivity Environment							
		Site 4	Site 8	Avg	Site 1	Site 2	Site 3	Site 6	Site 7	Site 9	Site 10	Avg
12-mil coal-tar-based polymer coating	1.0	1.2	1.0	1.1	1.2	1.2	1.2	3.6	4.2	2.0	3.8	2.5
20-mil coal-tar-based polymer coating	3.0	2.2	1.6	1.9	1.2	2.8	3.2	3.8	4.8	4.2	2.5	3.2
Polyethylene coating, 10-mil interior, 3-mil exterior	1.8	2.6	2.8	2.7	1.0	1.8	2.6	4.0	4.2	4.0	^a	3.1
Polyethylene coating, 12-mil interior, 5-mil exterior	1.0	2.2	1.8	2.0	1.4	1.4	2.0	4.2	3.6	4.0	^a	2.9
Vinyl coating, 10-mil interior, 3-mil exterior ^b	1.0	1.4	1.0	1.2	1.0	1.2	1.4	3.6	4.0	3.5	4.0	2.7
Asbestos-bonded asphalt coating	1.6	1.6	3.2	2.4	3.8	3.6	2.8	1.6	1.8	3.2	3.2	2.9
Asphalt coating on galvanized steel	2.4	2.8	4.2	3.5	4.8	4.8	4.8	4.0	4.8	4.0	4.8	4.6
Asphalt coating on aluminum alloy	2.8	3.2	4.2	3.7	4.4	4.8	4.0	4.8	4.6	4.8	4.8	4.6

Note: 1 mil = 0.025 mm.

^aNo rating available; test culvert missing.

^bFour-year ratings instead of six-year (culvert was installed two years after the other culverts).

Figure 5. Six-year panel ratings of aluminum-alloy culvert versus electrical resistivity of environment.

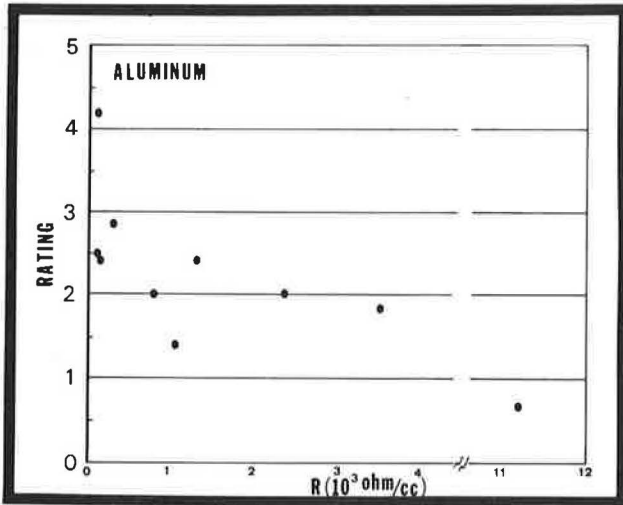
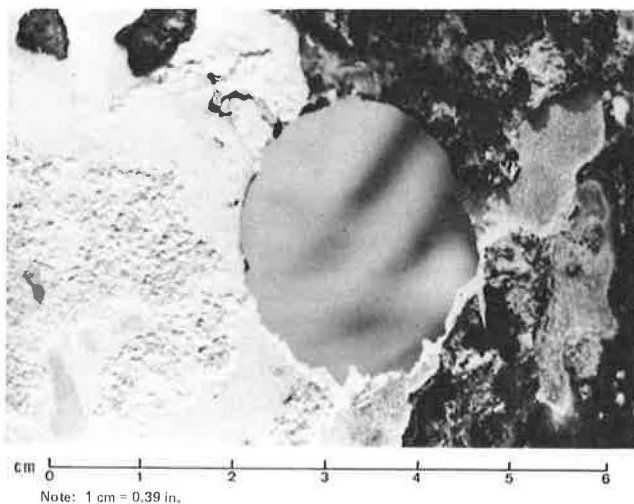


Figure 6. Pitting and thickness loss in aluminum-alloy plate arch after six years in low-resistivity environment (site 10).



near 5, and the aluminum-alloy culverts are performing well; 5 would thus appear to be a comfortably conservative minimum value.

If one assumes that electrical resistivity can be isolated as a dominant factor in the corrosion of aluminum alloy, one can develop an interesting plot of six-year panel ratings of this culvert from Table 2 versus critical resistivity values from Table 1 (see Figure 5). The rate of corrosion among the aluminum-alloy test culverts increases as the resistivity of the environment decreases. LADOTD policy states that the aluminum-alloy culvert will be excluded from consideration when the resistivity of the surrounding soil and water is less than 1500 Ω/cm . This minimum level is considered to be conservative but comfortable.

Asphalt-Coated, Aluminum-Alloy Culvert

Comparison of six-year panel ratings of bare versus asphalt-coated, aluminum-alloy test culverts (Table 2) reveals that the coating has improved the corrosion-related performance of the metal culvert

only slightly for two of the three environments. A similarity in the six-year ratings for bare versus asphalt-coated, aluminum-alloy culverts can be seen in a site-by-site review of Table 2.

The asphalt coating on the aluminum-alloy culverts has not proved to be sufficiently durable (Table 3). The durability that has been exhibited is greatest in the moderate environment and least in the low-resistivity environment.

Aluminum-Alloy-Plate-Arch Culvert

Panel ratings for the aluminum-alloy-plate-arch test culvert are also given in Table 2. These ratings have generally been on the borderline between good and fair except at sites 6, 7, 9, and 10 (the four brackish-water sites), where the panel has through the years noted severe pitting and thickness loss and perforation of two of the test plates.

The plate arch is not clad with another aluminum alloy as were the round aluminum-alloy pipes mentioned above. However, the aluminum-alloy plate and pipes did experience similar signs of deterioration--pitting and thickness loss. These signs are shown in Figure 6, a close-up field photograph of the plate arch at site 10.

CONCLUSIONS

1. The rate of corrosion of the galvanized-steel test culverts increases as the environment changes from moderate to acidic to low electrical resistivity. The rate of corrosion of the aluminum-alloy test culverts is similar for moderate and acidic environments and is faster in the low-electrical-resistivity environment.

2. In the moderate environment ($\text{pH} = 6-8$, $R > 2000 \Omega/\text{cm}^3$), all of the test culverts are performing satisfactorily. The asbestos-bonded, bituminous-coated, galvanized-steel culvert; the bituminous-coated, aluminum-alloy culvert; the bituminous-coated, galvanized-steel culvert; and the uncoated aluminum-alloy culverts seem well suited for the acidic environments ($\text{pH} = 5+$, $R > 2000 \Omega/\text{cm}^3$). The asbestos-bonded, bituminous-coated, galvanized-steel culvert stands out in its ability to resist corrosion in the low-electrical-resistivity environments ($\text{pH} = 6-9$, $R = 0-2000 \Omega/\text{cm}^3$).

3. The rate of deterioration of coatings generally increases as the characterization of the environment changes from moderate to acidic to low electrical resistivity. Bituminous coating is susceptible to removal during transport and installation (especially in hot weather) and to cracking as it ages. Polymeric coatings cannot be relied on to seal moisture and air from metal culverts. Factors such as delaminations at the culvert edge undermine this ability to seal, and blisters occur. The thick asbestos-bituminous coating is the most durable of the coatings being evaluated.

ACKNOWLEDGMENT

This paper was prepared as part of a research project that is being conducted by the Research and Development Section of LADOTD in cooperation with the Federal Highway Administration, U.S. Department of Transportation. Major support for the project has been provided by the Materials Laboratory and the district maintenance offices. Appreciation is extended to the manufacturers and suppliers who provided the test culverts.

The contents of this paper reflect my views, and I am responsible for the facts and the accuracy of

the data presented. The contents do not necessarily reflect the official views or policies of the state of Louisiana or the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

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Publication of this paper sponsored by Committee on Corrosion.

Motor-Vehicle Corrosion from Deicing Salt

FRANK O. WOOD

The benefits and costs of road deicing and its resultant effects on motor vehicles are discussed. A dollar figure cannot be put on benefits such as lives saved in quicker response time to medical and fire emergencies and lives saved by reduced traffic accidents. Even without these factors, the benefit/cost ratio justifies the use of deicing salts. A carefully designed and executed test reported by the American Public Works Association appears to indicate that it is not possible to inhibit salt. Chemicals other than salt and nonchemical methods that have been considered for ice control are briefly described. Other methods, in addition to alternate deicing compounds, include Verglimit, hydrophobic substances, electrically heated pavements, earth-heated pavements and geothermal heating, urethane foam and styrofoam insulation under bridge decks, air-jet plows, high-velocity sprays of deicing chemicals, infrared heating lamps, and underbody-bladed trucks. Salt, however, continues to be the least expensive deicer. Current techniques of corrosion protection in automobile manufacture are described.

A report by the Institute for Safety Analysis on the benefits and costs of road deicing, issued in November 1976, indicates the following annual economic benefits and costs (1):

Category	Annual Economic Benefits (\$000 000s)
Reduced wage losses	
Lateness to work	7 600
Work absenteeism	3 000
Reduced production losses	7 000
Reduced losses in goods shipment	600
Reduced fuel costs	200
Total	18 400

Category	Annual Costs of Road Salting (\$000 000s)	
	Institute for Safety Analysis Study (1)	Abt Associates Study (2)
Utilities	2	10
Vehicle corrosion	643	2000
Highway bridge decks	160	500
Trees and vegetation	0	50
Water supplies	10	150
Salt and application	200	200
Total	1015	2910

Among the benefits of road salting that cannot be given a dollar figure is reduced fuel consumption.

The Institute for Safety Analysis report (1) cites a saving of 0.37-1.2 billion gal/year. It has been calculated, based on a study by Claffey (3), that automobiles in the Milwaukee area would consume an additional 6 million gal of gasoline per year if the streets were not maintained during the wintertime. This calculation was made in response to a proposal by Milwaukee officials that the amount of salting be reduced by 50 percent to save 200 000 gal of gasoline used to operate salt-spreader trucks.

Other benefits of deicing include the lives saved by quicker response time to medical emergencies--e.g., heart attacks, burns, poisonings, home accidents, and work accidents--and fire alarms.

Lives saved because of a reduction in traffic accidents are also not given a dollar figure. Although no precise cost can be placed on this, the Institute for Safety Analysis report (1) quotes research that shows that about 26 percent of all noninstantaneous traffic fatalities have a potential for survival. If medical care for critically injured accident victims is delayed 1 h, the percentage of those likely to survive drops to about 6.25 percent and to less than 2.5 percent in 3 h.

The estimated annual cost of vehicle corrosion in the United States is \$643 million. The study by Abt Associates (2) estimates this cost to be \$2 billion/year.

The traditional cost of \$100/vehicle/year has been largely discounted. In fact, a task group report of the National Association of Corrosion Engineers (4) states the following:

It has been widely quoted that corrosion damage devalues the average automobile by about \$100.00 annually. The widespread use of this number is regrettable because it is not derived on the basis of an economic study. The \$100.00 annual loss was simply estimated and should not have been construed in any way as being quantitative or even semi-quantitative.

The Institute for Safety Analysis study estimates that the damage to bridge decks was \$160 million based on actual repair costs, whereas the Abt Associates study estimates \$500 million based on repair costs and estimated time loss during bridge repair in the summertime. However, the Abt Associates study estimates that there would be no economic losses as a result of leaving snow and ice on the highways in the wintertime. The Institute

Table 1. Chemicals that have been used in ice control.

Chemical	Eutectic Temperature ^a (°F)	Concentration at Eutectic Temperature ^a (%)	Cost per Ton ^b (\$)
Sodium chloride	-5.8	23.8	1
Potassium chloride	+13	19.5	4
Ammonium sulfate	-2.2	38.3	6.5
Calcium chloride	-67	29.8	6.7
Fertilizers			
Magnesium chloride	-28	21.6	14
Methanol	-144	100	14
Aluminum chloride	-65	25.3	18
Urea	+11	32.6	26.7
Urea/Ca formate (2:1)	-3	42.5	
Urea/Ca formate/formamide (1:1:1)	<-8	42	
Ethylene glycol	-59.8	60	53
Propylene glycol			
Tetrapotassium pyrophosphate	-38	60	79
Lithium chloride	-112	25.3	294
Calcium and magnesium acetates			

^aFrom Lange (6, p. 1191).

^bFrom Chemical Marketing Reporter, April 16, 1979.

for Safety Analysis study estimates that there would be no cost for trees and vegetation and a \$10 million cost for water supplies, whereas the Abt Associates study estimates a cost of \$50 million and \$150 million, respectively, for these items. These are small costs and will not be debated in this paper.

The main point is that the benefit/cost ratio of road salting is 6.3:1 if one uses the Abt Associates figures and 18.1:1 if one uses the Institute for Safety Analysis estimates. It would certainly appear, even with incomplete figures, that the benefits justify the cost.

TEST OF A SALT INHIBITOR

The only way in which the salt industry can assist in alleviating corrosion would appear to be through the development of an inhibitor or inhibitors that would be sprayed on the salt prior to shipping to inhibit the action of salt on metal. During the three winters from 1967 to 1970, the American Public Works Association sponsored a field test in Minneapolis to determine how much corrosion is caused by salt and how effective inhibitors are. This three-year test was completed and reported in September 1970 (5).

The testing procedure consisted of driving nine Ford Falcons on three routes in the Minneapolis area that are approximately the same length and have approximately the same vehicle traffic. Three of the Falcons were driven over a route maintained with abrasives, three were driven over a road maintained with salt, and three were driven over a road maintained with Carguard, an inhibited salt that has given excellent results in alternate immersion tests, in vehicle simulators where a tire was rotated in a 3 percent salt solution treated with the proportionate amount of Carguard, and in field tests in which automobiles were driven through the inhibited salt solution in the morning and afternoon to simulate travel to and from work. The test was evaluated by placing coupons under the front bumper and in the grill area of the cars and corrosion probes under the fenders, by inspection by trained Ford field observers, and finally by "can opening",--that is, cutting the cars completely apart to show corrosion on the interior of door panels, etc.

The conclusion was that the abrasives caused the

least amount of corrosion and that there was no difference between straight salt and inhibited salt. The explanation for the excellent results with Carguard in the earlier tests and the lack of inhibition in this final road test appeared to be that the inhibitor was applied at regular intervals in all preceding tests whereas, under actual test conditions, the salt was applied to the road only when it was needed for deicing. The winters during the test were fairly mild, and there were long periods of time when the inhibited deicing salt was not used. There appear to be very few situations in which the need for inhibited salt would be as great as the need for deicing. This would appear to be true of almost any inhibitor, and the salt industry is very discouraged at the prospect of approaching the inhibition of corrosion from deicing salts in this manner.

SUBSTITUTE FOR SALT

Many people inquire as to whether chemicals other than salt or nonchemical methods have been considered for ice control. The answer is yes. A large number of reports have been issued and continue to be issued on this subject. Not only have a number of alternate deicing compounds been considered but so have other methods for ice control, such as Verglimit, hydrophobic substances, electrically heated pavements, earth-heated pavements and geothermal heating, urethane foam and styrofoam insulation under bridge decks, air-jet plows, high-velocity sprays of deicing chemicals, infrared heating lamps, and underbody-bladed trucks. These will be discussed later in this paper.

ALTERNATE DEICING COMPOUNDS

Any chemical that dissolves in water will lower the freezing point. As more and more chemical is added to water, the freezing point will decrease to a certain level and then begin to increase. The lowest temperature at which the chemical will melt ice is known as the eutectic. A list of the chemicals that melt ice would be almost endless and completely meaningless.

Therefore, some criteria must be chosen for preparing a list of possible deicing chemicals. Table 1 gives a list of deicing chemicals that have been tried and provides for each three critical values: the eutectic temperature, the concentration (required to melt ice) at the eutectic temperature, and cost per ton. The chemicals are arranged according to increasing cost.

The deicing of highways is a dynamic rather than a static situation: The ice-melting chemical should therefore not be expected to melt ice at the eutectic temperature but at a temperature approximately 15°F above the eutectic. For example, sodium chloride would not be expected to remove ice from a highway at -6°F but probably would perform satisfactorily at 10°F.

Potassium chloride will not melt ice at as low a temperature as sodium chloride and is more expensive. It is a chloride and therefore should have the same corrosion characteristics. Ammonium sulfate is less corrosive to automobiles but cannot be used because it attacks concrete. Calcium chloride will work at lower temperatures than sodium chloride, but it is not sufficiently better to justify the higher cost and it has approximately the same corrosion characteristics as sodium chloride.

Mixtures of 75 percent sodium chloride and 25 percent calcium chloride and of 80 percent sodium chloride and 20 percent calcium chloride have been used. Currently, 8-10 gal of a 32 percent solution

is being added to a ton of salt either by applying it to an entire truckload or by spraying it on at the spinner. Fertilizers have been considered to minimize damage to grass and trees, but they cause population explosions of algae in rivers and lakes. Since magnesium chloride has a higher eutectic temperature than calcium chloride and approximately the same corrosion characteristics, if the economics do not justify calcium chloride, they certainly do not justify magnesium chloride.

Methanol and calcium and magnesium acetates have been suggested in a recent report (7). The Federal Transport Ministry of Germany has tried alcohol and has found that it poses a serious fire hazard and has a limited action time because of evaporation (8).

Aluminum chloride not only is corrosive but also undergoes a violent reaction with ice. Urea is not as corrosive as sodium chloride, but 1.5 times as much urea would be required (with a higher temperature limitation). This means that it would cost 26.7 times as much to use urea, to melt the same amount of ice, as it would cost to use sodium chloride. Urea is used on airport runways, although the roads to and from the airport are treated with salt or salt and calcium chloride. Urea has also been used selectively on bridge decks.

