NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE **57**

DURABILITY OF CONCRETE BRIDGE DECKS

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

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RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS IN COOPERATION WITH THE FEDERAL HIGHWAY ADMINISTRATION

AREAS OF INTEREST:
- STRUCTURES DESIGN AND PERFORMANCE
- CEMENT AND CONCRETE
- CONSTRUCTION
- MAINTENANCE
- (HIGHWAY TRANSPORTATION)

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. MAY 1979

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are responsibilities of the Academy and its Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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PREFACE

There exists a vast storehouse of information relating to nearly every subject of concern to highway administrators and engineers. Much of it resulted from research and much from successful application of the engineering ideas of men faced with problems in their day-to-day work. Because there has been a lack of systematic means for bringing such useful information together and making it available to the entire highway fraternity, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize the useful knowledge from all possible sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series attempts to report on the various practices, making specific recommendations where appropriate but without the detailed directions usually found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems. The extent to which they are utilized in this fashion will quite logically be tempered by the breadth of the user's knowledge in the particular problem area.

FOREWORD

By Staff
Transportation Research Board

This synthesis will be of special interest and usefulness to bridge engineers and others seeking information on design, construction, and maintenance of bridge decks. Detailed information is presented on the causes, prevention, evaluation, and rehabilitation of deck deterioration related to corrosion of steel reinforcement.

Administrators, engineers, and researchers are faced continually with many highway problems on which much information already exists either in documented form or in terms of undocumented experience and practice. Unfortunately, this information often is fragmented, scattered, and unevaluated. As a consequence, full information on what has been learned about a problem frequently is not assembled in seeking a solution. Costly research findings may go unused, valuable experience may be overlooked, and due consideration may not be given to recommended practices for solving or alleviating the problem. In an effort to correct this situation, a continuing NCHRP project, carried out by the Transportation Research Board as the research agency, has the objective of synthesizing and reporting on common highway problems. Syntheses from this endeavor constitute an NCHRP report series that collects and assembles the various forms of information into single concise documents pertaining to specific highway problems or sets of closely related problems.

NCHRP Synthesis 4, "Concrete Bridge Deck Durability," published in 1970, is still an excellent source of information on the nature of scaling and spalling and the causes of these mechanisms. However, engineers continue to search for methods of obtaining durable bridge decks, and the problem is becoming more urgent. It was reported in 1978 that nearly one-third of all highway bridge decks in the United States are seriously deteriorated due to corrosion of reinforcing steel. The cost of restoring these decks has been estimated at $6.3 billion.

Virtually all protective systems currently in use to prevent corrosion of bridge deck reinforcing steel were developed following publication of *Synthesis* 4. In addition, methods of evaluation and rehabilitation have changed significantly since then. The current synthesis is, therefore, intended to supplement the earlier effort. This report of the Transportation Research Board reviews design and construction techniques currently in use to prevent deterioration of new bridge decks and also evaluation and rehabilitation techniques used to extend the service life of existing decks. Recommendations are included for research needs related to bridge deck durability.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the researchers in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis is an immediately useful document that records practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As the processes of advancement continue, new knowledge can be expected to be added to that now at hand.

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Information on current practice was provided by many highway and transportation agencies. Their cooperation and assistance was most helpful.

DURABILITY OF CONCRETE BRIDGE DECKS

SUMMARY

Over the years, bridge deck durability has continued to be a problem, especially because of deterioration resulting from corrosion of embedded reinforcing bars. This synthesis reviews recent developments in condition evaluation methods and protective systems for new and existing bridge decks.

Evaluation of an existing deck begins with a condition survey. This may range from a quick visual inspection to a detailed survey requiring many hours of inspection, testing, and analysis. The visual examination by a skilled inspector should report locations and descriptions of all spalling, scaling, and cracking on both top and bottom surfaces of the deck. The detailed survey may incorporate several of the available physical tests and survey methods.

For new construction, careful design, proper selection of materials, and good construction practices are essential to the achievement of a durable bridge deck. Among design practices that improve durability are lesser skews, better drainage, thicker slabs, and greater reinforcement cover. Construction practices that contribute to durability include achievement of the specified cover, use of concrete with the lowest possible water-cement ratio, and good consolidation. Protective coatings on the reinforcing steel reduce susceptibility to corrosion. The most effective coating is fusion bonded epoxy powder; zinc (galvanizing) has also been widely used, but there are conflicting reports of its effectiveness.

For either new construction or as a repair technique, sealants, impregnants, overlays, membranes, or cathodic protection have been used to improve durability. Sealants are not effective in preventing corrosion damage. Polymer impregnation of bridge decks shows promise and research is continuing. Concrete overlays may be applied as the second stage of new deck construction or as preventive maintenance on an existing deck. The overlay may be low-slump concrete (the "Iowa method"), latex-modified concrete, or internally-sealed concrete (wax beads). Membranes are available in a variety of systems, but field experience has been highly variable and there is some doubt as to long-term performance. Cathodic protection has been used successfully to stop active corrosion; it is the only practical method to ensure this.

Repair of deteriorated decks is a complex process. Within the constraints of budget, work force, traffic control, and weather, the most cost-effective treatment must be chosen. The most difficult task is determining how much concrete should be removed; estimating errors can be minimized by a thorough condition survey as close as possible to the time the work is done. Patching of a deteriorated deck, no matter which material is used, is seldom more than a temporary measure to restore riding quality. Epoxy injection of delaminated areas can be a cost-effective method of extending the life of a deck until permanent repairs are made. Chloride removal through electrochemical means is under development; it appears to stop active corrosion but more work is needed to make the process practical and economical.

Research is needed in the areas of fundamental studies, test methods, materials development, construction practices, and repair practices and methodology. Among the specific needs are: defining the conditions under which reinforcement corrodes;

determining the role of concrete quality and cover in spalling; monitoring the long-term performance of deck protective systems; and developing test methods to measure corrosion rate nondestructively, oxygen concentration in concrete, and permeability of concrete. Other research needs include development of corrosion inhibitors; improvements to existing materials; better methods of setting reinforcing steel and placing concrete to assure achievement of the design cover; refinement of polymer-impregnation techniques; improved methods of removing chlorides; and means of rehabilitating decks with active cracks.