The urea/calcium-formate composition was suggested by the Research Institute of the Illinois Institute of Technology (9) as the result of an investigation performed under the auspices of the Highway Research Board. Although this paper was widely circulated, the cost of the mixture, which would certainly be greater than the cost of urea, has discouraged all highway departments from considering its use. As the urea/calcium-formate mixture was an attempt to propose a salt substitute, the urea/calcium-formate/formamide mixture was an attempt to propose a substitute for calcium chloride. It, too, proved to be too expensive for consideration by highway departments.

Ethylene glycol and propylene glycol have been used by various highway departments. The following comment has been made concerning their use (8): "Applications resulted in slippery conditions, a dark coloration, and a wet appearance of the surface of the bridge decks."

Tetrapotassium pyrophosphate (TKPP) is used for frost prevention on bridge decks in mildly freezing weather in California. Because of the high cost of TKPP, it is only considered for use on bridge decks in areas that are free of ice and snow and where the temperature rarely falls below 25°F. Although lithium chloride would be a very fast-reacting ice melter, it is still a chloride and probably as corrosive as any other chloride, and the price would certainly prohibit any consideration of its use on highways.

ALTERNATE METHODS OF DEICING

Verglimit

Verglimit is a bituminous concrete surface course in which calcium-chloride-coated fines replace the 5 or 6 percent stone fines that would ordinarily be a part of the regular bituminous concrete wearing course. As the surface wears under traffic, the calcium chloride capsules on the immediate surface dissolve and produce a calcium chloride solution between the road surface and the ice.

Conflicting results have been obtained. The fact still remains that Verglimit costs about \$900/ton. At this price, it may be used on some bridge decks, hills, curves, and intersections that have a history of winter accidents, but it will certainly not be used for general highway construction. If Verglimit

were to be produced in the United States, the cost might be reduced to \$200/ton.

Hydrophobic Substances

The Environmental Protection Agency (EPA), as far back as 1971, designated as one of their projects the development of a hydrophobic substance that would be an alternative to salt by preventing ice from bonding to the pavement. One substance that was developed was a modified traffic paint that contains a room-temperature-curing silicon rubber as a release agent. The other substance developed to be combined with this silicon rubber substance was a silicon resin waterproofing compound.

In 1975, inconclusive tests were conducted on the residential streets of Boulder, Colorado. During the winter of 1977, other inconclusive tests were run on a special track at Washington State. Since then, there have been only sporadic reports from Washington State with no particularly impressive results.

Electrically Heated Pavements

Probably no alternative to deicing chemicals has been experimented with as much as electrically heated pavements, and the results have been mixed. Electric heating is expensive and is only to be considered for sensitive areas such as bridge decks.

Earth-Heated Pavements and Geothermal Heating

An experimental pavement that uses earth heat with a heat pump and an earth storage system has been reported in a paper by Winters (10). It would appear from that paper and the paper by Zenewitz (8) that earth heating of pavements is not feasible. Geothermal heating, however, might be a real possibility. It has also been suggested that heated water be piped from nuclear reactors to heat pavements, but, after the Three Mile Island incident, it would not appear that anyone would care to experiment with nuclear reactors under varying heat loads.

Urethane Foam and Styrofoam Insulation

Both urethane and styrofoam insulation have been tried on the underside of bridge decks in order to maintain the deck at a temperature similar to that of the approach pavement. The state of Nebraska has reported that sometimes it works and other times it does not. The state of New Hampshire has reported trying it without success, and the state of Wisconsin has also reported that this technique failed to control ice formation.

Air-Jet Plows

The state of Connecticut reported that in the early 1970s it experimented with an air-jet snowplow that was able to deliver air at 1000 ft³/s. The plow clogged up, however, and the air tended to take the exit of least resistance--toward the rear, opposite to the desired direction.

High-Velocity Streams of Deicing Chemical

The state of Connecticut is cooperating in a Highway Planning and Research Program study of the development of a high-velocity spray of deicing brine. They have had some success with this equipment in breaking up snowpack and ice pack. This particular application might reduce the use of salt by approximately 20 percent.

Infrared Heat Lamps

The state of Colorado has been using infrared heat lamps to heat the underside of a bridge deck to prevent icing. The system, which uses 7 W of electricity per square foot of surface, was found to be inadequate because of excessive lag time and insufficient power. The experimenters felt that a 20-W/ft² system that costs \$8.90/ft² to install would give 20 years of satisfactory operation. The effect of insulation, which was included in the study, was found to depend on wind direction. Insulation helped to prevent icing when wind direction paralleled bridge direction and increased chances of icing when wind direction was perpendicular to the bridge direction.

Underbody-Bladed Trucks

The state of Michigan has been using underbody-bladed trucks to remove ice and snow. The blades are of tungsten carbide and work well except in severe cold. A drawback of the system is that the blade, if used on roads that have up-and-down rolls and resultant depressions, may not reach the ice at the low points. Although these plows have been used in Michigan for many years, their use in other states has never been widespread.

The paper by Zenewitz (8) demonstrates a good understanding of the status of alternate deicing compounds and methods in its summary based on telephone inquiries to state highway departments in the United States and provincial highway departments in Canada. That summary reads, in part, as follows:

Information obtained in the manner indicated above shows that all the states, provinces, and countries which were contacted are using salt for highway deicing. Most of the on-going research on deicing in the United States is generally concerned with the more efficient storage and use of salt for deicing....The general feeling among questioned highway personnel is that available chemical alternatives are too costly and less effective when compared with sodium and calcium chlorides. This feeling is even more pronounced with respect to the cost of nonchemical alternatives.

IMPROVEMENTS IN CORROSION PROTECTION OF AUTOMOBILES

Most of the complaints concerning salt and automobile corrosion occurred around the mid-1950s, when automobile designers were given complete free license and built-in sources of corrosion--such as eyebrows over headlights, high tail fins, and rocker panels that formed a trough completely accessible to moisture, dirt, and other corrodents--were permitted. This situation became so serious that the Society of Automotive Engineers (SAE) wrote a standard that set out principles to be included in design to prevent corrosion. Some of the features specified in SAE standard J447A, which was originally adopted in 1957 and later published in a handbook supplement in December 1964 (11), can be summarized as follows:

1. Keep the underbody surface dry. Avoid ledges, flanges, and pockets where dirt can accumulate and hold moisture.
2. Where appearance is of primary importance, use solder-filled, double off-set lap joints.
3. Make joints watertight.
4. Seal joints with a mastic-type compound and cover the entire faying surface and riveted surface.

5. Provide a protective flange for lap joints in line with wheel splash to prevent water and road contaminants from being driven into the area of faying surfaces.

6. To prevent galvanic corrosion, avoid wherever possible the use of dissimilar metals in contact with each other.

7. Use open construction wherever possible. Avoid box sections and closed areas.

8. Provide adequate drainage areas in doors and bodies that have movable windows.

9. When box sections or enclosed areas are used, provide sufficient openings for application and drainage of protective coatings.

10. Keep electrical connections free of moisture.

11. Design fuel tanks and other fuel-containing components to eliminate solder joints and the use of corrosion solder plugs.

Another milestone came in 1963-1964, when the rocker panel, which is the section below the door opening, started to be constructed of galvanized steel rather than steel. This virtually eliminated inside-outside corrosion and perforation at this point.

At that time, a few automobile plants were using whole-body dipping and electrostatic methods to attract paint to all metal surfaces. Since then, new materials such as Zincrometal have become available.

Zincrometal is a trade name for a proprietary zinc-organic coating developed by the Diamond Shamrock Corporation. It is applied to cold-rolled steel right after it comes from the rolling mill. The parts used by automobile plants are stamped out of galvanized steel, plain cold-rolled steel, and steel protected with Zincrometal. Steel coated with Zincrometal is used on the interior surfaces, some parts of the fenders, the inner part of the hood, and the deck lid. Ford Motor Company is using Zincrometal extensively, whereas General Motors uses it selectively.

Plain cold-rolled steel is used in the roofs, which are above the belt line and have very little corrosion. In the mid-1960s, the automobile industry claimed that galvanized steel could not be welded. Today, they have found ways to use Zincrometal-coated steel and galvanized steel in the same way that they used to use cold-rolled steel.

Reinforcement pieces and crossbars are often made more corrosion resistant by using prepainted steel. Frames are coated with a protective wax coating prior to assembly.

The car body is then given an electrophoretic coating by dipping it into a tank of primer paint. The coating is applied by giving the body one charge and the paint the opposite charge. The paint is attracted to the body and deposited on the sheet metal. The virtue of the system is that the paint is particularly attracted to any "holidays" that developed during the coating process so that the entire body is given a uniform coating. The only difference between the electrophoretic coating of the mid-1960s and today's type of coating is that the car body was formerly charged anodic and the paint cathodic but today the body is more often charged cathodic and the paint anodic.

Some Zincrometal parts are given a coating of zinc-rich primer after the stamping operation. The inner panel of the car door, for example, has deep draws in it along the lock facing and the hinge facing. The Zincrometal has a tendency to scrape off as the draw dye closes on the part, and zinc-rich primer is sprayed on these areas, which became unprotected as a result of damage to the coating in the forming operation.

Aluminized waxes are used to repair any damage done to the coating in subsequent operations. For example, after the door and inner door panel are formed and the coating is restored, these two parts are joined together by welding and hemming operations that may break through some of the protective coating applied in the manufacturing process. Aluminized waxes are used to coat all the seams in the bottom 6-8 in of the door. The aluminized wax runs down to fill up the joint. The inside of the door thus has three protective coatings in critical areas. Aluminized waxes are also used where the quarter panel joins the wheelhouse and along the back side of the quarter panel in the pockets on each side of the luggage compartment.

Heavy vinyl sealer is used to paint the insides of the rear wheelhouse. The lower exterior portions of the car--under the quarter panel, under the front fender, and along the rocker panel, for example, where stone damage would cause chipping and corrosion--are also sprayed with this paint. Strips of plastic are also used to line the inside of the fenders. These strips melt and flow into crevices when the car passes through the bake oven. Plastic face-shield panels are used for extensions of the hood, around the grill panel, and around the front headlight area.

Trim materials have also been improved. Research has been conducted to study the physical and chemical properties of multimetal coating systems. The concept of microporous or "cracked" chromium over nickel has led to less coating penetration by pitting. Other changes have added additional years to the durability of electroplated coatings.

The manner of fastening trim materials to the automobile has also been improved. For example, holes that were used for clip attachment of body side moldings and ornaments have been partially eliminated by using externally applied weld studs, thus minimizing the chance for moisture to enter through the hole. The contact of dissimilar metals, which causes galvanic corrosion, has been prevented by using adhesively bonded, plastic body-side moldings and aluminum-clad, stainless-steel moldings. This has prevented the unsightly corrosion of the body steel adjacent to the molding.

These improvements have resulted in Canadian automobile manufacturers announcing a 3-year warranty on cars sold in Canada. American Motors and Ford Motor Company are giving a 3-year corrosion warranty

on 1980 models. The objective of General Motors is 5 years without cosmetic exterior corrosion and 10 years without perforation.

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Publication of this paper sponsored by Committee on Corrosion.

Notice: The Transportation Research Board does not endorse products or manufacturers. Trade and manufacturers' names are included in this paper because they are considered essential to its object.

Durable Pavement-Marking Materials

HENRY J. GILLIS

Work done by the Minnesota Department of Transportation during the past 10 years to develop and evaluate a durable yet economical road-stripping material is described. The development of equipment capable of applying a two-component epoxy resin is discussed. Epoxy, polyester, and thermoplastic resins and their cost-effectiveness are evaluated. Field evaluation of the various materials consisted of visual observations, photographs, macrophotographs, and measurements of retroreflectivity. The available data suggest that epoxy can be placed on a high-volume bituminous or portland cement concrete roadway, at a thickness of 10 mils, and provide adequate delineation for 12 months or longer while remaining as economical as paint. The polyester material did not adhere well to portland cement concrete and the aggregate in the bituminous pavement. Thermoplastic was found to be generally

unacceptable because it is too susceptible to removal by traffic and snowplows when placed at the manufacturer's recommended minimum thickness of 30 mils and it does not bond adequately to portland cement concrete.

For many years traffic and maintenance engineers and industries around the world have worked to improve conditions for the motorist. There is an ongoing search for new delineation materials and devices. The Manual on Uniform Traffic Control Devices (MUTCD) (1) states, "Traffic lane markings have a

definite and important function to perform in a proper scheme of traffic control...they have the advantage...of conveying warnings or information to the other driver without diverting his attention from the roadway." At higher speeds, drivers must assign priorities to what they will see and react to. These priorities are (a) positional information, (b) situational information, and (c) navigational information.

If the first priority, position in the traffic lane, requires all of the driver's time, other significant forms of information will pass by unnoticed. For this reason, a well-defined roadway delineation system is important under all conditions. In most states, the restriping or replacement of paint stripes is a continuous process restricted only by weather conditions.

In Minnesota, the conventional paints currently in use for roadway delineation do not have the durability required for year-around life on roadways with a high average daily traffic (ADT) volume. Based on past research in Minnesota, epoxy resin shows promise as a durable striping material. Although the initial cost for epoxy placed is much higher than that for a painted line (three to six times), an extended service life can result in a reasonable comparison of costs over a period of time.

Epoxy was first placed in Minnesota in 1969 on a trial basis by using the extrusion method. All work done since 1970 has been done by applying a thin-film, high-pressure spray. Three formulas were selected from the initial test areas. These three formulas, which were placed on I-94 in St. Paul in 1973, were reduced to one formula based on cure time, color retention, and durability, and 182 km (113 miles) of this material was placed on the metropolitan Interstate system in 1975. In 1976, bead treatment and the capability of the epoxy to bond to itself under field conditions were studied. The results of this work led to the use of epoxy as one of the materials applied on the lower-volume, non-Interstate trunk highway system in 1977 and 1978 under a program authorized in Section 205 of the Federal-Aid Highway Act of 1973.

STUDY OBJECTIVE AND SCOPE

The objectives of this study were to evaluate three resins--epoxy, polyester, and thermoplastic--as pavement-marking materials and to develop and evaluate a spray-on epoxy/glass-bead system that would be appropriate for both bituminous and concrete pavements. This included developing automated equipment capable of applying the materials on a production basis.

In 1975 and 1976, approximately 225 km (140 miles) of epoxy skip stripe was placed on high-volume roads in the Minneapolis-St. Paul area for evaluation. Some variations were made in film thickness and bead type so that the effects of these factors on performance could be determined. Applications were also made over a range of pavement surface temperatures. About 64.4 km (40 miles) of polyester skip line was placed in 1976 as a comparison with the performance of the epoxy. In 1977 and 1978, approximately 457 000 m (1.2 million ft) of thermoplastic and 244 000 m (800 000 ft) of epoxy were placed statewide.

BACKGROUND

To understand the work done in this study, it is essential to proceed chronologically and to begin with the initial reasons for the use of durable pavement-marking materials in Minnesota. Figure 1 shows the typical condition of high-volume roadways

in Minnesota in late December when conventional traffic paint is used: Very little, if any, delineation is visible. The inability of any material to remain throughout a winter is the main reason the Minnesota Department of Transportation (DOT) became interested in developing a more durable pavement-marking material.

In 1969 and 1970, the Minnesota DOT placed a thick (62.5-mil) epoxy resin by using the extrusion method. This proved to be an extremely durable installation but had several drawbacks: slow placement, long cure time (60 min), and poor retroreflective qualities. The poor retroreflective qualities [less than 20 as measured by the electronic retroreflectivity measuring apparatus (ERMA)] were caused by the long cure time, which allowed the glass beads to sink below the surface.

Once it was demonstrated that epoxies could be formulated to adhere to pavement surfaces, even without primers, subsequent work was oriented toward cost reduction, cure time, and retroreflectivity for night visibility. Film thicknesses of 12-20 mils were used, since this gives the typical drop-on glass beads ample projection above the material surface so as to provide retroreflectivity, resist snowplow damage, and act as an integral part of the pavement.

Since the epoxy components are more viscous than conventional paints at ambient temperatures, a special heating system was necessary. In addition, because the two components react chemically, they must be proportioned extremely accurately and mixed thoroughly. Special equipment had to be developed or existing equipment modified to spray the material.

In 1971, a trial consisting of 25 stripes of a sprayable two-component epoxy resin was placed on a portland cement concrete (PCC) pavement with an ADT of 26 000. There was no difference in adhesion between cleaned and uncleaned surfaces; it was excellent for both.

In 1972, to accelerate the evaluation of the more durable pavement-marking materials, transverse-stripe studies on both bituminous and PCC surfaces were initiated. The center 0.6 m (2 ft) of each wheel path was selected as the evaluation section. Materials were placed in a typical film thickness of 20 mils, and all received an application of drop-on glass beads. The control materials included the initial spray epoxy formula (5143) and Minnesota DOT specification 20-s-dry and chlorinated rubber-based traffic paints.

Experimental materials placed on the bituminous surface included two new epoxy formulas, 5144 and HA-1, and a polyurethane-based material. All three were durable, but the cure times for the 5144 epoxy and the polyurethane were slow compared with those for the original 5143 and HA-1 formulas.

A near-duplicate test was established on a PCC surface that had an ADT similar to that of the bituminous section. In addition to the materials placed on the bituminous surface, this section also included the original spray epoxy (5143), with additional pigment, now designated formula 5143-1; a slow-cure (30-min) epoxy, 5144; and the same material with a different catalyst, 5144-A. After field evaluation, the 5143-1 epoxy and the two new formulas, 5144 with the slow (30-min) cure time and HA-1 (10-min cure time), were selected for further evaluation. Work also continued on equipment design and fabrication.

In 1973, these three materials were placed on a 1.6-km (1-mile) section of I-94 in St. Paul. At this location, there are eight lanes on which the ADT approaches 100 000. The painted lines placed in October were completely obliterated by wear, whereas the epoxy was still 95-100 percent intact after a

Figure 1. Typical midwinter delineation conditions on high-ADT roadway in Minnesota.



Figure 2. Condition of best-formulation epoxy (HA-1) after three years (left stripe).



winter. Figure 2 shows the HA-1 material in 1979 after six years.

Although some chipping occurred in each individual stripe, this did not reduce delineation because, from a typical sight distance [45.7 m (150 ft)], a stripe can be 20-30 percent removed by chipping and still appear to be a full or complete stripe.

As a result of the successful test application on I-94 in 1973, the formula finally selected was HA-1, the fastest-curing of the three materials placed. This selection was based on cure time, application properties, adhesion to the surface, durability, and color retention.

In 1974, a maintenance contract was awarded to H.B. Fuller Company to furnish 72 L (19 gal) of HA-1 material, application equipment, and technical experts to assist Minnesota DOT personnel in the application of 1828.8 m (6000 ft) of skip stripe on a high-ADT Interstate roadway in the Minneapolis-St. Paul metropolitan area. In early June of 1975, this contract was completed. Although there were minor problems, the result looked very satisfactory. But within four days, a severe (80-90 percent) loss of material was noticed. It was established that the components had been heated too high--at 87.8°-93.3°C (190°-200°F). This advanced the polymerization too fast in the system before spraying, which reduced

the wetting action of the epoxy system and the adhesive action of the material. In July 1975, the material was replaced at the proper temperatures--part A (pigmented) at 79.4°C (175°F) and part B (catalyst) at 73.8°C (165°F)--and it appeared to bond well.

In all previous work, there had been no significant difference in adhesion between sandblast-cleaned and uncleaned pavement surfaces. No cleaning was required on this contract.

In 1975, the Minnesota DOT applied for federal funds and, upon approval by the Federal Highway Administration (FHWA), a contract was let for the purchase of 3400 L (900 gal) of epoxy and polyester resins, including the application equipment and technical experts. This contract was completed in 1976.

1975-1976 PHASE: APPLICATION TO HIGH-ADT URBAN ROADWAY

Equipment

Epoxy System

The equipment used in 1973 to place the epoxy material was a system capable of heating, metering, mixing, and spraying the two-component epoxy. However, it was a basic machine and was manually operated. This severely limited the capabilities of the application system. The purpose in the 1975-1976 phase of development was to design and fabricate a fully production-capable machine to be used and operated in the same way as a typical highway striper.

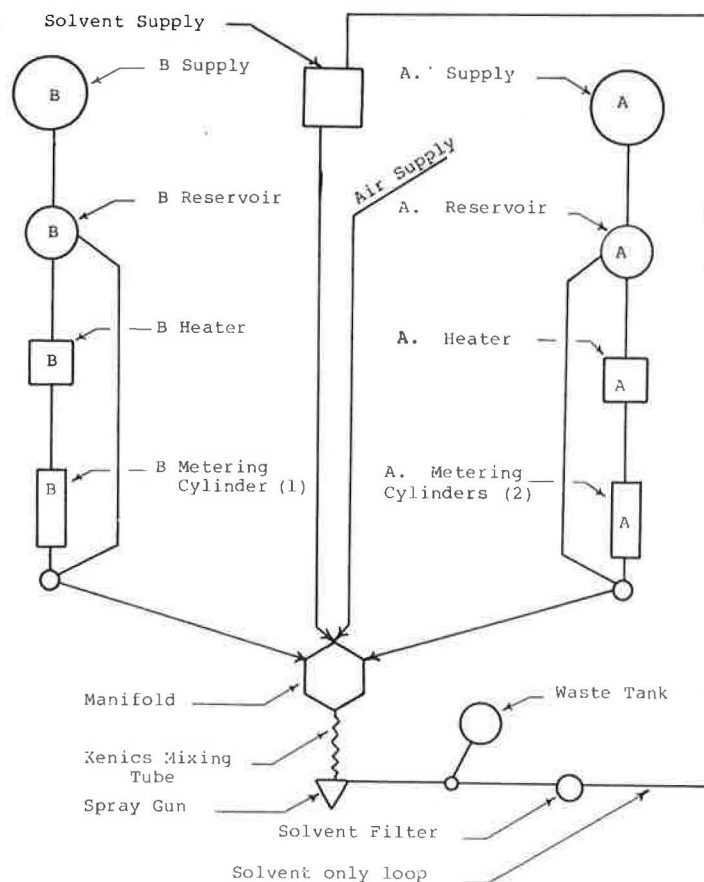
Figure 3 shows the schematic for the plumbing of the epoxy spray. The flow in both the A and B sections of the apparatus is from the reservoir (which is kept supplied as needed from the main supply tanks) to the heaters. Each component is heated to reduce viscosity, facilitate spraying, and reduce cure time. From the heaters, each component is introduced to the metering pump. The metering pump consists of a Graco Bulldog hydraulic power head with a 23:1 ratio of fluid to oil, simultaneously operating three double-acting cylinders of equal capacity. The cylinders are calibrated to deliver exactly two parts A and one part B to the manifold when the system is in a spray mode. When it is out of a spray mode, parts A and B are recirculated to their respective reservoirs.

When the system is in the spray mode, parts A and B meet at the manifold and proceed through the kenics mixer where complete mixing occurs. After the kenics mixer, the mixed components proceed to the spray gun for application. Each time the equipment goes from a spray mode to a nonspray mode, the system from the manifold through the spray gun must be cleaned with solvent. In nonspray modes, parts A and B are recirculated from the metering section back to the reservoirs, thus maintaining proper temperature in each component.

The solvent is introduced under pressure at the manifold, flushing the system from where parts A and B meet and down through the kenics mixer and spray gun. The gun is momentarily opened to flush the tip. With the gun nozzles closed, the solvent passes through the manifold, kenics mixer, and spray-gun body and is dumped in the waste tank. To purge the system of the solvent, the same procedure is followed and air is used.

The system must be flushed for any delay, for 30 s or more, to prevent the mixed components from curing in the mixing system. During long delays, prior to shutdown and before starting, the system is put in the nonspray mode and solvent is circulated

Figure 3. Plumbing of epoxy spray.



through the mixing system, the gun body, and a filter. This is an important part of the system because laminar flow in the system deposits a thin film of cured epoxy that is loosened by the solvent flush and plugs the spray gun. By recirculation through the filter, the system is kept clean.

A Graco model 205-163 material spray gun was modified by placing a Graco 205-163 gun at right angles to facilitate the flushing and cleaning operation.

In the course of development, many critical points were reached that required the development or modification of available equipment to achieve a production-capable automatic application system. The end result was a control system that is basically composed of electrically controlled air or hydraulic valves and a spray gun.

A final design is being prepared, and a system has been constructed by a major striper manufacturer. Application techniques are discussed later in this paper.

Initially, standard paint guns (Binks 21 and 33), modified to handle glass beads, were used. In certain areas where traffic volume is heavy, glass beads were poured on the epoxy stripes in an attempt to eliminate tracking of the epoxy. This proved to work so well that a free-fall system capable of flooding the epoxy stripes with glass beads at a rate of 2.4-2.9 kg/L (20-25 lb/gal) was developed.

The initial placement and several subsequent placements of epoxy were done by using sandblasting to clean the pavement surface. Because no apparent difference showed between the sandblast-cleaned surface and an uncleaned surface, it was decided that no preliminary cleaning was needed. The initial 1975 work on I-94 in St. Paul was done on a

56-km/h (35-mile/h) curve where there is regularly heavy weaving in traffic.

Laboratory tests performed on samples from these initial installation failures showed definite signs of contamination caused by the weaving traffic; oil, ethelene glycol, and rubber were very apparent. This pointed out the need for an economical system of cleaning the pavement to increase the adhesion of the epoxy to the pavement.

In 1975, it began to rain after striping had started across a long bridge. It was later noticed that the wet surface had not adversely affected the adhesion and, in fact, may have enhanced it. A decision was then made to evaluate cleaning with hot water [60°C (140°F)] at a pressure of 17.5 MPa (2500 lbf/in²) and using an air-dry system.

Polyester System

In 1976, a two-component polyester-resin pavement-marking material was placed under the FHWA contract for comparison with the epoxy. This was done under a contract with the Clark Company of Lake City, Michigan. The application system is simpler in design than the epoxy system because the mixing requirements of polyester resins are much less critical.

Optimum Bead Type

A main objective of this study was to determine the proper bead gradation and treatment. In this portion of the study, three types of glass beads were used to determine the best bead to use with epoxy resin.

Minnesota DOT Specification Beads

Minnesota DOT specification beads were used exclusively in the 1975 phase of the study, and there was a complete loss of retroreflectivity. The loss was caused by the powdered flow treatment working as a bond breaker. The epoxy did not have sufficient time to penetrate the powder and create a bond to the glass beads. Figure 4 shows the data collected on four sections where these beads were used. The figure shows that an epoxy stripe placed in October can still be functional as a daylight traffic delineator after 18 months. Except for the initial tests in the fall of 1976, stripes with this bead type were consistently lower and reached the minimum acceptable reflectivity level within nine months after placement.

Floating Glass Beads

The bisymmetric, or floating, type of bead was used based on recommendations from the striping industry and from other states on paints and thermoplastics. Work done in Minnesota had shown that the use of the floating bead improved retroreflective qualities substantially; in fact, it improved the retroreflective qualities of yellow paint above that of white paint with the standard Minnesota DOT specification beads. A total of 10 test sections were placed.

Figure 4 shows that the floating bead initially has about the same reflectivity as the Minnesota DOT bead but surpasses it from that point on. On the average, stripes with the floating bead reached the minimum acceptable level of reflectivity about 10.5 months after placement.

Chemically Moisture-Proofed Beads

The bead treatment and gradation used in the initial trial of the sprayable epoxy (in 1971), which involved chemical moisture proofing, still had acceptable reflectivity after three winters on a roadway with an ADT of 26 000. Because this type of treatment had been so successful in 1971, it was felt that it could be the answer to the loss-of-bead problem encountered in 1975. The data in Figure 4 show this bead to be substantially better initially and also throughout the 18-month test period. It also resulted in stripes reaching the minimum acceptable reflectivity level much later. This is the bead treatment and gradation recommended and used in the epoxy-resin pavement-marking system.

Observed Effectiveness of Epoxy

Observation has shown that the epoxy is an adequate delineator in both day and night conditions after 24 months, providing nighttime delineation under lighted roadway conditions. Observation has also shown that overhead lighting washes out the vehicle-headlight/glass-bead reflective system. Under wet-night conditions, the shiny enamel type of surface on the epoxy serves as a mirror and reflects the overhead lighting and the taillights of the vehicle ahead, which serves to amplify the delineation system for the vehicle operator.

Typically, conventional traffic paints are completely worn off each winter and therefore have no residual delineation capabilities unless they are renewed as shown in Figure 5. Figure 5 shows a plot of retroreflective qualities versus time for a conventional traffic paint placed six times and epoxy placed once in a two-year period on the I-94 test sections. There are periods in the winter season of each year when the stripes are worn off and should be painted but cannot be because of

weather conditions. This would increase the number of paintings to eight in comparison with one application of epoxy.

From Figure 5, it can be determined that, for the conditions described,

1. Epoxy has better reflectivity than conventional traffic paint 60.6 percent of the time,
2. Paint has better reflectivity than epoxy 32 percent of the time,
3. The reflectivity of paint is below the minimum acceptable level 52.7 percent of the time, and
4. No form of delineation exists 18.2 percent of the time when conventional traffic paints are used.

Even without retroreflective qualities, the epoxy is still serving as a traffic delineator after 24 months and is serving as a form of delineation 100 percent of the time under lighted roadway conditions. In most cases, epoxy is superior to paint.

Application Techniques

Application techniques for an epoxy-resin pavement-marking material are very similar to those for conventional traffic paint and other sprayable pavement-marking materials. The equipment is automated in the same way as standard paint striping equipment but differs in that exact control must be maintained over component mixing ratio, temperature, and glass-bead type, treatment, and application.

Component Mixing Ratio

The component ratio of 2:1 (two parts A to one part B) must be controlled within 3 percent or the cure time, adhesion qualities, bead retention capabilities, and durability of the placed epoxy-resin stripe will be adversely affected.

Temperature

The individual components are heated separately before mixing. The temperature at the time of mixing and spraying is critical for several reasons:

1. Part B, or component B as it is properly called, contains the catalyst and is sensitive to heat. If heated above 76.7°C (170°F) repeatedly or held too long at a high temperature, it will crystallize.
2. If the components are heated to too high a temperature before mixing, they will begin to polymerize too soon and reach the gel point (partial cure) too quickly to allow the material to properly wet the pavement surface and achieve a good bond. Durability will thus be affected.
3. Glass-bead retention will also be adversely affected by the same lack of wetting action as described in item 2 above.
4. Temperatures must therefore be limited to 79.4°C (175°F) maximum for component A and 73.9°C (165°F) maximum for component B.

Glass-Bead Application

Several attempts were made to place the drop-on glass beads. A pressure system was tried but, even with three bead guns, the "instant no-track" condition required to eliminate the setting and retrieving of traffic cones could not be reached. The free-fall, drop-on system of bead application developed in this study (Figure 5) has proved to be the best for reflectivity and for achieving an instant no-track condition.

Figure 4. Effect of bead type on reflectivity.

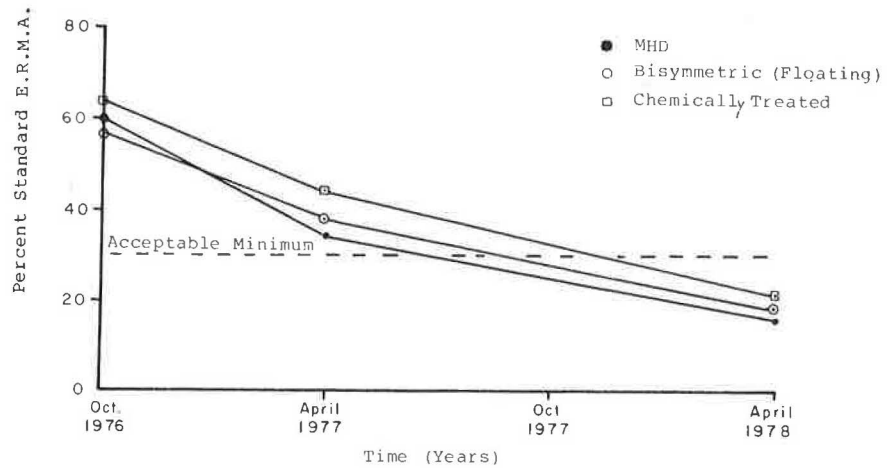
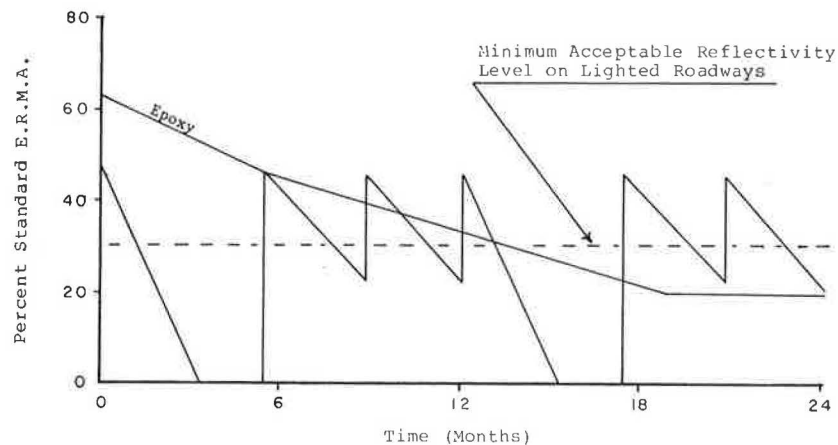


Figure 5. Retroreflective qualities of one epoxy application versus six placements of conventional traffic paint over two-year period.



As noted previously, the application of epoxy as a delineation system requires specialized equipment. When properly designed and built, however, the equipment is as easy to operate and maintain as a standard high-capacity paint striper. The cure time is longer than that for conventional paints and requires flooding with glass beads to compete with fast-drying conventional paints on a labor and equipment basis.

Curing and "No-Track Time"

In 1975, epoxy material was placed on both PCC and asphaltic concrete pavement surfaces. In 95 percent of this work, traffic cones were used to protect the epoxy stripes. Cure time ranged from 5 to 25 min, and the variation in cure time was caused by pavement surface temperatures: The higher the pavement temperature, the faster was the cure. A significant observation was made at an extremely cold surface temperature [-3.9°C (25°F)]: On the cold surface, the sprayed epoxy cooled rapidly and thickened, which lengthened the cure time well beyond that expected.

Five percent of the striping was done in areas where, in spite of traffic cones, it was known that traffic could not be stopped from crossing the fresh, uncured epoxy line. At one of the first installations where this problem was encountered, a bag of glass beads was opened and poured directly onto the fresh stripe and the traffic cones were removed. This allowed traffic to pass over the stripes on the glass beads and not contact the epoxy resin, which gives an instant no-track system. It

was felt that, because of the cost of coning and because crew members are exposed to danger while they set and retrieve cones, the flooding with glass beads would be a worthwhile investment.

Flooding with glass beads reduced the epoxy striping operation to the same two- or three-vehicle striping train as is used in Minnesota in the application of conventional fast-drying paint.