CHAPTER ONE

BACKGROUND

SCOPE

The first synthesis report on bridge deck durability was published in 1970 (*1*). It covered in depth the pioneering bridge deck investigations of the 1960s, the processes of deterioration, the consequences of design and construction methods, the effects of materials and the environment, and the solutions in practice at the time.

In the intervening years, the premature deterioration of concrete bridge decks has continued to be a major problem for highway agencies. There have, however, been significant developments in the methods used to evaluate the condition of existing decks and in the protective systems in use on both new and existing structures. This synthesis has been prepared to encompass these developments and to indicate the most promising practices for satisfying the requirements of bridge deck durability.

This synthesis has been written primarily for the highway design, construction, materials, and maintenance engineer having operational responsibility. The practical aspects of bridge deck condition surveys and the solutions that have been developed and implemented since 1970 are emphasized. Presentation of the theoretical background has been limited to that which is necessary to understand the mechanisms of deck deterioration and appreciate the limitations of the test procedures and the protective systems. For more detailed information, the reader should consult the references listed in this report.

EXTENT OF THE PROBLEM

Bridge deck deterioration is not a new phenomenon. There have always been decks that have cracked or developed defects because of environmental effects such as frost action, the use of inferior materials, or poor workmanship. What has changed the perspective on bridge deck deterioration is the extent of the problem and the vast sums of money required to maintain the existing highway network. For many years the causes of bridge deck deterioration were not clearly identified. Exhaustive studies of the effects of stress, materials, and methods of construction were conducted (*2*) before it was established that the primary problem is corrosion of the reinforcing steel. The widespread occurrence of corrosion in bridge decks, which is a direct result of using deicing salts in winter maintenance operations, has added a new dimension to the problem because of the consequences of this type of damage. Not only does corrosion destroy the smooth riding quality of the deck, it may eventually reduce the structural integrity and safety of the deck slab, and it is very difficult to make permanent repairs.

There are approximately 564,000 bridges in the United States, of which about 235,000 are on the federal-aid system. Approximately 39,900 structures on the federal-aid system and at least 65,600 state and county bridges need replacement or repair. The funds required to undertake such a program are staggering: Conservative estimates are $12.4 billion on the federal-aid system and $10.6 billion for the off-system bridges (*3*). It has been said that the transportation system is deteriorating faster than it is being constructed and maintained (*4*).

Many of the bridges identified as being structurally deficient or functionally obsolete are older structures. However, there is equal concern for the premature deterioration of newer structures. The bare pavement policy adopted by many states in the early and mid-1960s coincided with the rapid expansion of the Interstate network. Consequently, the number of structures increased substantially because of interchanges and grade separations with secondary highways. Many of these structures are large, and many are located in urban areas where traffic densities make maintenance operations difficult. The majority of these new structures were designed and built to specifications which have subsequently proven to be inadequate. Decks only a few years old are showing signs of distress. The estimated cost of repaving and upgrading structures on the Interstate System alone is in excess of $2 billion (*5*).

Prospects for the immediate future are no brighter. In more than 75 percent of the states, less than 10 percent of the federal-aid bridges have been built with deck protective systems. It is estimated that by 1980 the proportion of protected decks in these states will increase to no more than 20 percent (*6*).

DEFINITIONS

Cover: The least distance between the surface of the reinforcement and the outer surface of the concrete (*7*). In this report, cover is usually discussed with reference to the uppermost reinforcing steel in the deck slab.

Corrosion: Degradation of a material by reaction with its environment. In this report, corrosion refers specifically to the electrochemical corrosion of reinforcing steel in concrete.

Delamination: A separation along a plane parallel to the outer surface of the concrete, generally located at the level of the reinforcing steel and caused by corrosion of the reinforcement.

Scaling: Local flaking or peeling away of the near-surface portion of hardened concrete or mortar (*7*) caused by frost action and aggravated by the presence of deicing chemicals.

Spall: A depression resulting from the separation and removal of the surface concrete and caused by the corrosion of embedded reinforcing steel.

The following terms differ from standard cement and concrete terminology but have been defined as follows to avoid ambiguity in this report.

Coating: A material applied to the surface of reinforcing bars to prevent corrosion of the steel.

Impregnant: A liquid applied to penetrate and fill the interstices of portland cement concrete using positive methods to assure a depth of penetration in excess of 0.25 in. (6 mm).

Membrane: A continuous sheet of material, either preformed or cured from a liquid, applied to a bridge deck surface and protected from the action of traffic by a wearing course.

Overlay: A layer of portland cement, bituminous, or polymer concrete applied to, and usually bonded to, the deck surface and exposed to the action of traffic.

Sealant: A liquid applied to the surface of portland cement concrete using only gravity or spray application. The liquid may cure to form a continuous film on the concrete surface or may seal the voids of the concrete to a depth not exceeding 0.25 in. (6 mm).

NATURE OF THE PROBLEM

The bridge deck environment is one of the worst imaginable exposure conditions for concrete. Engineers have known since the principles of concrete technology were first postulated that a vertical surface is more durable than a horizontal surface, that alternate wetting and drying is a more severe exposure than total submersion, and that freezing and thawing is more damaging than constant freezing. It also was recognized that in a marine environment a clear concrete cover of at least 3 in. (75 mm) was necessary to protect the steel against corrosion by salt water. Yet bridge decks in many parts of the country are subjected to frequent applications of deicing salts on a horizontal surface, alternate wetting and drying, and freezing and thawing. Despite the exposure, the specified concrete cover, until recently, was typically 1½ in. (38 mm).

The use of deicing salts increased dramatically in the 1960s and 1970s. Throughout the United States, 1.8 million tons (1.6 Tg) of salt were used in 1961, 3.1 million tons (2.8 Tg) in 1965, and more than 11 million tons (10 Tg) in 1975.