Weather Conditions

Weather conditions were considered a major variable to be evaluated in the 1975 epoxy program. Material was placed at pavement surface temperatures ranging from -3.9° to 37.8°C (25°-100°F) with the majority placed at 21.1°-26.6°C (70°-80°F). The only variation in the material caused by temperature was cure time. The increase in cure time at extremely cold temperatures did not hold up or delay the production schedule. The striped area had no tracking because of the increased viscosity.

Sky cover varied from cloudy and cold to clear and sunny. Pavement surface conditions ranged from dry to free water on the surface. None of these variations had any significant effect on the serviceability of the epoxy.

Snow and Ice Control

The average annual snowfall in the metropolitan area where the high-ADT test sections were located is 1.14 m (45 in). Minnesota DOT standard snow-removal and ice-control procedures were used on all test areas. Since no records were kept of snowplow

passes, no correlation can be made between snowplow passes and wear. Sanding and salting records are extremely difficult to evaluate because of the method of record keeping and the overlap of servicings by different maintenance crews at a given location.

All state DOT plows are equipped with tungsten-carbide blades and are placed directly on the pavement surface. Because a 15-mil thickness of epoxy is less than the inherent texture in a pavement surface, it is felt that the actual snowplowing does not contribute significantly to the wear of epoxy stripes; the combination of sanding, salting, and tire action is thought to be the most significant cause of stripe deterioration. The chipping that does occur is caused by lack of adhesion of the epoxy to the contaminated pavement surface.

The polyester stripes were damaged by winter wear, and again the damage was from chipping caused by poor adhesion to the pavement surface.

The 30-mil-thick thermoplastic stripes were severely damaged by snowplows. The surface was scraped so that the glass beads were removed and the filler crystals exposed.

1977-1978 PHASE: APPLICATION TO HIGH-ADT URBAN AND LOW- TO MEDIUM-ADT RURAL AREAS

As the evaluation of epoxy and polyester resins as pavement-marking materials on high-ADT urban roadways approached completion in 1976, a durable-marking-materials program on rural and urban trunk highways, funded by FHWA Section 205 safety funds, was begun. Two materials were used: hot-spray thermoplastic and the two-component epoxy-resin system developed in the 1975-1976 phase of the study. The glass beads used were bisymmetric beads on thermoplastic and chemically moisture-proofed beads on epoxy resin. The materials were placed as follows:

Application	Material	
	Thermoplastic	Epoxy
Bituminous surfaces	X	X
PCC		X
Undivided roadways		
Edge and centerline	X	X
Centerline only	X	
Divided roadways,		
centerline only	X	X
Urban roadways	X	X
Rural roadways	X	X
ADT		
160-60 000+	X	
825-11 000+		X

A total of 89 085.5 kg (98.2 tons) of thermoplastic and 14 383 L (3800 gal) of epoxy were purchased, including equipment and technical experts for placement. Yellow and white were included for both materials.

Table 1 compares the performance of epoxy resin and thermoplastic resin when both were placed as pavement-marking materials in Minnesota in 1977 and 1978. All thermoplastic was placed in the fall of 1977. The epoxy resin was placed in the fall of 1977 and 1978, and this accounts for the smaller number of test sections in the 12- and 18-month data for the epoxy resin.

The term failure is defined as retroreflectivity readings of less than 30 percent of standard Minnesota DOT ERMA, which requires repainting as soon as possible. The six-month (or spring 1978) data show a high number of failures caused by dirt and a salt-film coating on the stripes at the time of the early spring (March and April) testing. This

explanation becomes more plausible when the 12-month (fall 1978) data show the number and percentages of failures to be significantly lower for both thermoplastic and epoxy. The 18-month (spring 1979) data again show an increase in failures, but these data were acquired in May and June of 1979, which should have allowed enough time for rainfall to clean the stripes prior to testing.

The significant point to be made is the high number and percentage of failures of the thermoplastic in comparison with those of the epoxy resin. Thermoplastic failed because it was either worn off or plowed off the pavement surface or because the glass beads were removed and sheared off by snowplows as a result of the greater thickness of the material (30 mils).

Epoxy failed basically because of adhesion problems caused by contamination of the pavement surface (i.e., oil, rubber, and ethylene glycol). The majority of failures were in traffic weaving areas; areas of natural traffic drift, such as curves; and under bridges, where traffic carries contaminants and rainfall does not flush them off.

Delineation

Pavement markings have definite and important functions to perform in the proper scheme of traffic control. In some cases, they are used to supplement the regulations or warnings of other devices such as traffic signs or signals. In other instances, they are used alone and produce results that cannot be obtained by using any other device. In such cases, they serve as a very effective means of conveying certain regulations and warnings that could not otherwise be made clear.

Pavement markings have definite limitations: They are obliterated by snow, may not be clearly visible when wet, and may not be very durable when subjected to heavy traffic.

A delineation system has two elements: daytime and nighttime delineation. Both are necessary to the motoring public and are analyzed here separately.

Daytime Delineation

Daytime roadway delineation is not as critical as nighttime delineation because the overall topography of the roadway typically provides the driver with sufficient guidance information. Many people who are responsible for road striping are of the opinion that, if a stripe is not solid (that is, as originally placed), it should be redone. Many gallons of paint are used each year because someone decides "Those stripes look bad" or "We always paint that area at this time every year."

In dealing with durable pavement-marking materials, old habits in evaluating stripes must be set aside. An inspection trip over a section of highway that has been striped with epoxy, polyester, or thermoplastic will show some chipping after a period of time. When viewed from some distance, newly placed skip stripes appear to join together to form a solid, continuous line. This same phenomenon occurs on each individual stripe when chipping exists. This means that the average driver does not see the chipping until it becomes very severe--i.e., until probably more than 50 percent of the stripe is removed. At the present time, epoxy does chip because of poor adhesion caused by various types of surface contaminants.

Another phenomenon that has become apparent is that a nonbeaded stripe may show up under dirt and snow or slush and sand better than a beaded stripe because the nonbeaded stripe "holds" a minimum of dirt. The shiny enamel type of surface on the epoxy

Table 1. Performance of pavement-marking materials tested in Minnesota in 1977 and 1978.

Material	Number of Test Sections	Number of Months	Failures	
			Number	Percent
Thermoplastic	44	6	41	93
		12	34	77
		18	38	86
Epoxy	51	6	12	24
		25	2	8
		25	4	16

Note: Differences in the number of epoxy test sections are accounted for by the incomplete work in 1977.

Figure 6. Conventional traffic paint: new and one year old.

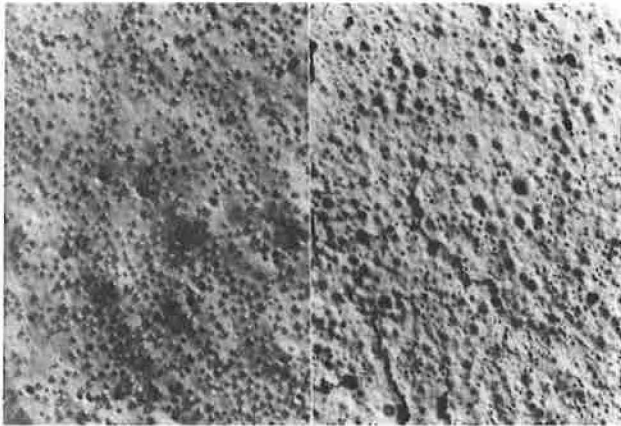


Figure 7. Polyester resin: new and two years old.

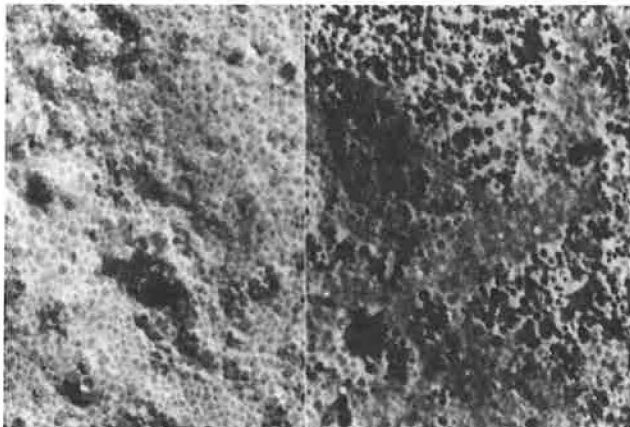


Figure 8. Epoxy: new and two years old.

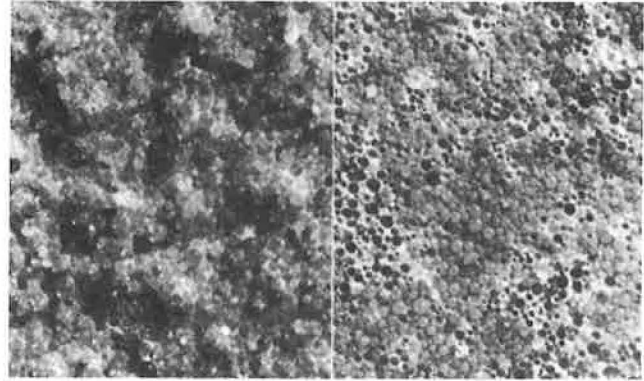
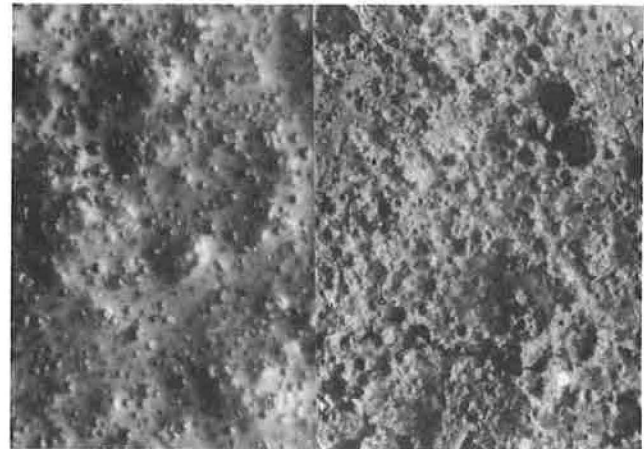


Figure 9. Thermoplastic: new and one year old.



is even better for this reason. Another distinct advantage of epoxy is the fact that it is so much more durable than a conventional paint and can be placed on both asphaltic concrete and PCC, whereas other durable materials cannot.

Placement of epoxy on the Interstate system in the Minneapolis-St. Paul metropolitan area and on rural roadways in Minnesota has given the motoring public an adequate daytime pavement-marking system throughout the year, which was never possible with conventional traffic paints.

Nighttime Delineation

Delineation at night is related to daytime

delineation in that it uses the daytime delineator as its color medium. The addition of glass spheres to the daytime delineation system gives the color (white-yellow) to the light the spheres reflect back to the driver for nighttime visibility.

Figures 6-9 are composite macrophotographs (about eight times actual size) of conventional traffic paint and the three durable materials, new and after one or two winters of use. Figure 6 shows a conventional traffic paint. In Figure 7, which shows a polyester resin, sufficient beads remain after two years to produce an acceptable retroreflective value. Unfortunately, the "It doesn't look too good--paint it" philosophy prevailed, and the majority of the material was overstripped after one year. Figure 8 shows an epoxy resin that continued to exhibit acceptable retroreflective value after two years. In Figure 9, which shows thermoplastic, very few glass beads remain after one year, and many pieces of filler crystals have been exposed by snowplowing. The material placed on the high-ADT urban system was 90 percent removed by spring. The retroreflective qualities were virtually nonexistent after one winter's service, and a moratorium was placed on the use of thermoplastic pavement-marking materials in Minnesota.

Cost Comparison

Table 2 gives cost figures associated with the three

durable-type pavement-marking materials used in this study and conventional traffic paint. It is assumed in the table that the cost of labor and equipment operators is equal for all four materials. The major difference per lineal foot is material and bead cost. Beads are applied to paint, polyester, and thermoplastic in approximately the same amount, and therefore the cost is shown to be equal. Epoxy requires flooding with glass beads at a rate four times that of paint to achieve the instant-no-track condition.

The basic total cost of epoxy placed is shown to be four times as high as that of conventional traffic paint, 44 percent higher than that of polyester, and 89 percent higher than the cost of thermoplastic. However, if the visible life of each material is considered, after one year epoxy equals paint in cost per lineal foot per day. If the epoxy has a life of two years, the cost is exactly half the cost per lineal foot per day of conventional paint, based on 4 paintings/year.

Four major points must be considered when one compares the three durable materials with conventional paint:

1. Conventional traffic paint has a very short life in comparison with durable materials on high-ADT (>70 000) roadways.
2. Two of the durable materials, polyester and thermoplastic, are not compatible with PCC pavements.

Table 2. Cost comparison of pavement-marking materials.

Item	Epoxy	Paint	Polyester ^a	Thermoplastic ^{a,b}
Cost ($\$/ft$)				
Material ^c	0.14	0.0125	0.093	0.063
Beads ^c	0.011	0.003	0.003	0.003
Application ^c	0.024	0.024	0.024	0.024
Traffic delay ^d	0.002	0.002	0.002	0.002
Traffic increase ^d	0.003	0.003	0.003	0.003
Total	0.18	0.045	0.125	0.095
Length of visible life (days)	365	90	365	180
Cost per day of visible life ($\$/ft$)				
1 year	0.000 49	0.0005	0.000 34	0.000 26
2 years	0.000 25	0.0005	0.000 34	0.000 26

^aNot compatible with PCC.

^bNot reflective after one winter in Minnesota.

^cBased on 1977 prices.

^dBased on Bernard Chaiken (1969), updated for inflation.

3. The reflectivity of thermoplastic does not survive one winter season in Minnesota.

4. On high-ADT roadways, conventional traffic paint does not survive the winter season reflectively or otherwise.

One additional point to be made concerning the economics of using epoxy is that it is a relatively new product in the pavement-marking field. Because of this, it is felt that material costs should go down as production goes up.

Material Service Life and Cost by ADT

Table 3 gives projected cost comparisons for applications of paint, epoxy, and thermoplastic for three ADT levels (and for polyester for the intermediate ADT level). These comparisons are based on number of applications over two years and four years and on the service life of each material in days.

For an ADT of <5000, a two-year epoxy life equals that of paint when the epoxy is placed at a 15-mil film thickness. A 10-mil-thick application of epoxy, at two years, shows a savings of 5 cents/ft. In addition, using epoxy requires less equipment and energy to service the same road distance.

For an ADT of 5000-15 000, paint equals epoxy in cost at two years. After that length of time, epoxy costs less on lower-volume roads.

For an ADT of 70 000, the cost is the same for 16 paintings (over four years) and four 15-mil-thick epoxy applications; for 12 paintings (over two years) and four 10-mil-thick epoxy applications, epoxy is 2 cents cheaper. Again, the thermoplastic is more expensive: 76 cents/ft over two years. In addition, thermoplastic is not acceptable for use on PCC. Again, epoxy has the added advantage of requiring the use of less equipment, consuming less energy, and providing an effective pavement-marking system 12 months out of the year. There has not been sufficient experience with polyester on high-ADT roadways to include it in the high-ADT cost comparison.

These data indicate that epoxy is basically cost effective when placed by Minnesota DOT crews. When epoxy is placed in areas where two or more paintings per year are normally done, it saves energy. Fewer paintings means a decrease in the use of state DOT equipment. For example, reducing three paintings on I-94 and other metropolitan-area roads to one epoxy application releases the vehicle used for painting

Table 3. Comparison of service life and cost of pavement-marking materials by ADT level.

ADT	Material	Service Life (days)	Two Years		Four Years	
			Number of Applications	Cost ($\$/ft$)	Number of Applications	Cost ($\$/ft$)
<5000	Paint	365	2	9	4	18
	Epoxy	>730				
	10 mils		1	13		
	15 mils		1	18		
5000-15 000	Thermoplastic	<180	4	38	8	76
	Paint	180	4	18	8	36
	Epoxy	>730				
	10 mils		1	13		
70 000	15 mils		1	18		
	Thermoplastic	<180	4	38	8	76
	Polyester	365	2	25	4	50
	Paint	90	12	54	16	72
	Epoxy	365				
	10 mils		4	52		
15 mils		4	72			
Thermoplastic	<180	8	76			

for work in other areas, which in turn reduces fleet requirements.

CONCLUSIONS

Epoxy

1. Epoxy adheres to both PCC and asphaltic concrete.

2. Epoxy can be applied in the same way as conventional traffic paints.

3. Epoxy can be placed on a wet pavement (where there is no free water), whereas other currently available materials cannot.

4. The temperature of the two epoxy components used in this study must be controlled. Part A (pigmented) has a maximum temperature of 85°C (185°F), and part B (catalyst) has a maximum temperature of 79°C (175°F). This must be adhered to regardless of ambient temperatures. A higher component temperature will result in too rapid polymerization and an epoxy stripe that does not bond to the pavement surface.

5. Epoxy withstands high traffic volumes, Minnesota winters, sanding, salting, and plowing and provides a delineation system that performs effectively for 12 months or more.

6. In most cases, the pavement surface can be adequately cleaned for epoxy placement by using an economical material (i.e., water).

7. The equipment and expertise are available to build a striper that is capable of placing both paint and epoxy.

8. Epoxy is economically justified for use on high-volume and some lower-volume roadways.

9. Epoxy will bond to itself and many other materials. However, if it is placed over materials such as conventional paints, its bond to the pavement will only be as good as the adhesion of that material. Therefore, it should not be placed over other pavement-marking materials unless they are substantially worn off or removed.

10. A 20-30 percent/stripe removal by chipping does not seriously affect the delineation provided by the stripe.

11. Overhead illumination reduces the retroreflective capability of the glass-bead/vehicle-headlight system.

12. The end result of this study is an epoxy-resin pavement-marking system that can be recommended for use on high-volume roadways, regardless of pavement type, especially in the snow-belt states.

13. Epoxy placed at a thickness of 10 mils can be an effective pavement-marking system on high-ADT roadways.

14. The use of epoxy can reduce equipment requirements and energy consumption.

15. The use of epoxy reduces environmental pollution because, unlike paint, it is 100 percent solids and contains no solvent that is evaporated into the atmosphere.

Polyester

1. Polyester adheres well to bituminous materials but not to exposed aggregate.

2. Polyester does not adhere well to PCC.

3. Application costs for polyester are higher than those for paint or epoxy because cones must be placed to protect the stripe during the longer cure time.

4. The retroreflective qualities of polyester were better after one year than those of conventional paint.

Thermoplastic (Spray)

1. Thermoplastic does not last through one winter on high-ADT urban roadways in Minnesota.

2. Thermoplastic does not meet Minnesota DOT standards for reflectivity after one winter.

RECOMMENDATIONS

1. Epoxy should be used as a durable pavement delineation system on high-volume roadways and considered for use on lower-volume roadways.

2. Surface cleaning such as sandblasting and hydroblasting should be used in specified areas, such as on curves, under bridges, in areas where traffic weaving is heavy, and on the extreme right skip line on multilane roadways that have frequent entrances and exits.

3. The inherent surface laitance of new PCC surfaces must be removed by sandblasting or hydroblasting to attain the adhesion needed for durability of epoxy.

4. Currently available equipment should be improved so that epoxy can be placed at a faster speed.

5. New epoxy formulations should be sought to create a competitive bidding situation and reduce material cost.

6. Currently available polyester and thermoplastic resins should not be considered for use on PCC surfaces under climatic conditions such as those in Minnesota.

7. Research should be done to develop an instant-no-track polyester system for use on bituminous surfaces.

8. Thermoplastic (spray) should not be considered for use in Minnesota unless snowplows are equipped with shoes.

9. Epoxy should be used for environmental reasons and to conserve energy.

10. Glass beads other than those recommended in this study should not be used without adequate testing.

ACKNOWLEDGMENT

The epoxy evaluated in this paper was produced by H. B. Fuller Company, the thermoplastic by Prismo, Inc., and the polyester by Glidden-Durkee. The beads used with the polyester resin were furnished by the Cataphote-Ferro Corporation. The beads used with the thermoplastic were bisymmetric (floating) and were furnished by Canasphere Corporation. The work done in this study included several years of experimentation before the involvement of and funding by FHWA.

I wish to thank the following individuals and organizations, without whose assistance this study could not have been adequately conducted: H. B. Fuller Company; Richard S. Gurney and Frank T. Buettel of H. B. Fuller Company; Dennis Sandstrom of Midway Industrial Supply; Robert Krueger of District 9 of the Minnesota DOT and his striping-crew foreman, Lloyd Josephson; Charles Neissner of the Office of Development, FHWA; the striping crews from the various Minnesota DOT districts; and John W. Zollars, Tom Robinson, and Ronnie Goodrich of the Minnesota DOT.

REFERENCE

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Upgrading of Marginal Aggregates for Improved Water Resistance of Asphalt Concrete

JAMES G. CHEHOVITS AND D. A. ANDERSON

Work done at Pennsylvania State University on the upgrading of marginal aggregates in asphalt concrete mixtures in order to improve the water resistance of the mixtures is reported. A number of alternate approaches or treatment classes that show promise for improving the water resistance of asphalt concrete mixtures were identified. Typical treatments from each class were used with six marginal aggregates that have performed unacceptably in the field. The water resistance of treated and untreated mixtures was studied by using a freeze-soak conditioning procedure and resilient modulus, E-modulus, and tensile-strength test procedures. Test procedure variability, effect of treatment on mixture properties, and effectiveness of the treatments in improving water resistance are discussed. It is concluded that a variety of approaches should be considered for improving the water resistance of asphalt concrete mixtures. These include the use of conventional antistripping additives, surfactants, hydrated lime, aggregate pretreatment, aggregate coatings, and modified binders. It was found that the effectiveness of the treatments varied with the different aggregates and that the treatments must be selected according to the mechanism responsible for moisture damage. The treatments affected the mechanical properties (stiffness and strength) of the mixtures to varying degrees.

The resistance of an asphalt concrete mixture to the effects of water is influenced by the properties of the aggregate, the composition and consistency of the asphalt, and the properties of the mixture. A number of aggregate properties, such as surface chemistry, surface texture, weathering, dust coatings, and surface abrasion, affect the resistance of asphalt mixtures to water. The literature has been reviewed by others (1,2) and is much too extensive to summarize here, except to note that the classic stripping mechanisms may be inadequate to explain many stripping problems in the field. For example, the classic stripping mechanisms do not account for the moisture damage caused by such factors as dust coatings or alteration resulting from weathering.

Laboratory studies have shown that asphalt properties can also affect the water resistance of asphalt mixtures. More viscous asphalts may improve water resistance, but this improvement is not always reflected in the field (3,4). Dohaney (4) has reported that asphalt composition can affect water resistance. Others (5) suggest that the importance of chemical composition is related to specific aggregate-asphalt combinations. Maupin (6) has, on the other hand, found no significant difference in laboratory stripping tests performed with asphalts from different sources. Finally, mixture properties play an important role in determining water resistance. For example, asphalt concrete mixtures with high permeability and low asphalt content are more susceptible to moisture damage than denser mixtures with high asphalt contents (1).

The resistance to water of a mixture made with a moisture-sensitive aggregate may be improved by treating the aggregate, treating the asphalt, or changing the mixture design. The addition of antistripping additives to the asphalt is routinely specified by many agencies. For maximum effectiveness, these additives must be matched to the particular aggregate being used; moreover, their effectiveness has been found to vary with the asphalt source (7) and the pH of the water present (8). Although antistripping additives may improve moisture resistance in the laboratory, similar improvements have not always been obtained in the field (5). This may be attributable to the heat

stability of the additives or may reflect the inadequacy of present laboratory test procedures. In some instances, antistripping additives have been found to change the physical properties of asphalt cements and to greatly increase loss on heating (ASTM D1574) (9,10).

Additives applied directly to the aggregate may also improve the moisture resistance of some asphalt-aggregate mixtures. Heavy metal ions dissolved in a sodium oleate solution and added to the aggregate have been shown to improve the watability of asphalt on aggregate surfaces (3). Fromm (11) determined that heavy metal cations, when applied to aggregate surfaces, could improve moisture resistance. Ferric naphthate was found to be particularly effective. Sodium dichromate has also been effective when added to the aggregate in an aqueous solution.

The effectiveness of hydrated lime for improving the resistance of certain asphalt mixtures has been known for many years. Hydrated lime is often added as a mineral filler, but it is more effective as an antistripping additive if it is added as a slurry and allowed to cure for several days. Recent research suggests that the hydrated lime absorbs carboxylic acids in the asphalt, which results in a more water-resistant asphalt-aggregate bond (12). Hydrated lime has also been found to change the mechanical properties of asphalt mixtures (13).

RESEARCH APPROACH

The objective of the research described in this paper was to identify alternative approaches to improving the water resistance of asphalt concrete mixtures made with marginal aggregates. Based on a review of the general problem, a number of approaches appeared to be valid:

1. Traditional antistripping additives--surfactants added to the asphalt to provide a physical-chemical bond between the asphalt and the aggregate;
2. Metal ions--surfactants added directly to the aggregate surface that change the surface charge of the aggregate;
3. Hydrated lime--an additive that alters the chemical composition of the asphalt;
4. Sulfur-extended asphalt--modification of the binder;
5. Acid wash--acid treatment of the aggregate to remove surface contaminants and, if possible, to alter the surface chemistry of the aggregate; and
6. Epoxy coating--total encapsulation of the aggregate to isolate it from the asphalt.

Typical treatments representing each of these approaches were used on six marginal aggregates that have given unacceptable field performance. The water resistance of the treated and untreated mixtures was evaluated by using resilient-modulus and tensile-strength testing procedures. Moisture damage was induced by means of a freeze-soak conditioning procedure. Test variability, the effect of the treatments on mixture properties, and the effectiveness of the treatments in improving the water resistance of the mixtures were determined. The scope of the research project required that off-the-shelf materials be used for the treatments,

and no attempt was made to optimize the various treatments.

Asphalts

The asphalts were supplied by those states that indicated that the source of an asphalt can influence its resistance to water. Otherwise, an AC-20 asphalt selected by the researchers was used. The properties of the asphalts used in the study are given in Table 1.

Aggregates

Each of the aggregates used in the study is described below. Source and other pertinent data for each of the aggregates are given in Table 2.

1. The Grayson, Georgia, granite is a coarse-grained, partially metamorphosed crushed granite with loosely bonded grains on factured faces.

2. Granite from the Piedmont region of southeastern Virginia is coarse-grained, crushed granite. Compared with the Grayson granite, the Piedmont granite exhibits a much higher degree of interlocking between grains and contains less silica.

3. Colorado gravel is a crushed, siliceous river gravel from a heterogeneous deposit that consists of granites, cherts, and quartzites.

4. Roseburg, Oregon, basalt is a crushed, homogeneous basalt from the Mt. Nebo formation that weathers rapidly, forming a surface with a dust coating, and is typical of many coastal Oregon basalts. The source of the asphalt reportedly influences the water resistance of this aggregate.

5. Gravel from Pocatello, Idaho, is composed of uncrushed, rounded particles that are mainly quartzites and some basalt, limestone, and metamorphic material. The asphalt source reportedly influences the water resistance of this aggregate.

6. Holbrook, Arizona, gravel is a crushed river gravel composed mainly of siliceous materials that range from cryptocrystalline particles to sandstones. Some of the particles tend to expand and disintegrate when exposed to water. The asphalt source reportedly influences the water resistance of this aggregate.

7. The minus-4.75-mm (No. 4) sieve material (fine aggregate) used in all mixtures is a blend of sand and mineral filler. The sand is a washed, siliceous river sand from Montoursville, Pennsylvania, that has a good performance record in asphalt mixtures. The filler is a ground dolomitic limestone.

Mixture Design

The mixtures were proportioned by using the Marshall method of mixture design with 50-blow compaction. The gradations of the mixtures met the limits of ASTM D3515-77 for a 3A binder mixture and in most cases approximated the gradations used by the supplying agencies. The intent of the mixture design procedure was to proportion the mixtures to meet gradation and voids-in-mineral-aggregate requirements and to contain 4.5 ± 1 percent air voids. Gradation and asphalt content were the same for the treated and untreated (control) aggregates except for the epoxy-coated aggregates. The gradation of the epoxy-coated aggregates was corrected for specific gravity, and the asphalt content was reduced to account for reduced asphalt absorption. Additional information about the mixture design is given elsewhere (14).

Test Procedures

A freeze-thaw procedure recently proposed by Lottman (15) was used to induce moisture damage in the compacted specimens. The specimens were vacuum saturated at room temperature, frozen at -17.8°C (0°F), and then soaked for 24 h in a 60°C (140°F) water bath.

Moisture damage resulting from the conditioning procedure was evaluated by performing E-modulus, tensile-strength (16), and resilient-modulus (17) tests on untreated and treated triplicate specimens before and after conditioning. The testing schedule used in the study is shown in Figure 1. Details of the test procedures can be found elsewhere (16,17). In addition to the mechanical testing, the broken surfaces of each specimen subjected to the tensile-strength test were examined visually to evaluate the degree of stripping of the coarse aggregate.

TREATMENT METHODS

Six modifications were made to the asphalt concrete mixtures to improve their resistance to water damage. These treatments modified either the aggregate or the binder. As a result of limited aggregate supplies, all treatments were not used with all aggregates. The asphalt-aggregate treatment combinations used in the study are given in Table 3. Only the coarse (plus-4.75-mm) aggregate was treated with epoxy, hydrated lime, acid wash, and sodium dichromate treatments. Specific details of the treatment methods can be found elsewhere (14).

Epoxy Encapsulation

Epon 828, a diglycidyl ester of disphenol A, manufactured by Shell, was used to encapsulate the aggregates. Both the specific gravity and absorption (Table 2) of the aggregate were lowered as a result of the coating. The coating was so thick that much of the surface texture of the aggregate was lost and the surfaces acquired a smooth, glasslike texture.

Hydrated Lime

A high calcium-hydrated lime was applied to individually batched coarse-aggregate fractions (plus 4.75 mm) in slurry form. Approximately 1.0 percent hydrated lime (based on the weight of the coarse aggregate) was added to the coarse aggregate. The mineral filler content of the mixture was reduced by the amount of hydrated lime added to the mixture--approximately 0.5 percent by weight of the total mixture.

Acid Wash

A commercially available material, composed primarily of sulfuric acid, was used in treating the coarse-aggregate surfaces. The acid wash removed loosely bound surface material from the Georgia and Arizona aggregates and caused foaming on the surfaces of the Georgia, Virginia, and Oregon aggregates. The surfaces of the Georgia and Virginia aggregates were abraded during the acid-wash procedure, and they were thus smoother and less angular than the untreated aggregate. The physical properties of the treated aggregates were essentially unchanged (Table 2), except that absorption by the Oregon aggregate was reduced from 3.5 to 2.8 percent, primarily because of the removal of dust coatings.

Table 1. Physical properties of asphalts.

Property	Asphalt			
	AC-20	120-150	AR 4000	AR 2000
Original				
Penetration at 25°C, 100 g, 5 s	56	109	140	84
Softening point (°C)	48.9	45.3	43.3	44.2
Specific gravity	1.028	1.024	—	—
Dynamic (60°C) viscosity (P)	1941	495	1142	1128
Kinematic (135°C) viscosity (cSt)	365	175	338	194
TFOT loss (%)	0.05	0.34	0.84	0.21
TFOT residue				
Penetration at 25°C, 100 g, 5 s	28	57	79	56
Softening point (°C)	53.9	49.2	49.7	47.2
Dynamic (60°C) viscosity (P)	5843	1586	3322	2010
Kinematic (135°C) viscosity (cSt)	562	289	577	265
Specific gravity	—	—	1.034	1.019
Rostler composition				
Asphaltenes (%)	19.1	22.2	16.0	38.7
Nitrogen base (%)	30.6	14.6	42.2	28.9
Acidifins (%)				
First	19.3	22.3	14.7	19.4
Second	10.5	24.0	19.0	10.7
Paraffins (%)	20.5	11.9	8.1	2.3
Rostler parameter	1.61	1.03	2.10	3.72

Notes: $t^{\circ}\text{C} = (t^{\circ}\text{F} - 32)/1.8$; 1 P = 0.1 Pa-s; 1 cSt = 0.01 cm²/s.
TFOT = thin-film oven test.

Table 2. Properties of aggregates.

Aggregate	Untreated		Epoxy Coated		Acid Wash	
	Bulk Specific Gravity	Absorption (%)	Bulk Specific Gravity	Absorption (%)	Bulk Specific Gravity	Absorption (%)
Georgia granite	2.615	0.6	2.510	0.5	2.622	0.5
Virginia granite	2.631	0.5	2.450	0.8	2.625	0.5
Colorado gravel	2.598	1.0	2.510	0.8	2.600	0.9
Oregon basalt	2.711	3.5	2.581	0.5	2.724	2.8
Idaho gravel	2.587	0.9	2.508	0.8	2.566	1.0
Arizona gravel	2.569	1.3	2.490	0.9	2.541	1.6
Sand	2.530	2.8	—	—	—	—
Filler	2.816 ^a	—	—	—	—	—

Note: Properties determined by using ASTM C127-77.
^a Apparent specific gravity measured in kerosene.

Surface Active Agent

The surface active agent used was amine, a commonly used asphalt antistripping additive, which was mixed directly into the asphalt cements at a concentration of 0.5 percent by weight of the asphalt cement. The results of the consistency tests performed on the treated asphalts are given in Table 4. The addition of the amine had no measurable effect on either the AR-2000 or A-4000 asphalt; however, noticeable hardening did occur with the AC-20 asphalt. Although no firm conclusions can be drawn from the small amount of data presented here, it appears that the addition of antistripping agents may affect the properties of the asphalt.

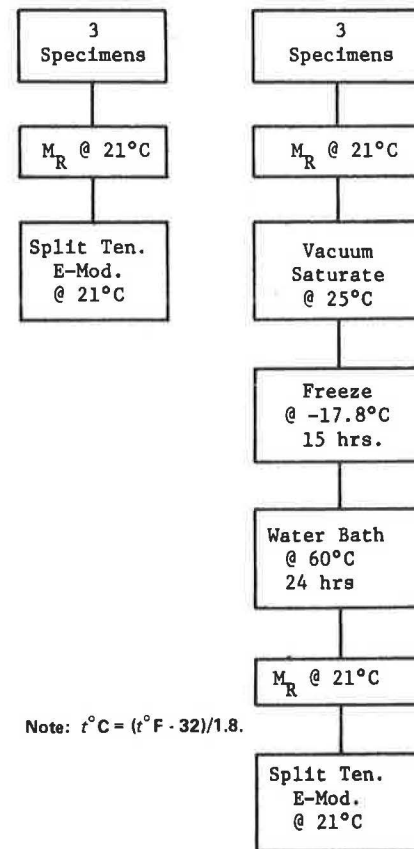
Sodium Dichromate

The coarse aggregate was moistened with a 2 percent (by weight) aqueous sodium dichromate solution. Sufficient solution was added to yield a 0.025 percent sodium dichromate residual (by weight of aggregate) on the dried coarse-aggregate surfaces.

Sulfur-Extended Asphalt

The sulfur-extended asphalts were prepared by adding

Figure 1. Conditioning sequence for test specimens.