Bridge decks also are subjected to severe temperature gradients and high live-load stresses, including fatigue and impact. They contain a congestion of reinforcement, making the use of highly workable concrete essential. Because of finishing (often by hand) and bleeding, the worst quality of concrete in the deck is at the surface. Clearly, of all the elements in highway construction, bridge decks require special attention in all facets of design, materials selection, and construction.

Three conditions of bridge deck deterioration are commonly identified (*8*): cracking, scaling, and spalling [photographs showing examples of these defects are contained in a report prepared by a committee of the American Concrete Institute (*9*)]. In addition, two performance criteria need to be satisfied: adequate skid resistance and lack of wear, especially differential wear in the wheel tracks.

Cracking

Cracking is a characteristic of concrete because of its low tensile strength and the relatively large volume changes that occur in response to changes in humidity and temperature. The significance of cracks and their effect upon the durability of a deck is dependent on their origin. Cracks appearing at the time of construction due to shrinkage or settlement of the falsework are usually fine and, though undesirable, may not adversely affect the performance of the bridge deck. Conversely, map or pattern cracking resulting from the use of reactive aggregates may occur several years after construction, increase in magnitude and intensity, and eventually result in complete disintegration of the concrete. In such cases, replacement of the deck is usually the only solution. Structural cracks can also be troublesome, especially when there is significant crack movement, because this severely restricts the choice of repair method.

The common belief that cracks are necessary for widespread corrosion damage to occur is erroneous (*10*). Corrosion of reinforcing steel can occur in uncracked, high quality concrete if there is little cover for the reinforcing steel. It is generally acknowledged that cracks perpendicular to the reinforcing steel will hasten corrosion of intercepted bars by facilitating the ingress of moisture, oxygen, and chloride ions to the reinforcement. A reduction in the pH in a crack has been observed as a result of leaching by sodium chloride solution (*11*). Studies have shown, however, that narrow cracks [width less than 0.01 in. (0.3 mm)] have little influence on the over-all corrosion of the reinforcing steel (*12, 13*), and the corroded length of the rebar is likely to be no more than 3 bar diameters (*14*). Wider cracks accelerate the onset of corrosion, but over a period of several years, crack width has little effect on the amount of corrosion (*14*). The extent of the long-term damage to the bars is determined by other factors, primarily the depth and quality of concrete cover. Cracks that follow the line of a reinforcing bar are much more serious, because not only is the corroded length of the bar roughly equal to the length of the crack, but the crack reduces the resistance of the concrete to spalling.

Scaling

Scaling is the flaking of surface mortar, often accompanied by the loosening of surface aggregates. In cases of severe scaling, the mortar fraction of the concrete is completely broken down and loose aggregate can be scooped out by hand.

Scaling is the result of the frost deterioration of concrete. When concrete cools below the freezing point of water, there is an initial period of super-cooling, after which ice crystals form in the larger capillaries. Because water in cement paste is in the form of a weak alkali solution, the alkali content in the unfrozen portion of the solution in these capillaries increases (*10*). An osmotic pressure is created, and water migrates from unfrozen pores to the frozen cavities. The combination of dilative pressure due to ice accretion and osmotic pressure in the pores can cause mechanical damage or cracking in the paste.

Air entrainment is a well-proven method of minimizing and even avoiding freezing damage in cement paste. The large pores do not fill with water except after prolonged exposure to 100 percent relative humidity, and they empty on the slightest decrease below this value (*15*). Consequently the entrained air voids are available to act as reservoirs and compete with the larger capillaries for the water migrating from the smaller pores, thus shortening the migration path and the time of response. The resistance of concrete to frost damage can be improved not only through the provision of air entrainment but also by reducing porosity (and hence the water-cement ratio), because this reduces the amount of freezable water per unit volume of concrete.

The mechanism by which aggregate particles fail during freezing differs from that of cement paste in that water migrates away from and not toward the sites of ice accretion. Aggregate fails when the pores become critically saturated and pressures are developed because of resistance to the migration of water away from the regions of freezing.

Aggregate in concrete is not normally in a critical state of saturation at the end of the construction period because of the self-desiccation that occurs during hydration of the cement and loss by evaporation. Consequently, under most exposure conditions, the aggregate is unlikely to become critically saturated.

In the presence of deicing salts, air entrainment does not always give complete protection against scaling (*16*). The salt increases the degree of saturation of the concrete because the low vapor pressure of the salt solution makes it much more probable that the entrained air voids will fill with water and not be available to act as reservoirs. After it is nearly saturated, air-entrained concrete is very susceptible to frost damage because of its relatively high porosity and the large amounts of freezable water it contains. Exposed concrete deck slabs are unlikely to become saturated with water. Such a condition may exist on asphalt covered decks without an intervening membrane. Figure 1 shows severe scaling beneath the asphalt on an older, nonair entrained concrete deck slab.

Wear and Polishing

Except in isolated instances, notably urban freeways, wear on bridge decks has not caused serious deterioration, though it is a factor in determining the service life of a deck. Differential wear in the wheel tracks, usually where studded tires are permitted, has caused water to be ponded there, accelerating deterioration of the concrete and also decreasing the safety of the highway.

Even in the absence of studded tires, wear can cause a safety hazard because of aggregate polishing. Few states currently specify a minimum coefficient of friction. As the techniques for the measurement of skid resistance are further developed, skid resistance is likely to play an increasingly important role in determining the serviceability of a bridge deck.

Spalling

Although the importance of cracking, scaling, and wear is not to be minimized, their significance palls in comparison with the cost of repairing damage from corrosion spalling, which is the major curse of contemporary concrete bridge decks.

The occurrence of both scaling and spalling is directly related to the increase in the use of deicing chemicals. However, scaling is the deterioration of the concrete. It is a surface phenomenon that can be prevented by the use of a low water-cement ratio, air-entrained concrete. Spalling results from corrosion of the reinforcing steel. Substantial thicknesses of concrete may be involved and, once initiated, it is difficult to halt the corrosion process and permanently repair the damage.

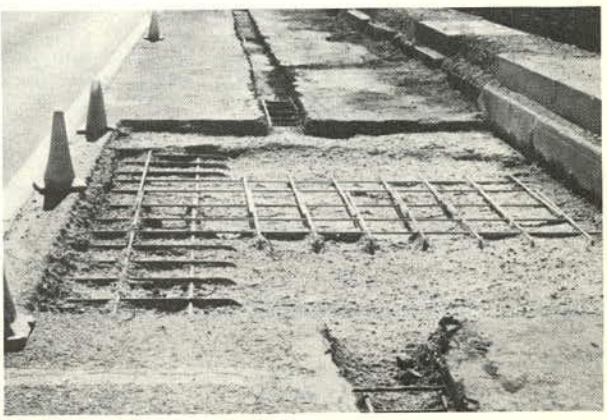


Figure 1. Severe scaling of concrete beneath an asphaltic concrete wearing course.

Corrosion of Steel in Concrete

Corrosion is an electrochemical process and, for an electrochemical cell to function, three basic elements are necessary (*17*): an anode, where corrosion takes place; a cathode, which does not corrode but maintains the ionic balance of the corrosion reactions; and an electrolyte, which is a solution capable of conducting electric current by ionic flow.

Iron, being relatively high in the electromotive-force series, has a substantial tendency to enter into solution, thereby liberating electrons at the anode (equation 1).

$$\text{anodic reaction: } Fe \rightarrow Fe^{++} + 2e^{-} \qquad (1)$$

In order to maintain equilibrium, electrons must be consumed at the cathode and, provided oxygen and moisture are present, hydroxyl ions are formed (equation 2).

$$\text{cathodic reaction: } O_2 + 2H_2O + 4e^{-} \rightarrow 4OH^{-} \qquad (2)$$

Ferrous hydroxide is deposited at the anodes (equation 3), and this is usually converted to ferric hydroxide to produce the familiar reddish-brown rust (equation 4).

$$2Fe + 2H_2O + O_2 \rightarrow 2Fe\,(OH)_2 \qquad (3)$$

$$4Fe\,(OH)_2 + 2H_2O + O_2 \rightarrow 4Fe\,(OH)_3 \qquad (4)$$

Corrosion is unlikely to occur in a medium of perfect uniformity. Reinforced concrete is not homogeneous, and electrical potential differences can occur at various places in the concrete because of differences in moisture content, oxygen concentration, cracking, and residual stresses in the steel. In such concrete, a corrosion cell is established along a reinforcing bar. The distance between the anodic and cathodic areas on the bar may range from less than 1 in. (25 mm) to more than 20 ft (6 m). The presence of anodes and cathodes on reinforcing steel that has been exposed prior to repairing a deck is shown in Figure 2.

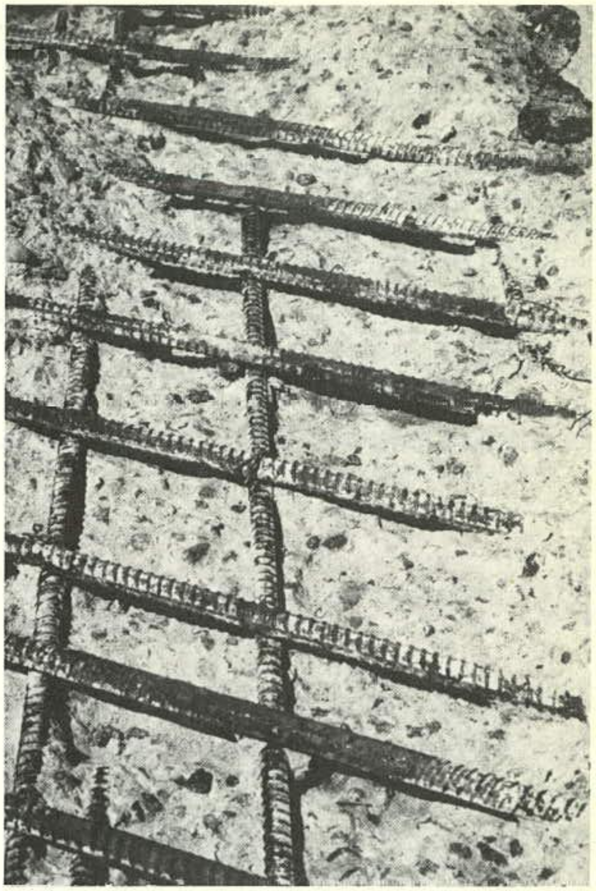


Figure 2. Location of anodes (dark or rusted areas) and cathodes on reinforcing steel exposed during repair operations.

Moisture is required not only to support the cathodic reaction but also to act as the electrolyte. Oxygen also is an essential factor in the corrosion process. Measurements have shown that sufficient oxygen will penetrate high quality wet concretes to support a considerable degree of steel corrosion (*18*). Neither concrete quality nor the thickness of cover was found to have a significant effect on the movement of dissolved oxygen through the concrete.

Uncracked, uncontaminated concrete normally has ample resistance to corrosive attack because of its high pH value, which results from the presence of calcium hydroxide and alkalis in the cement and which inhibits corrosion of the reinforcing steel. However, the passivity of the steel can be destroyed by soluble chlorides in the concrete. Once a certain concentration of chloride ion is exceeded, corrosion may begin if oxygen and moisture are present (*19, 20*).

Chloride in concrete may be in water soluble form or chemically combined with other ingredients (*21*). Soluble chlorides induce corrosion, whereas combined chloride is believed to have little effect. Research has shown that in concrete the threshold value for soluble chloride necessary for corrosion of embedded reinforcing steel can be as low as 0.15 percent by weight of cement (*22, 23*). The methods of measuring chloride content and the relationships between the soluble and total chloride content of hardened concrete are discussed in Chapter Two.

The permeability of concrete is a major factor affecting the corrosion of reinforcing steel (*10, 24, 25*). Concrete of low permeability also has a low porosity so that less water can enter and remain; hence the concrete is likely to have a low electrical conductivity. It also resists the penetration of salts to embedded steel.