Note: $t^{\circ}\text{C} = (t^{\circ}\text{F} - 32)/1.8$.

elemental sulfur to the AC-20 asphalt cement, which was heated to 135°C (275°F). Properties of the sulfur-extended asphalt are given in Table 4.

EFFECT OF TREATMENTS ON THE MECHANICAL PROPERTIES OF UNCONDITIONED MIXTURES

The sample mean, standard deviation, and coefficient of variation were computed before and after moisture conditioning for each mixture-treatment-test combination. To provide a comparison of the variability between the different test procedures, coefficients of variation were averaged for each mixture-treatment-test combination. The coefficient of variation for Marshall stability and tensile strength was 9.7 percent, and the average coefficient of variation for resilient modulus (M_R) was 18.2 percent. Although the variability of the resilient-modulus results was similar before and after conditioning (19.2 versus 17.2 percent), the variability of the tensile-strength results nearly doubled after conditioning (6.7 versus 12.6 percent). An increase in variability after conditioning might be expected because differences in the test samples should be amplified by the conditioning procedure. We consider the increased variability to be a reflection of the sensitivity of the tensile-strength test.

Throughout the study it was noted that the treatments not only affected the water resistance of the mixtures but also affected the mechanical properties of the mixtures before moisture conditioning. The effect of each treatment on the mechanical properties of the mixtures before conditioning was evaluated by dividing the average test parameter for the treated mixture by the

Table 3. Aggregate-asphalt treatment combinations used in the study.

Aggregate	Untreated	Epoxy Coating	Acid Wash	Hydrated Lime	Amines	Sodium Dichromate	Sulfur-Extended Asphalt	
							15 Percent	30 Percent
Georgia granite	1	1	1	1	1	1	1	1
Virginia granite	1	1	1	1	1 ^a	1 ^a	1 ^a	1 ^a
Colorado gravel	1	1	1	1	1	1	1 ^a	1 ^a
Oregon basalt	1,2	2	2	2	2	2	1	1
Idaho gravel	1,3	3	3	3	3	3	1	1
Arizona gravel	1,4	4	4	4	4	4	1	1

Note: 1 = AC-20 asphalt, 2 = Oregon (AR-4000) asphalt, 3 = Idaho (120-150 per) asphalt, and 4 = Arizona (AR-2000) asphalt.
^aNot tested.

Table 4. Physical properties of treated asphalts.

Asphalt	Treatment	Penetration at 25°C	Softening Point (°C)	Viscosity	
				Dynamic, 60°C (P)	Kinematic, 135°C (cSt)
AC-20	Untreated	56	48.9	1941	365
	Amines	46	49.4	2630	414
	Sulfur-extended asphalt				
	15 percent	105	44.7	698	143
	30 percent	61	45.6	744	167
120-150	Untreated	109	45.3	495	175
	Amines	121	42.8	572	192
AR-4000	Untreated	140	43.3	1142	338
	Amines	135	42.5	1149	338
AR-2000	Untreated	84	44.2	1128	194
	Amines	85	43.9	1118	193

Note: t°C = (t°F - 32)/1.8; 1 P = 0.1 Pa·s; 1 cSt = 0.01 cm²/s.

average test parameter for the corresponding untreated mixture. The resulting ratios are given in Table 5. Ratios greater than 1.0 indicate an increase in the test parameter as a result of the treatment. The treatments reduced the resilient modulus to a greater degree than the tensile strength, which indicated that the treatments had a greater effect on stiffness than on strength.

Epoxy Coating

The epoxy coating increased the flow values and lowered the tensile strength and resilient modulus of all the mixtures. These effects were attributable to the smooth surface texture created by the epoxy coating. In contrast, except in the case of the Arizona aggregate, Marshall stability values were little affected by the epoxy coating. The epoxy reacted with the Arizona aggregate (14) so that some of the particles decomposed when exposed to water. On the basis of these observations, we suggest that the surface texture of coated aggregates should be considered in the development of new coating procedures. Potential coating-aggregate reactivity and durability should also be considered.

Hydrated Lime

The effect of hydrated lime on the mechanical properties of the aggregates varied. The hydrated lime decreased the resilient modulus of the Virginia mixture by 64 percent but increased the resilient modulus of the Oregon mixture by 64 percent. Other investigators (13) have noted that the addition of hydrated lime to asphalt concrete mixtures reduces the resilient modulus and that this reduction varies with aggregate type and asphalt source. Further research on additional aggregates and asphalts is needed to determine the effects of hydrated lime.

Table 5. Ratios of treated to untreated mixture properties.

Treatment	Test	Aggregate						
		Georgia	Virginia	Colorado	Oregon	Idaho	Arizona	
Epoxy coating	M _R	0.35	0.33	0.57	0.37	0.37	0.21	
	Tensile strength	0.77	0.76	0.76	0.55	0.63	0.47	
	Stability	1.00	1.08	1.09	0.99	1.33	0.46	
Hydrated lime	Flow	1.31	1.21	1.27	1.21	1.33	1.13	
	M _R	0.51	0.46	1.14	1.64	1.00	0.84	
	Tensile strength	0.84	1.02	1.19	1.05	1.03	0.90	
Acid wash	M _R	0.43	0.31	0.73	1.00	0.61	0.63	
	Tensile strength	0.86	0.86	1.00	0.97	0.99	0.85	
	M _R	0.51	-	0.65	0.88	0.57	0.51	
Amines	Tensile strength	0.75	-	-	0.99	1.07	0.93	
	M _R	0.77	-	1.05	0.88	0.56	0.58	
	Tensile strength	0.82	-	-	1.09	-	0.89	
Sulfur-extended asphalt	15 percent	M _R	0.46	-	-	0.56 ^a	0.39 ^a	0.53 ^a
		Tensile strength	0.64	-	-	0.70	0.70	0.69
	30 percent	M _R	0.40	-	-	0.61 ^a	0.44 ^a	0.86 ^a
		Tensile strength	0.61	-	-	0.75	0.78	0.87

^aComputed with respect to the AC-20 asphalt.

Acid Wash

The acid-wash treatment reduced the resilient modulus and tensile strength of all mixtures. The effect on resilient modulus was more pronounced. The surfaces of the Georgia and Virginia aggregates were abraded by the acid-wash procedure, which caused those aggregates to be smoother and more rounded than the untreated aggregates. This abrasion may be the cause of the lower resilient modulus and tensile strength of these mixtures.

Amine

The addition of an amine to the asphalts lowered the resilient modulus of the mixtures but lowered the tensile strength only of the Georgia mixture. The various aggregates and asphalts responded differently to the amine treatment and the testing procedures, and no trends could be identified.

Sodium Dichromate

The addition of sodium dichromate to the aggregates followed the same pattern observed for the amines, lowering the resilient modulus of the mixtures but lowering the tensile strength only of the Georgia mixture. Both of the treatments are of the surfactant type, but the amines were added to the

asphalt cements and the sodium dichromate was added to the aggregates.

Sulfur-Extended Asphalt

The sulfur-extended asphalt changed the resilient modulus and the tensile strength of the mixtures. The 15 percent replacement ratio resulted in a decrease in the modulus and tensile strength of the mixtures investigated. Except for the Georgia aggregate, the mixtures containing the 30 percent sulfur-extended asphalt had a higher modulus and tensile strength than those containing the 15 percent sulfur-extended asphalt. This agrees with the findings of others (18,19). This effect is the result of changes in the consistency of the sulfur-extended asphalt, which is a function of replacement percentage.

EFFECT OF TREATMENTS ON MOISTURE SENSITIVITY

The effectiveness of the treatments for improving the water resistance of the mixtures was evaluated by making comparisons of the data and by visually examining fractured surfaces of the conditioned specimens for stripping. The results of the visual examination are given in Table 6. All of the untreated mixtures were severely damaged by the moisture conditioning procedure. The effectiveness of the treatments varied with the different mixtures and treatment types.

Retention ratios were calculated for each mixture-treatment-test combination by dividing the average test parameter after conditioning by the average test parameter before conditioning. Retention ratios for resilient modulus and tensile strength are given in Table 7 and represent the fraction of the test parameter retained after conditioning. Improvement ratios were calculated for each mixture-treatment-test combination by dividing the treated test parameter obtained after conditioning by the untreated test parameter obtained after moisture conditioning. These ratios are given in Table 8.

An analysis of variance was performed by using the retention and improvement ratios as dependent variables. Aggregate source, treatment method, and test method were considered as independent variables. Test methods included in the analyses were resilient modulus (17), tensile strength (15), and E-modulus (15). A series of full factorial models was constructed based on the aggregate-asphalt-treatment combinations given in Table 3. Results for a typical analysis-of-variance model are given in Table 9. The statistical analyses indicated that the treatments varied in their effectiveness and that the effectiveness of each treatment varied with the different aggregates. The structure of the models is as follows: Retention ratios = aggregate + treatment + test + all second-order interactions + error. The formulas for the null hypothesis (H_0) and the alternative hypothesis (H_1) are given below ($R^2 = 0.947$). For H_0 ,

$$\mu_1 = \mu_2 = \dots = \mu_n \quad F^* \leq F(\alpha, v_1, v_2) \quad (1)$$

For H_1 ,

$$\mu_1 \neq \mu_2 \neq \dots \neq \mu_n \quad F^* > F(\alpha, v_1, v_2) \quad (2)$$

The effectiveness of the treatments, as measured by improvement and retention ratios, was dependent on the type of test. In general, tensile strength was affected differently than stiffness (E-modulus and resilient modulus). Improvements in tensile

strength were not necessarily associated with improvements in stiffness and vice versa. The second-order interactions were significant for most of the models, indicating that, as measured by the improvement and retention ratios, (a) the different aggregates reacted differently to the different treatments, (b) the different test procedures gave different measures of the effectiveness of the different treatments, and (c) the different test procedures gave different measures of treatment effectiveness for the different aggregate sources.

Epoxy Coating

Encapsulation of the aggregates with epoxy provided some degree of moisture improvement in all of the mixtures, but the effect was particularly evident for Georgia aggregate. It is surprising that all of the treated aggregates did not have similar retention ratios because they were all totally encapsulated in epoxy. The most likely explanation is the differences in the shape and surface texture of the coated aggregates. The epoxy-coated aggregates did show visual evidence of stripping and, if further research is done with aggregate coatings, particular attention should be paid to the compatibility of the coating with asphalt as well as the thickness of the coating and the texture of the coated aggregate.

Hydrated Lime

Both the improvement and retention ratios were significantly improved by the addition of hydrated lime to the mixtures. Based on visual examination (Table 6), the use of hydrated lime reduced the stripping on all aggregates investigated, but this was not always reflected in the improvement or retention ratios. In view of this and the improvement in the mechanical properties of the mixtures, the hydrated lime was the most successful of the treatments investigated.

Acid Wash

The acid wash produced slight improvements in the moisture resistance of the Arizona and Oregon mixtures but lowered the moisture resistance of Virginia, Colorado, and Idaho mixtures. Although loose surface material was removed from the Georgia and Oregon aggregates, the remaining surfaces were sensitive to stripping, as Table 6 indicates.

Amines

The addition of amines to the asphalts improved the retention ratios and improvement ratios of the mixtures investigated. All of the amine-treated mixtures exhibited less stripping than the untreated mixtures (Table 6). Although the amines did significantly improve the water resistance of the aggregates, none of the treated mixtures can be considered successful based on a 0.75 tensile-strength retention ratio as recommended for field acceptance (6,15).

Sodium Dichromate

Both the improvement and retention ratios increased for all the mixtures treated with sodium dichromate. The improvement in moisture resistance was approximately the same as it was for the amine-treated mixtures. Based on visual examination of stripping, all of the sodium-dichromate-treated mixtures stripped to a lesser extent than the untreated mixtures. The observed stripping also

Table 6. Observed stripping in freeze-soak-conditioned specimens.

Treatment	Aggregate					
	Georgia	Virginia	Colorado	Oregon	Idaho	Arizona
Untreated	H	H	H	H	H	H
Epoxy coating	M	M	L	L	M	M
Hydrated lime	L	L	L	L	L	L
Acid wash	H	H	H	M	H	H
Amines	L	-	M	L	L	M
Sodium dichromate	L	-	L	M	L	M
AC-20 asphalt	-	-	-	H	H	H
Sulfur-extended asphalt						
15 percent	H	-	-	H	H	H
30 percent	H	-	-	H	H	H

Note: H = high (>70 percent stripping on coarse aggregate), M = medium (30-70 percent stripping on coarse aggregate), and L = low (<30 percent stripping on coarse aggregate).

Table 7. Average retention ratios for different treatments.

Treatment	Test	Aggregate					
		Georgia	Virginia	Colorado	Oregon	Idaho	Arizona
Untreated	M _R	0.08	0.14	0.27	0.21	0.21	0.21
	Tensile strength	0.27	0.39	0.46	0.45	0.43	0.39
	Stability	0.47	0.68	0.60	0.35	0.26	0.28
Epoxy coating	M _R	0.96	0.58	0.49	0.52	0.59	0.49
	Tensile strength	0.75	0.67	0.80	0.77	0.66	0.72
	Stability	1.13	0.57	0.53	0.44	0.42	0.61
Hydrated lime	M _R	0.72	0.63	0.54	0.62	0.41	0.46
	Tensile strength	0.62	0.69	0.59	0.72	0.59	0.70
Acid wash	M _R	0.27	0.19	0.19	0.42	0.06	0.31
	Tensile strength	0.24	0.27	0.33	0.66	0.12	0.53
Amines	M _R	0.34	-	-	0.53	0.53	0.29
	Tensile strength	0.58	-	-	0.66	0.57	0.46
Sodium dichromate	M _R	0.41	-	-	0.37	-	0.42
	Tensile strength	0.63	-	-	0.49	-	0.58
Sulfur-extended asphalt							
15 percent	M _R	0.02	-	-	- ^a	0.03 ^b	0.11 ^b
	Tensile strength	0.06	-	-	0.19 ^b	0.07 ^b	0.30 ^b
30 percent	M _R	0.08	-	-	0.02 ^b	0.06 ^b	0.04 ^b
	Tensile strength	0.23	-	-	0.50 ^b	0.19 ^b	0.16 ^b

^aToo weak to test.

^bComputed with respect to AC-20 mixtures.

followed the same trends as those for the amines.

Sulfur-Extended Asphalt

Neither of the sulfur-extended asphalts improved the moisture resistance of the mixtures investigated. This observation is based on the values of the retention and improvement ratios as well as observed stripping in the mixtures (Table 6). Based on measurements recorded during the mechanical testing, mixtures made with the sulfur-extended asphalt tended to expand more than the other mixtures and tended to turn slightly brownish when subjected to the freeze-soak conditioning procedure. Fromm (11) noted that the presence of a brownish coloration in

Table 8. Average improvement ratios for conditioned samples.

Treatment	Test	Aggregate					
		Georgia	Virginia	Colorado	Oregon	Idaho	Arizona
Epoxy coating	M _R	4.22	1.39	1.04	0.89	1.03	0.49
	Tensile strength	2.09	1.30	1.31	0.93	1.10	0.86
	Stability	2.40	0.91	0.97	1.24	2.18	1.01
Hydrated lime	M _R	4.59	2.11	2.30	3.02	1.92	1.86
	Tensile strength	1.90	1.81	1.54	1.68	1.63	1.61
Acid wash	M _R	1.42	0.43	0.50	1.96	0.17	0.93
	Tensile strength	0.97	0.60	0.72	1.41	0.32	1.16
Amines	M _R	2.19	-	1.18	2.18	1.40	0.72
	Tensile strength	1.57	-	1.44	1.45	1.63	1.10
Sodium dichromate	M _R	3.93	-	1.95	1.51	1.04	1.17
	Tensile strength	1.88	-	1.64	1.18	1.48	1.33
Sulfur-extended asphalt							
15 percent	M _R	0.10	-	-	- ^a	0.37 ^b	0.79 ^b
	Tensile strength	0.15	-	-	1.09 ^b	0.26 ^b	1.06 ^b
30 percent	M _R	0.42	-	-	0.47 ^b	1.10 ^b	0.48 ^b
	Tensile strength	0.51	-	-	0.30 ^b	0.75 ^b	0.72 ^b

^aToo weak to test.

^bComputed with respect to AC-20 mixtures.

Table 9. Typical analysis of variance for retention ratios.

Source	df	SS	MS	F*	F _{v1, v2}	
					α = 0.1	α = 0.01
Aggregate	3	0.158	0.527	8.03	2.23	4.31
Treatment	7	2.94	0.420	64.22	1.87	3.12
Test method	2	0.752	0.376	57.44	2.44	5.18
Aggregate x treatment	21	0.448	0.213	3.26	1.60	2.35
Aggregate x test method	6	0.0619	0.0103	1.58	1.93	3.29
Treatment x test method	14	0.295	0.0211	3.22	1.67	2.54
Error	40	0.262	0.00654	-	-	-

an asphalt-aggregate mixture that was subjected to water signified the formation of an expanding asphalt-water emulsion that was highly sensitive to water. This type of emulsion may have formed in the conditioned sulfur-extended-asphalt specimens. The claim that sulfur-extended asphalts in general show high resistance to water (20) does not seem to be justified on the basis of the findings of this study.

SUMMARY AND CONCLUSIONS

Inadequate resistance to the effects of water is a problem with many asphalt concrete mixtures. To improve the performance of many aggregates in current use and to upgrade aggregates that are currently unacceptable, new treatment methods and procedures need to be developed. Traditional antistripping additives cannot be expected to upgrade all problem aggregates because of the variety of mechanisms that are responsible for moisture damage. A classification scheme that considers various moisture-damage mechanisms should be developed so that treatments can be selected on a rational basis.