Concrete permeability depends upon numerous factors including water-cement ratio, cement-aggregate ratio, aggregate grading, air-entrainment, consistency, degree of consolidation, and adequacy of curing. Although no conventional concrete is completely impermeable, concrete having a low permeability can be achieved by careful attention to good concrete technology, i.e., good quality materials, a minimum water-cement ratio consistent with placing conditions, good consolidation and finishing practices, and proper curing. The effects of water-cement ratio and degree of consolidation on the rate of ingress of chloride ions are shown in Figures 3 and 4 respectively (*26*). Concrete with a water-cement ratio of 0.40 was found to resist penetration by deicing salts significantly better than concretes with water-cement ratios of 0.50 and 0.60. A low water-cement ratio will not, however,

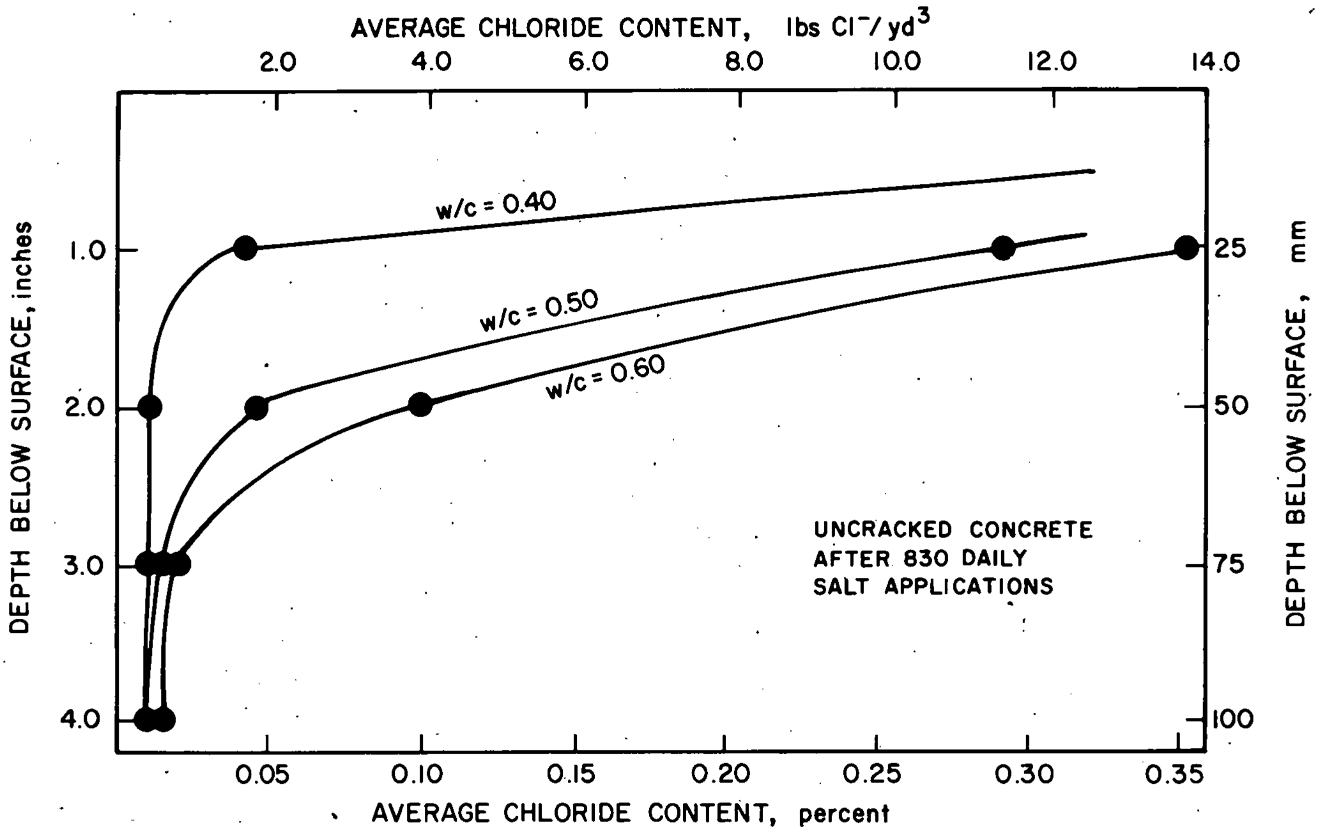


Figure 3. Effect of water-cement ratio on salt penetration (26).

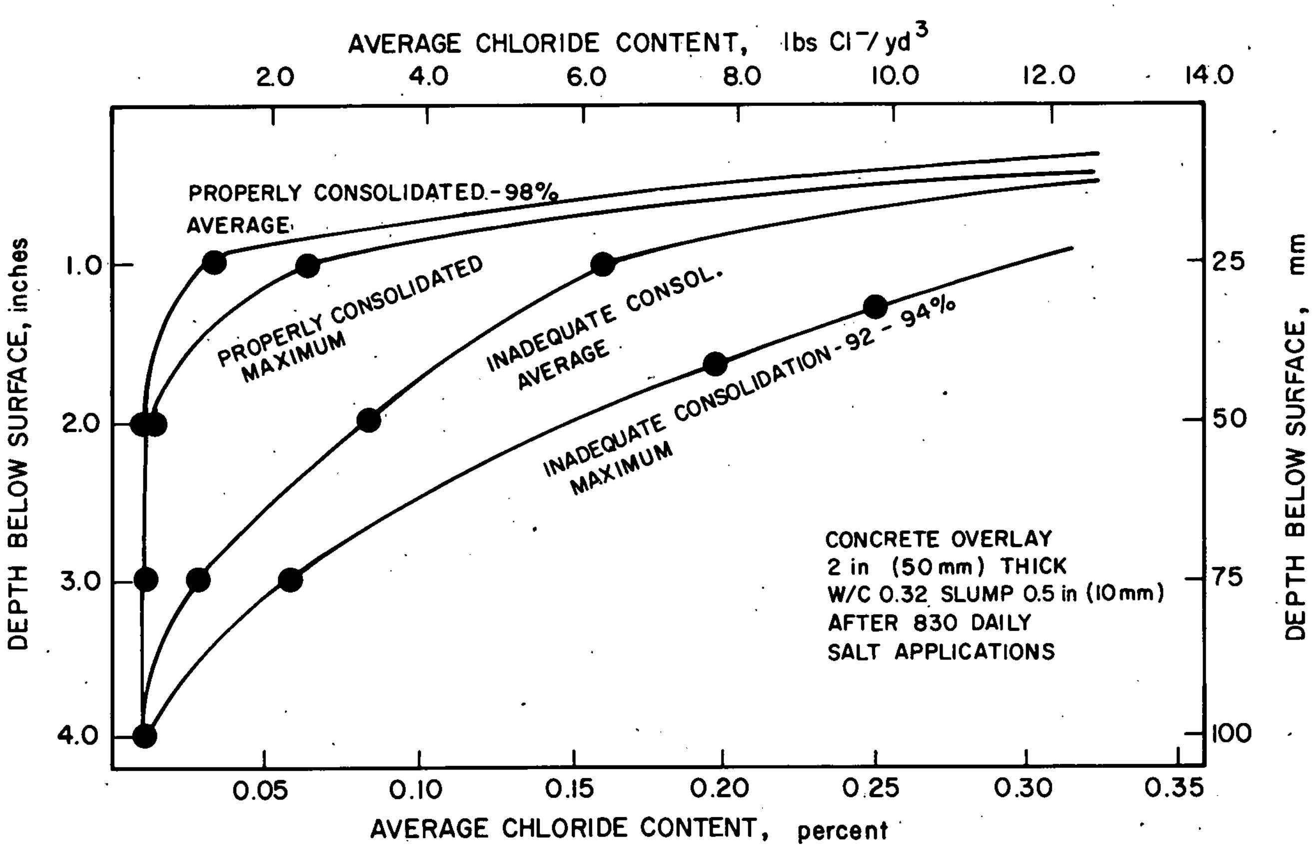


Figure 4. Effect of inadequate consolidation on salt penetration (26).

ensure low permeability, because, as shown in Figure 4, without proper consolidation, concrete may be readily penetrated by chloride ions.

The increase in corrosion protection achieved with an increase in cover is more than a simple arthimetic relationship. The diffusion of chloride ions through the cement paste results in the formation of calcium chloroaluminate, which reduces the chloride ion concentration and hence increases the tendency for further inward diffusion (*27*). Houston et al. (*12*) found that the onset of corrosion varies with the square of the clear concrete cover. Corrosion protection was found to increase with the ratio of clear cover to bar diameter (C/D), with good protection against corrosion-induced cracking and spalling being afforded by a C/D of 3.0 or greater. The effect of depth of cover and water-cement ratio of the concrete, as determined in FHWA time-to-corrosion studies, is shown in Figure 5. In the FHWA tests, the time to corrosion was determined, not by the observation of rust on the steel as was done by Houston et al., but by recording the number of daily applications of salt before active corrosion potentials were measured by the half-cell method. The bars in the FHWA study were 0.5 in. (13 mm) diameter. Although there is no direct relationship between the number of salt applications in these studies and the number of applications under field conditions, the results indicate that a C/D of 3.0 is not sufficient to prevent long-term corrosion of embedded reinforcing steel, even with a water-cement ratio of 0.40, though it is not clear whether such corrosion would progress to the point of spalling the concrete. The American Concrete Institute (ACI) committee report, "Guide to Durable Concrete" (*10*), recommends a minimum of 2-in. (50-mm) cover for bridge decks if the water-cement ratio of the concrete is 0.40 and 2½ in. (65 mm) for a water-cement ratio of 0.45.

Mechanics of Spalling

The mechanics of spalling begin when chloride ions penetrate the concrete to the level of the reinforcing steel, either slowly permeating through the concrete or penetrating more quickly by means of cracks. In poor quality concrete, channels develop from bleed water and speed the ingress of chloride ions. Cracks often form over the topmost reinforcing bars, usually transverse to the road alignment. These cracks may result from dead-load deflection of the deck while the concrete is still in its plastic stage, plastic shrinkage, drying shrinkage, or live load or thermal stresses, especially if the concrete has a high water content. Once the chloride ion concentration exceeds the threshold value, corrosion of the reinforcement begins in the presence of oxygen and moisture. The onset and extent of corrosion-induced damage is determined by the quality of the concrete, the thickness of the cover, and the size and spacing of the reinforcing bars.

The first visible sign that corrosion is taking place is usually the appearance of rust stains on the surface of the concrete. The rust stains become darker, increase in area, and eventually the concrete cracks. In bridge decks, it is common to experience horizontal cracks above corroding bars, thus producing a delaminated area. In lower quality concrete, fracture planes may occur at the level of the steel because a weak plane develops as a result of the bars restraining the sedimentation of high-water-content concretes after placing and finishing (*28*). The repeated action of vehicular loading and the formation of ice in the delaminated area then result in the familiar spall or pothole unless remedial action is taken. In areas where there is little concrete cover, vertical cracks directly over the reinforcing bars often occur.