In this study, a number of approaches to the upgrading of water-sensitive aggregates were identified. Further study is needed to optimize the

various treatment approaches and to develop a rational procedure for matching the treatment with particular aggregates and aggregate-asphalt mixtures. Based on the results of this study, the following conclusions can be made:

1. A variety of mechanisms influence the resistance of asphalt mixtures to the effects of water, and this must be recognized in the process of selecting an appropriate upgrading procedure.

2. Upgrading procedures can affect the mechanical behavior of mixtures. Strength and stiffness may be increased or decreased.

3. Different test procedures (tensile strength, E-modulus, or resilient modulus) provide different measures of treatment effectiveness, and improvements in retained strength are not always accompanied by equivalent improvements in retained modulus and vice versa.

4. The effectiveness of a particular treatment may depend on the properties of both the aggregate and the asphalt.

5. Several of the treatment approaches show promise for improving water resistance and should be studied further.

ACKNOWLEDGMENT

This work was sponsored in part by the American Association of State Highway and Transportation Officials, in cooperation with the Federal Highway Administration, and was conducted as part of the National Cooperative Highway Research Program (NCHRP). P.D. Cady was the principal investigator for the NCHRP contract. P.R. Blankenhorn and D.E. Kline were responsible for developing the epoxy coatings. Their assistance is gratefully appreciated.

The opinions and findings expressed or implied in this paper are ours and are not necessarily those of the Transportation Research Board, the National Academy of Sciences, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, or the individual states participating in the National Cooperative Highway Research Program.

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Publication of this paper sponsored by Committee on Mineral Aggregates.

Preliminary Study of Coatings and Impregnants for Upgrading Frost-Sensitive Aggregates

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Laboratory tests carried out to evaluate the potential of various types of coatings and impregnants as means of upgrading frost-sensitive aggregates are described. Although some of the materials tested would probably not be economical for commercial application, they were chosen to represent classes of materials that might include some that could be economically feasible. The impregnants tested were epoxy (diluted with a solvent), methyl methacrylate, boiled linseed oil, polyethylene glycol, and sulfur. The coatings included epoxy and linseed oil emulsion. Two aggregates were used that were known to be highly frost-susceptible, but for different reasons. One was a chert river gravel of moderately high absorption but very low permeability. Its mode of failure under freezing and thawing conditions is internal fracturing and dilation of the aggregate particles as a result of the development of excessive hydraulic pressures. This aggregate was treated by impregnation because of the high capillary potentials offered by the small pore sizes. The second aggregate was a vesicular, highly porous andesite with very large pore sizes. Its primary mode of failure in concrete under freezing conditions involves the expulsion of water to the paste phase ahead of the advancing frost zone. Because of the large absorptive capacity of this aggregate, an attempt was made to block the pores by using external surface coatings. Triplicate sets of specimens were used in the freeze-thaw tests for each aggregate-treatment combination in accordance with ASTM C682. Both coatings and all impregnants except sulfur were found to render the test aggregates non-frost-susceptible. Specimens containing untreated (control) aggregates failed early in freeze-thaw tests.

Deterioration of portland cement concrete (PCC) as a result of freezing and thawing can occur because of deficiencies in either the paste phase or the aggregate phase. The paste phase can readily be protected by incorporating a suitable entrained air-void system.

The coarse-aggregate fraction can be damaged by freezing and thawing (with resultant damage to the concrete) in two ways or in a combination of these ways (1,2). When the aggregates have moderate to high porosity and low permeability, the hydraulic pressures developed in the rock pores ahead of the advancing ice front during freezing can be of sufficient magnitude to cause fracturing of the aggregate. When the aggregates have high porosity and high permeability, the pressures developed by water being forced out of the rock during freezing can break the paste-aggregate bond and disrupt the surrounding paste phase.

In view of the nature of the destructive mechanisms involved, it was considered highly likely that frost-sensitive aggregates could be benefited by a process of drying followed by sealing of the penetrable pores. Indeed, this approach has been suggested by others (3,4).

For aggregates that have small pore sizes (low permeabilities), this could probably best be accomplished by impregnation, since the high capillary potential of these aggregates will aid in the upgrading process. Aggregates having large pore volumes and sizes (high permeabilities) would probably be more amenable to coating treatments. Chert is an example of the first type and vesicular lava (andesite) an example of the second. In the case of vesicular lava, impregnation might not even be economically practical because of large porosities (as high as 40 percent), but blocking of the pores at the surface with a viscous coating material might be feasible.

TEST AGGREGATES

One aggregate source was selected to represent each

of the two failure modes (and, therefore, treatment processes) described above. The criteria used to select the two aggregate sources were (a) the frost susceptibility of the aggregate (the most deleterious aggregates received the highest priority), (b) the geographic area of the country (aggregates from areas that lack high-quality aggregates were preferred), (c) the degree of documentation of the performance of the aggregate (those aggregates on which the most documentation was available were favored), and (d) the availability of the aggregate.

Although the rationale behind the selection criteria is generally obvious, it should be mentioned that the decision to seek out the most deleterious aggregates for this test program was based on the premise that, if such aggregates could be upgraded, there should be no problem in upgrading less seriously affected aggregates. There are two difficulties in using "marginal" aggregates: The laboratory tests frequently lack sufficient discrimination to be of much use in showing improvements in marginal aggregates, and the applicability of a solution to a problem involving a marginal aggregate is much less general (i.e., the solution works for the aggregate tested and for better ones, but it may not work for worse aggregates).

Twenty-nine sources of documented frost-susceptible aggregates were identified in a detailed literature review. Based on the selection criteria given above, a chert gravel from Kentucky was selected for the impregnation tests and a hornblende andesite gravel from California for the coating tests.

According to the results of tests in previous research (5,6), concrete specimens containing these aggregates in the saturated state failed in only one or two freezing and thawing cycles in the laboratory (ASTM C671). The chert gravel was reported to display very low permeability (10^{-3} - 10^{-5} darcys, averaging 10^{-6}) and moderate effective porosity (3-12 percent, averaging 8 percent). The andesite gravel reportedly had permeability values that averaged 1000 times that of the chert (10^{-10} - 10^{-12} darcys, averaging 10^{-11}). In addition, the porosity of the andesite was appreciably greater than that of the chert (12-43 percent, averaging 28 percent). Petrographic descriptions of the two selected test aggregates, as abstracted from the references, are given below.

Chert River Gravel

Chert river gravel is an angular, yellow-brown and white, massive to laminated chert consisting mostly of microcrystalline quartz [average grain size 0.006 mm (0.0002 in) in diameter] and 0-20 percent fossil fragments composed of quartz grains with an average diameter of 0.06 mm (0.002 in). Most samples contain banded iron-stained chalcedony, which occurs as irregularly shaped masses as large as 1 cm (0.39 in) diameter or as interstitial cement, both commonly with quartz overgrowths. Most samples have a smooth weathered coating of limonite and chalcedony-rich chert. All contain as much as 9 percent dolomoids [average diameter of 0.5 mm (0.019

in]], most of which are disseminated but commonly occur in continuous layers. Many of these rhomb-shaped cavities are filled or partly filled with limonite. All samples, especially the chalcedony, contain numerous microfractures that appear to be incipient desiccation fractures. Many of the dolomoids are interconnected by these microfractures. The specific gravity of individual particles varies inversely with the percentage of dolomoids present.

Hornblende Andesite Gravel

Hornblende andesite gravel is somewhat variable in color, ranging from pale gray through dark gray, purplish gray, and grayish black. The structure varies from highly vesicular (pumiceous), crumbly, brittle, and fractured to massive and structureless. Occasionally, flow banding is exhibited and the aggregate is generally rich in hornblende phenocrysts, sometimes oxidized. Particle shapes vary from angular or subangular for the massive material to subrounded or rounded for the more vesicular material.

TREATMENT MATERIALS

The treatment materials chosen for coating or impregnating the test aggregates were selected to represent classes of potential coatings and impregnants. Given the preliminary nature of this study, no attempt was made to optimize the selection of treatment materials within the various classes with regard to economic or practical feasibility. However, a high-volume waste material (sulfur) was included as an impregnant in view of its obvious economic advantages. As a coating material, epoxy was selected to represent the most durable polymer type of material that would be expected, in general, to seal pore entrances on the aggregate particles. Linseed oil emulsion was selected to represent a class of coating materials that is intended to coat pore surfaces and reduce pore moisture by increasing the contact angle. The impregnants chosen and the classifications that they represent are as follows:

1. Boiled linseed oil--self-polymerized (oxidation) polymeric material,
2. Epoxy (diluted with organic solvent to reduce viscosity)--condensation-type polymeric material,
3. Methyl methacrylate--addition-type polymer,
4. Polyethylene glycol--water-miscible polymer of high molecular weight, and
5. Sulfur--a large-volume waste material.

Preliminary tests were carried out with some of the treatment materials to examine the effects of the treatments on the water-absorption properties of the aggregates. The results are given below:

<u>Treatment</u>	<u>Reduction in Water Absorption (%)</u>	
	<u>Chert</u>	<u>Andesite</u>
Epoxy coating		86
Linseed oil emulsion coating		60
Epoxy impregnation	76	
Methyl methacrylate impregnation	59	
Boiled linseed oil impregnation	46	

Tests were also carried out to determine the uptake of most of the treatment materials by the test aggregates. These results were as follows:

<u>Treatment</u>	<u>Treatment Uptake (percent by weight)</u>	
	<u>Chert</u>	<u>Andesite</u>
Epoxy coating		8.9
Linseed oil emulsion coating		3.8
Epoxy impregnation	2.8	
Methyl methacrylate impregnation	2.4	
Boiled linseed oil impregnation	1.0	
Polyethylene glycol impregnation	6.2	
Sulfur impregnation	4.0	

EXPERIMENTAL PROCEDURES

The test aggregates were graded into four size groups and recombined to provide a standard gradation, as given below (1 mm = 0.039 in):

<u>Sieve Size Range (mm)</u>	<u>Percent by Weight</u>
19.0-25.0	25
12.5-19.0	25
9.5-12.5	25
4.75-9.5	25

Specific gravities, absorptions, and dry rodded unit weights were determined for each aggregate, before and after treatment, to provide mixture design data. Details on the treatment techniques are given in Table 1.

The portland cement used in this research came from a single production lot of low-alkali (0.51 percent Na_2O equivalent) type 1 cement. The fine aggregate was a high-quality alluvial quartz sand. The concrete mixtures were proportioned to give slumps in the range of 5-7.5 cm (2-3 in), air contents of 5 ± 1 percent, and a water-cement ratio of 0.45 by weight.

The specimens used in this test were cylinders 7.5 cm (3 in) in diameter by 15 cm (6 in) high that had cast-in-place gage studs at each end. Each aggregate-treatment combination was represented by three replicate specimens. Specimens that contained untreated aggregates were included for control purposes.

The freezing and thawing tests were carried out in accordance with ASTM C671 and C682, except for the following:

1. Aggregate conditioning prior to mixing consisted of soaking for 24 h.
2. Specimen conditioning consisted of soaking in water at 1.7°C (35°F) for three weeks after the prescribed curing period (1 day in molds, 13 days in limewater).
3. Freeze-thaw cycling was continued for each specimen until either the dilation exceeded $25 \mu\text{m}$ ($1000 \mu\text{in}$) or 12 cycles were completed, whichever came first (test cycle once every two weeks).

Changes in specimen length during freezing were monitored by using linear variable differential transformers, and the expansions (dilations) that occurred were measured later from recorder charts. It can be shown that a dilation of approximately 75 microstrain represents the elastic limit of a typical PCC (6). For the 15-cm (6-in) long specimens used in this research, this represents a critical dilation of about $11.25 \mu\text{m}$ ($440 \mu\text{in}$). Therefore, it has been assumed here that dilation values that exceed this critical value are indicative of failure of the concrete as a result of freezing and thawing action.

RESULTS

The test results for the chert aggregate are summarized in Figure 1. Each curve shown represents the average for the three specimens tested for each treatment material. As usual in this test, the dilation values below the critical dilation tended to be somewhat erratic but, once the critical

dilation was exceeded, subsequent dilation values tended to increase exponentially. It is evident from Figure 1 that all of the impregnants except sulfur (which failed in the 11th cycle) were effective in preventing freezing and thawing damage. It should be noted that the control (untreated) specimens failed early in the test (third cycle).

Table 1. Coating and impregnation techniques.

Treatment	Formulation of Treatment Material	Treatment Technique ^a
Epoxy coating	DGEBA, TETA (50 percent stoichiometric)	Aggregate removed from the oven, placed in epoxy mixture for 5 min while still hot, then placed on screens to polymerize
Linseed oil emulsion coating	100 percent LOE	Aggregate placed in LOE for 5 min (room temperature), then placed on screen for curing (7 days)
Epoxy impregnation	DGEBA:TETA: xylene (100:14.1:25 parts by weight)	Aggregate soaked in epoxy mixture for 1.75-2 h; treated aggregate spread out on screen for room-temperature polymerization
Methyl methacrylate impregnation	MMA:TMPTMA:AZO (100:10:0.5 parts by weight)	Aggregate soaked in MMA mixture for 4 h at room temperature; treated aggregate placed in 75°C hot-water bath for 24 h to polymerize MMA system
Boiled linseed oil impregnation	100 percent BLO	Hot aggregate (105°C) soaked in BLO; aggregate allowed to cool to room temperature during impregnation (24 h); treated aggregate placed in 105°C forced-draft oven for about 24 h to accelerate polymerization of BLO
Polyethylene glycol impregnation	100 percent PEG	Aggregate placed in 60°C oven for 24 h while soaking in PEG, then placed on screen to cool
Sulfur impregnation	95 percent sulfur, 5 percent tar	Aggregates heated to 121°C placed in 121°C sulfur mixture for 30 min, then placed on screen to cool

Notes: $t^{\circ}C = (t^{\circ}F - 32)/1.8$.

DGEBA = diglycidyl ether of bisphenol A, TETA = triethylenetetramine, LOE = linseed oil emulsion, MMA = methyl-methacrylate, TMPTMA = trimethylol propane trimethacrylate, 2,2'-azobisisobutyronitrile (AZO), BLO = boiled linseed oil, PEG = polyethylene glycol.

^aAll aggregates were oven dried in a forced-draft oven at 105°C prior to receiving treatments.

Figure 1. Dilations in freezing and thawing cycles for concrete containing chert aggregate.

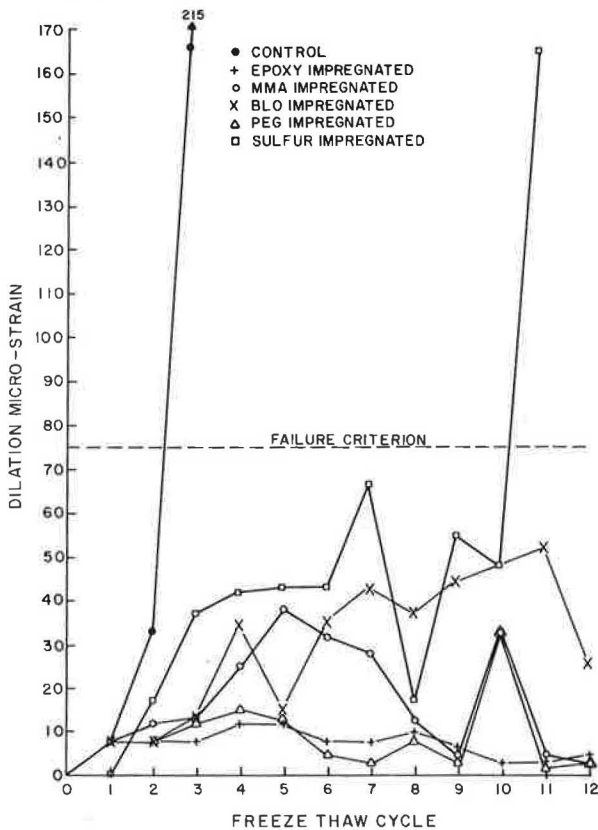
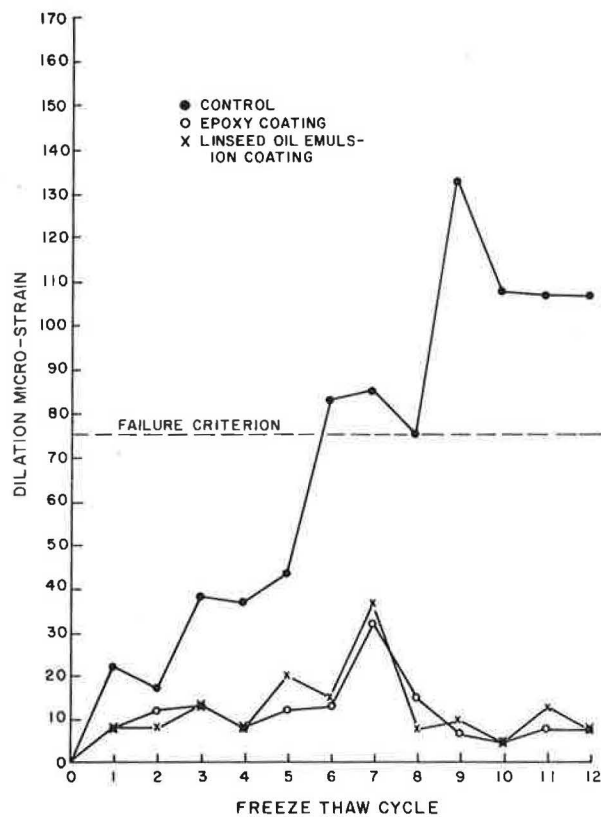


Figure 2. Dilations in freezing and thawing cycles for concrete containing andesite aggregate.



The test results for the andesite aggregate are shown in Figure 2. Again, each curve shown represents the average of three specimens. The tests demonstrate that both coating treatments tested were effective in preventing freezing and thawing damage. The control (untreated) specimen failed in six cycles of freezing and thawing.