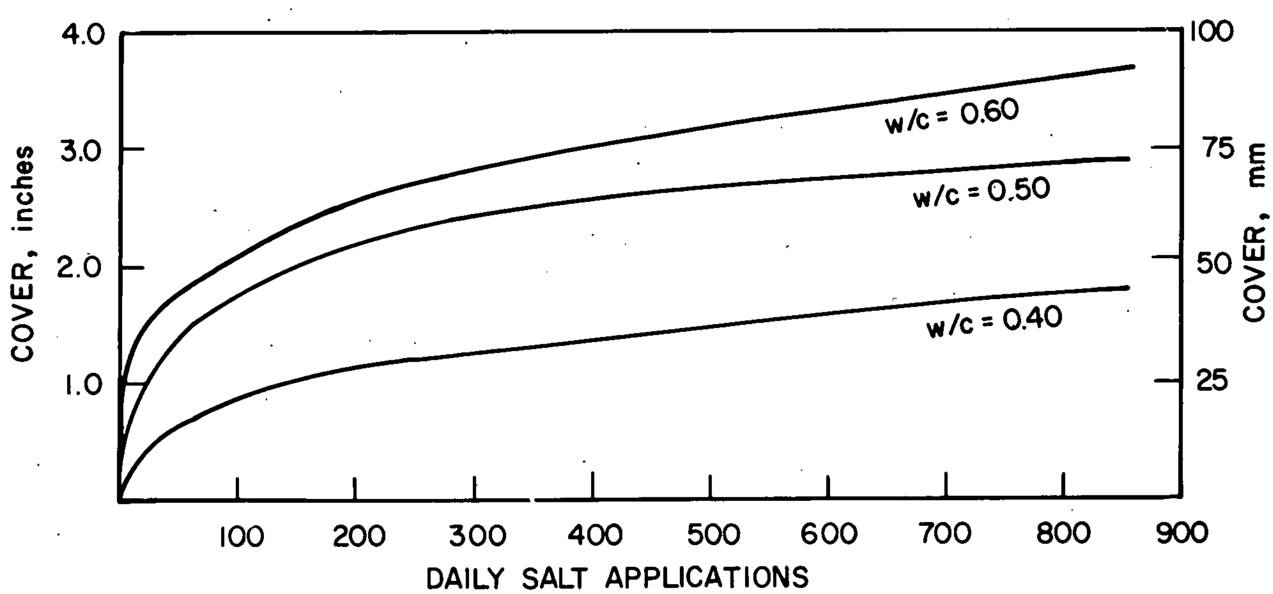


Figure 5. Effect of water-cement ratio and depth of cover on relative time to corrosion (26).

Cracks are formed because the iron oxides occupy a significantly larger volume, variously reported at 2.2 (*29*) to approximately 13 times (*30*) the volume of the original metal. Considerable pressure is exerted by the corrosion process when this expansion is prevented, and pressures as high as 4,700 lb/in.^{2} (32 MPa) have been reported (*2*). The amount of corrosion required to cause a crack is very small. A corrosion pit depth of 0.001 in. (0.03 mm) is sufficient to crack a 0.875-in. (22-mm) thick concrete cover (*31*). If the corroding area of the bar is sufficiently large, a trough or conical spall occurs. At this stage the loss of metal from the reinforcing steel is structurally insignificant, but the riding quality of the bridge is seriously impaired. At much more advanced stages of corrosion, severe pitting of the steel occurs with significant loss of cross-section. Large reductions in bar diameter usually occur when atmospheric corrosion of the steel is possible, such as in an open spall, and are the result of a relatively small anode protecting a much larger cathodic area of the adjacent steel.

CHAPTER TWO

EVALUATING EXISTING BRIDGE DECKS

PLANNING A CONDITION SURVEY

Bridge deck condition surveys range from quick visual examinations to detailed surveys requiring many man-hours of inspection, testing, and analysis. The actual expenditure on any particular survey is determined by the purpose of the investigation. Condition surveys fall into two categories: routine condition appraisals and preconstruction investigations. Routine condition surveys are normally performed on a regular basis, often at two-year intervals, for the purposes of data acquisition and record keeping. The information may be used to determine structural adequacy and to establish repair priorities or the need for a more detailed condition survey.

Detailed surveys are expensive and are normally undertaken only when work on a bridge deck is programmed and the most appropriate treatment is to be selected and contract documents prepared. The scope of the survey depends upon the alternative courses of action. If the deck must obviously be replaced, testing may not be required. If the deck is to be repaired, the detailed survey must yield sufficient information so that all necessary work can be included in the repair contract. If this is not done and the contractor is required to undertake extra work, delays often result, contract administration is difficult, and excessive costs may be incurred.

The precise nature of the work to be included in a detailed condition survey must be determined by each agency on the basis of its policies and procedures. Factors affecting this decision are:

- The type of structure.
- Its location and the traffic density.
- The nature and degree of deterioration.
- The priority and schedule for repairs.
- The policy for timing and type of repairs.
- The available human and financial resources.

Some states have prepared inspection manuals (*32, 33*) that describe in detail how bridge-deck evaluations are to be conducted and reported.

Because of the inherent variability in the condition of bridge decks, expertise and experience are required of at least one member of the inspection party. Whenever possible, the structural drawings should be obtained, and studied prior to visiting the site. If nondestructive testing is envisaged, it is good practice to lay out a grid on a site plan. This is especially important where the geometry of the structure is complex (for example, a large skew angle, a small radius of horizontal curvature, or variable width) to avoid mistakes in the field. Standard forms for recording pertinent information about the structure and the results of the condition survey normally will be developed.

VISUAL INSPECTIONS

A visual inspection is the first stage of any condition survey, and the effectiveness is determined by the skill of the inspector. The visual inspection will reveal defects in the structure, which should be noted on a site plan. Photographs are often used to supplement the field notes.

Elements that may be included in a deck-repair contract are normally included in the deck condition survey. Such elements may include the drainage system, expansion devices, approach slabs, curbs, parapet or barrier walls, and handrails. However, it is beyond the scope of this report to discuss the procedures for evaluating the condition of elements other than the deck. For complete structural evaluations (for example, to determine structural adequacy), many states have developed inspection manuals (*34*).