CONCLUSIONS

Epoxy and linseed oil emulsion coatings and epoxy, methyl methacrylate, boiled linseed oil, and polyethylene glycol impregnants were all found to be eminently successful in upgrading highly frost-susceptible aggregates. Even sulfur, the only impregnant that did not meet the failure criterion, improved the frost resistance of these aggregates by a factor of five. The high degree of success in preventing freeze-thaw damage is considered to be a matter of no small consequence. Frost susceptibility is generally considered to be the most common characteristic limiting the use of aggregate materials in PCC. In addition, the results suggest that there exists a wide range of coating or impregnation materials that are capable of eliminating the frost susceptibility of aggregates.

ACKNOWLEDGMENT

The work reported in this paper was sponsored by the American Association of State Highway and

Transportation Officials, in cooperation with the Federal Highway Administration, and was conducted as part of the National Cooperative Highway Research Program.

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

State of the Art in Use of Nuclear Density Gages on Portland Cement Concrete

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Proper consolidation during construction improves all of the important properties of portland cement concrete. The state of the art in state highway department use of commercially available, static nuclear density gages to monitor concrete consolidation is discussed based on the results of a 1977 Federal Highway Administration survey of state highway departments. The survey showed rapidly growing use of the gages for controlling the density of thin bridge-deck overlays, particularly of low-water-cement-ratio (0.32), low-slump (Iowa) concretes. Some use during construction of full-depth bridge decks and of pavements was also reported. A discussion of the problems involved in the use of nuclear gages—e.g., the effect of reinforcing steel on gage response and the selection of appropriate density standards—is also included.

Late in 1977, the Federal Highway Administration (FHWA) surveyed state highway departments regarding their use of nuclear gages to monitor the in-place density of fresh (plastic) portland cement concrete (PCC). This paper summarizes the responses to the survey and discusses various procedures and recommendations for future research. Portions of individual state responses are given in Table 1.

CURRENT STATE USE OF NUCLEAR GAGES

Eleven states reported extensive use of nuclear gages at the time of the survey, and another 16 reported current evaluation studies or plans for such studies in the immediate future. The responses

showed that attention is being focused on use of the gages to control the density of thin bridge-deck overlays, particularly of concretes with low water-cement (w/c) ratio and low slump (Iowa concretes). Some use of nuclear procedures during construction of full-depth bridge decks and of pavements was also reported.

CONSTRUCTION OF BRIDGE-DECK OVERLAYS

Mode of Gage Operation

The first step in the development of a procedure for monitoring overlay densities is the choice of the operating mode for the nuclear gage—either direct transmission or backscatter. The states that use the gages extensively on thin overlays are divided almost evenly between the two modes. Neither method is clearly superior, and each shows certain disadvantages.

Direct Transmission

In direct transmission, the probe containing the radioisotope source is immersed in the concrete. The commercial gages normally allow the probe to be inserted to set depths that vary from 50 to as much as 300 mm (2-12 in). Use of the 50-mm setting on

Table 1. Responses to survey of state highway departments.

FHWA Region	State	Extent of Use				Application			Operating Mode			State Test Method Available	Density Standard				Calibration			
		A	B	C	D	E	F	G	H	I	J		K	L	M	N	O	P	Q	
1	New Hampshire			X		X				X			X				X		X	
3	New York				X	X			X			X	X				X		X	
	Pennsylvania	X																		
	Virginia			X		X		X		X								X		
	West Virginia			X		X	X		X				X				X		X	
4	Alabama		X						X	X		X	X					X	X	
	Georgia	X							X					X					X	
	Kentucky				X	X					X	X			X				X	
	Mississippi	X							X	X									X	
	North Carolina			X		X				X		X	X						X	
	Tennessee			X		X			X	X		X	X						X	
5	Illinois			X		X				X		X		X			X		X	
	Indiana			X	X	X		X		X		X	X					X	X	
	Michigan			X	X	X		X	X		X	X	X			X		X	X	
	Minnesota			X	X	X	X		X		X		X						X	
	Ohio			X	X	X			X		X					X			X	
	Wisconsin			X	X	X		X		X	X	X	X					X	X	
6	Louisiana			X		X		X	X	X		X	X						X	
	New Mexico			X		X		X		X		X	X						X	
	Oklahoma			X	X	X		X		X		X	X				X		X	
7	Iowa			X	X	X		X		X		X	X						X	
	Kansas			X	X	X	X	X	X	X		X	X				X		X	
	Missouri			X		X		X		X		X	X						X	
	Nebraska			X	X	X		X		X		X	X						X	
8	Colorado			X	X	X	X	X	X	X		X	X	X	X				X	
	Montana			X		X		X		X		X	X						X	
	Wyoming		X			X													X	
9	California	X				X	X		X		X		X						X	
	Nevada			X	X	X		X		X		X	X				X		X	
10	Idaho			X		X		X	X		X		X					X	X	
	Oregon			X		X		X	X		X		X	X			X		X	
	Total number	4	2	14	11	26	6	10	18	13	4	17		22	4	2	2	8	19	17

Note: A = past, B = planned, C = current experimental, D = extensive; E = overlays, F = full-depth decks, G = pavements; H = direct transmission, I = backscatter, J = backscatter air gap; K = rodded unit weight, L = vibrated unit weight, M = theoretical unit weight, N = vibrated unit weight in box; O = use gage manufacturer's curves, P = use own calibration blocks, Q = correction with concrete on each project.

50-mm overlays is marginal, and on thinner overlays it is impossible unless special procedures are adopted. Gages have been modified by some users to create depth positions of 38 mm (1.5 in) and smaller, but the resulting data are susceptible to a variety of errors, and therefore such modifications should not be introduced without careful study.

Currently, the best direct-transmission procedure is to place test wells in the underlying concrete by removing material 25-50 mm (1-2 in) down over a small area before placing the overlay. It is then possible to use the 50-mm setting on the nuclear gage and properly run a direct-transmission test. This method has the disadvantage of identifying the test site prior to placement of the overlay.

Keeping gages clean is also a problem with the repeated insertion of the probe into the fresh concrete. This problem will be addressed later in this paper.

Backscatter

Effect of Existing Deck

In the backscatter mode, the source remains inside the main body of the nuclear gage, which in turn is seated on the concrete surface. On thin overlays, the principal problem is the effect of the underlying concrete--i.e., the existing deck--on gage response to the overlay density. Backscatter gages typically register 70 percent of their response from the top 38 mm (1.5 in) of a sample and 80-85 percent from the top 50 mm (2 in). With this sensitivity profile, if 38 mm of a 2560-kg/m³ (160-lb/ft³) overlay is placed and fully consolidated on an existing 2320-kg/m³

(145-lb/ft³) deck, a backscatter nuclear gage would read less than 2500 kg/m³ (156 lb/ft³).

Current state practices for correcting the density reading for the underlying material range from ignoring the effect to using correction factor tables based on nuclear readings on both the original and the overlaid deck and on the thickness of the overlay. At least one gage manufacturer has published an application note that outlines a correction procedure for his latest gage model (1). Because of the potential impact of this effect when the overlay and existing densities differ significantly, users of the backscatter procedures need to consider correcting gage results on a project-by-project basis.

Effect of Chemical Composition of Aggregate

Backscatter methods generally have a higher sensitivity to the chemical composition of the sample than do direct-transmission methods. Thus, two concretes, one containing siliceous aggregate and the other calcareous aggregate, can give different nuclear-gage readings even though they are consolidated to the same density. These differences have been reduced considerably in the latest generation of nuclear gages and are also not a problem in the few states that use air-gap procedures. (Air-gap procedures require two nuclear readings, one with the gage on the surface and the other with a small gap between the gage and the concrete. The ratio of these two counts is insensitive to the chemical composition of the sample.)

In general, the state users do not have problems with chemical composition errors because their test

procedures require development of a correction factor for their gages on each project. The correction factors are developed from comparisons of nuclear gage densities with densities established by weighing a fixed volume of concrete, typically in a 600x600x150-mm (24x24x6-in) box. The difference between the nuclear and weighed densities is then applied to all future nuclear readings on the same project. (Some states use a similar procedure to correct transmission-mode measurements.)

The validity of such correction procedures depends on the accuracy of the weighed-density determination and the uniformity of consolidation in the box. An accurate weighed-density determination depends, in turn, on an accurate measurement of the volume of the concrete. Several states reported problems with the procedures for developing correction factors, and at least one--Iowa--has eliminated the plastic concrete correction because of the difficulty of obtaining satisfactory correction factors in the field (and because of the lack of significant variation in chemical composition of the state's aggregate supplies). However, based on user experience, the establishment of a correction factor on each project is recommended.

Effect of Reinforcing Steel

Colorado and California both provided data showing that reinforcing steel has a significant effect on gage response in the backscatter mode if the device is placed directly over a steel bar. The depth of concrete cover at which this effect becomes significant depends on the gage model and the source position. Colorado (2) reported that some gage models showed no influence of the reinforcing steel with only 38 mm (1.5 in) of cover whereas others showed significant effects when the steel had 64 mm (2.5 in) of cover. In the field, then, this effect may be significant on overlay decks because the reinforcing steel may be at or near the surface of the original (underlying) deck concrete.

As an example of the size of the steel effect, California reported that concrete that gave a nuclear-gage reading of 2395 kg/m³ (149.5 lb/ft³) with 100 mm (4 in) of cover over reinforcing steel gave readings of 2419 and 2480 kg/m³ (151 and 154.8 lb/ft³) when the cover decreased to 64 and 38 mm (2.5 and 1.5 in), respectively. Taking a backscatter reading directly over the reinforcing steel can thus produce a misleadingly high density value and indicate adequate consolidation when that is not the case.

Density Standards and Percentage Compaction Requirements

The second major choices the user faces in developing a procedure for monitoring overlay densities are the selection of a test method for establishing the density standard and the selection of a percentage compaction requirement based on that standard.

The test method should be a reproducible and accurate procedure for determining the maximum density the concrete can be expected to attain in place. More than 70 percent of the users rely on rodded unit weight (AASHTO T121) as the density standard, but several expressed dissatisfaction with the method, particularly when it is used on low-w/c-ratio (Iowa) mixes. The problem is that samples are not always fully consolidated in the unit weight bucket, and low densities are obtained. As a result, reports from the field showing relative compactions from 100 to as much as 115 percent are

common. Vibrated and theoretical unit weights have also been considered as standards by a few states, but the suitability of either has not been demonstrated.

In addition to selecting the method for establishing the density standard, the user must set a compaction requirement as a percentage of the standard. The most common requirement is that the nuclear-gage reading be a minimum of 98 percent of the rodded unit weight. This value is apparently based on corrosion studies (3) that indicate that Iowa overlays (w/c ratio of 0.32) consolidated to 98 percent show very low permeabilities to chloride ions in long-term ponding studies. At 92-93 percent compaction, on the other hand, Iowa-type concretes proved to be very permeable. The validity of the 98 percent requirement obviously depends on obtaining an accurate rodded unit weight. Research is still needed to find a reproducible, foolproof, and reliable procedure for establishing the density standard and then to find what percentage of that standard is required in order to ensure an impermeable and durable overlay.

CONSTRUCTION OF FULL-DEPTH BRIDGE DECKS AND PAVEMENTS

According to the questionnaire responses, only a few states are currently using nuclear gages to monitor the densities of full-depth bridge decks and pavements. A recent slide package (4), prepared jointly by FHWA and the Colorado Department of Transportation, offers a good introduction to nuclear testing for density control of concrete pavement. For the future, the consolidation monitoring device, a nuclear backscatter gage that is mounted on a slipform paver, offers the possibility of monitoring the density of large volumes of new concrete during paving (5).

Much of the discussion in the previous section on overlays is relevant to the application of nuclear gages to full-depth slabs and decks. The user is faced with the same questions--e.g., direct transmission versus backscatter and choice of density standard--and many of the same options. The only additional problem is that of the effect of reinforcing steel on direct-transmission operations. Colorado is the only state that currently has a nuclear-gage-based specification for pavements. That state uses direct-transmission measurements to enforce a requirement that the concrete be vibrated to not less than 96 percent of the maximum theoretical field density. This percentage was established from a research study (6) that showed that cores from pavements with poor abrasion records had densities less than 97 percent of rodded unit weight.

The direct-transmission mode is easier to apply to full-depth slabs than to thin overlays. However, care must be taken to avoid interference with the gage response from the reinforcing steel. The critical distance from reinforcing mesh or rod has not been established. As a result most state procedures include a general directive to establish the test location "to avoid being near the steel". FHWA Rapid-Test Procedure RT-13 (7) recommends that the center of the test site be 200 mm (8 in) laterally from any steel unless the concrete cover is greater than the depth of the source probe. The 200-mm lateral distance was based primarily on the results of a 1973 California study (8) on how close nuclear-gage readings can be taken to adjacent structures. The instruction manual for the Troxler 3400 series gages says that "the back of the instrument (should be) no closer than 150 mm (6 in) to an obstruction" (9). A recent study (10)

suggests that the source probe may be placed up against dowel bars without affecting the gage response as long as the steel does not lie in the sensitive ellipsoidal volume between the source and the detector.

Steel interferences can be significant: As in the case of backscatter measurements, they can produce high-density readings even though adequate consolidation has not been achieved. For example, Colorado (2) reported that 50 mm (2 in) direct transmission measurements through concrete with reinforcing steel at 38 mm (1.5 in) depth showed errors in density determinations of 106 kg/m³ (6.6 lb/ft³). Until more specific data on critical distances from reinforcing steel are obtained, a 150 to 200-mm (6- to 8-in) separation between the center of the test site and the nearest steel appears sufficiently conservative (when the cover over the steel is less than the depth of the source probe). In heavily reinforced decks where such separations are not possible, care should be taken to keep the steel out of the sensitive volume between the source and the detector.

OTHER CONSIDERATIONS

Accuracy

There are few data in the literature on the accuracy of nuclear density measurements on concrete (accuracy is defined here as how closely the nuclear-gage-determined density matches the true density). Finding a suitable standard of comparison--i.e., the "true" density--is a problem; for example, core densities, a common standard, are based on different volumes of concrete than nuclear gage densities established at the same physical locations. When core densities were used as the basis for comparisons, one study (5) reported the standard error for backscatter-mode measurements to be ± 30 kg/m³ (± 1.9 lb/ft³); that is, there is a 95 percent probability that a nuclear-gage-determined density is within ± 60 kg/m³ (± 3.8 lb/ft³) of a core density at that location. In another pavement study (11), Colorado researchers compared direct-transmission test results with the densities of the same areas established from averages of core samples, beam samples, and the nuclear results. They reported the overall standard error to be ± 21 kg/m³ (± 1.3 lb/ft³).

Because concrete is relatively homogeneous compared with soils and soil-aggregates and because surface roughness errors on plastic concrete are minimal, the user can expect the accuracy of nuclear density determinations on concrete to be at least as good as, and probably better than, the accuracy expected on soil materials.

Calibration

Making absolute density determinations on concrete requires a calibration curve of nuclear gage count versus density. Most state agencies now have standards in their central and/or district laboratories for establishing their own calibration curves. In addition, the questionnaire responses showed that, on concrete, most states use a correction factor developed on each project. As discussed previously, nuclear gage readings are taken on a box of plastic, well-consolidated concrete (sizes range from 300x450x100 mm to 600x600x150 mm (12x18x4 in to 24x24x6 in), and the nuclear gage density is then determined from the standard calibration curves. The true density is also established by dividing the weight of the

concrete-filled box by its previously established volume. The difference between the nuclear gage density and the true density becomes a correction factor that is applied to all subsequent nuclear gage readings on the same project. The Iowa Department of Transportation originally used this technique but now establishes the correction factor only on standard blocks of granite or hardened concrete. This approach eliminates errors in establishing the true density on each project but may introduce slight additional chemical-composition errors if project materials differ significantly from the standard block.

A few states rely on their standard calibration curves without project-by-project corrections. However, if the true, weighed density is carefully established, development and use of a correction factor on each project should provide the most accurate density determinations. (It should be noted that several states reported problems with volume changes in the calibration box when it was filled with concrete.)

Radiation Safety

Safety practice for the use of nuclear gages on concrete is, with one exception, the same as that for their use on soils, soil aggregates, and bituminous pavements. The one additional problem is caused by the adherence of mortar to the probe in the direct-transmission mode: Repeated retraction of the probe tends to clog the wiper ring and critical parts on the bottom of the gage. Gage manufacturers state that the wiper ring is adequate to remove most of the mortar but recommend that the other easily accessible parts on the gage bottom be cleaned at the end of each day.

Most of the agencies that use direct-transmission measurements responded that their procedures include a wiping of the probe after each test. The dose rate at 10 mm (0.4 in) from an 8-millicurie cesium-137 source in a typical probe is 25 rem/h; one hundred 15-s wipings of the probe per week at this radiation level would yield a total exposure of about 500 millirem to the operator's hands. This is approximately one-third of the currently allowable dose to the hands. However, such an exposure is unnecessary and should be avoided. Most of the state test procedures prohibit direct hand contact with the probe but are unclear as to the actual wiping procedure allowed. Use of a squirt bottle and/or a sponge or rag held in a pair of tongs should minimize the operator's dose. Two states reported trying copper and plastic tubing to protect the probe while it is in the concrete, but results of those trials are not available.

ACKNOWLEDGMENT

I gratefully acknowledge the assistance of Frank Botelho of FHWA's Office of Highway Operations for carrying out the survey and of the many individuals in the state highway departments who took the time to respond to the questionnaire.

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Publication of this paper sponsored by Committee on Batching, Mixing, Placing, and Curing of Concrete.