The visual inspection of an exposed concrete deck surface is relatively straightforward. All defects on both the top and bottom of the deck surface need to be recorded. The size, location and depth of spalls and scaling are normally reported. Scaling is sometimes described qualitatively in terms of its depth as follows (*8*):

light scaling	0 to ¼ in. (0 to 5 mm)
medium scaling	¼ to ½ in. (5 to 10 mm)
heavy scaling	½ to 1 in. (10 to 25 mm)
severe scaling	over 1 in. (over 25 mm)

Cracks are classified with respect to width, orientation, and, where possible, cause. Precise measurement of crack widths is neither feasible nor desirable, though a description in the following terms is useful (*13*):

hairline (H)	less than 0.004 in. (less than 0.1 mm)
narrow (N)	0.004 to 0.01 in. (0.1 to 0.3 mm)
medium (M)	0.01 to 0.03 in. (0.3 to 0.7 mm)
wide (W)	greater than 0.03 in. (greater than 0.7 mm)

Cracks are usually measured from the grid lines and sketched on a site plan. The width may be indicated by the abbreviation H, N, M, or W beside each crack. If the cracks are described in report form, their orientation is usually classified according to one of the following terms: transverse, longitudinal, diagonal, radial, pattern, or random. Care is required in ascertaining crack widths; cracks usually look wider at the surface because of their broken edges. Consequently, the width ¼ in. (5 mm) below the deck surface is often reported. The amount of moisture on the deck has a dramatic effect on the degree of cracking that is visible. Fine cracks are difficult to identify on wet or dry decks. Conversely, if a deck is examined under drying conditions, when moisture is associated with each crack and all the cracks are visible and appear to be wider than they are, the inexperienced observer may produce an exaggerated report.

Moving cracks of any width are much more troublesome than nonmoving cracks, because they tend to enlarge and also limit the options when selecting the method of repair. It is often difficult to determine whether or not a crack is active, though a crack that is visible on both the top and bottom surfaces of the deck will almost certainly be active.

The visual inspection of asphalt covered decks is much more difficult, and more experience is needed to search out the clues that denote the condition of the deck slab. Key pointers to defective concrete beneath the asphalt are cracking, especially radial cracks, and wet spots in the asphalt. Careful examination of the underside of the deck gives a good indication of the general condition of the deck. Common deficiencies are leakage, wet spots, and cracks, as shown in Figure 6. Efflorescence on the underside of the deck is evidence of water seepage through cracks or joints. Efflorescence results from water dissolving minerals from the concrete, usually calcium hydroxide, as it seeps through the concrete. The water evaporates from the bottom surface, and the salts are deposited as a whitish precipitate. In cases where the underside of the deck is inaccessible or covered by permanent steel forms, greater emphasis must be placed upon the other evaluation techniques described in this chapter.

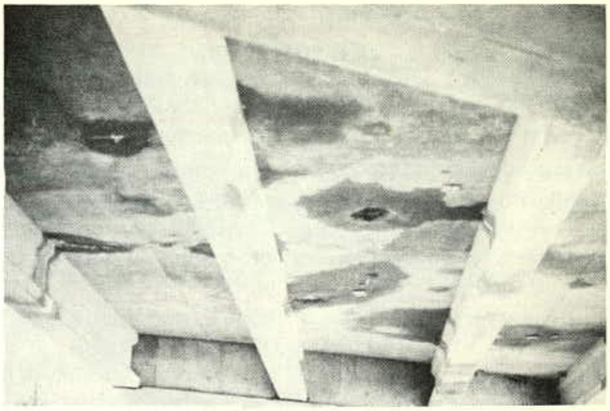


Figure 6. Underside deterioration of a deck slab showing wet spots, cracks, and efflorescence.

PACHOMETER SURVEYS

A pachometer is a device for measuring thickness. The name is derived from the Greek work pakhus, which means thick. In the context of bridge decks, a pachometer is used for measuring the clear concrete over embedded reinforcing steel.

Pachometer surveys may be included in an evaluation of bare concrete decks for one of the following reasons:

1. To determine if observed deterioration is the result of insufficient cover to the reinforcing steel.
2. To locate and measure the depth of reinforcing bars prior to taking samples for chloride content determination.
3. To locate areas with minimal cover that would prevent the use of rotary scarifying equipment.

There are several hand-held pachometers on the market, and each has a battery, a probe, and a scale. The battery generates a magnetic field between the two pole faces of the probe. The intensity of the magnetic field is inversely proportional to the cube of the distance from the pole faces. When external magnetic material is present, for example, a reinforcing bar, the magnetic field is distorted. The magnitude of the distortion is proportional to the size of the bar and its distance from the probe. The distortion is recorded on the scale of the instrument, which is calibrated to record the distance between the probe and the bar directly. The scale on some instruments also includes a correction for the bar diameter.

Before beginning a pachometer survey, it is desirable to check the bridge design drawings to determine the orientation of the upper most layer of steel in the deck and the diameter of these bars. The number of readings to be taken on a deck and their location must be decided depending upon the intended use of the data and the staff available for the survey. Although the survey can be done by one person, two persons are more usual—one to operate the pachometer and the other to record the measurements. In a complete survey, it is important that the readings be taken on a grid or some other procedure be used, that will produce a random sample, such as increments along a diagonal of the deck. Variations in the depth of cover on bridge decks are frequently not random and are caused by construction procedures. Such would be the case if a flexible finishing machine were used and the machine deflected between the travel (screed) rails with the result that the cover was reduced in the area of the midpoint between the rails. Another common example is when heavier bars are used in the negative moment areas of continuous structures without corresponding changes in the bar support system. Such biases can be avoided only by taking sufficient measurements to produce a statistically significant sample.

The pachometer is operated in accordance with the manufacturer's recommended procedure. Where a grid is used, the normal practice is to place the probe at a point on the grid with the long axis of the probe oriented parallel to the uppermost reinforcing bar. The probe is moved at right angles until the meter pointer indicates a maximum deflection, at which time it is directly over a bar. The probe is then moved in the opposite direction from the grid point until a bar is located. Common practice is to record the average depth of cover of the two bars either side of the grid point. If the structural drawings are not available and the orientation of the top bars is not known, the probe is rotated at several locations until a sharply defined minimum reading (maximum deflection) is obtained. This indicates the probe is directly above a bar, and the orientation of the bar coincides with the longitudinal axis of the probe.

A rolling pachometer has been developed that is capable of gathering data at a rate about twenty times that of conventional hand-held methods (*35*). The rolling equipment avoids the tedium of working on hands and knees with hand-held pachometers; instead it is rolled along the deck at prearranged grid lines. The equipment essentially consists of a hand-held pachometer with additional electronic components and a two-channel strip chart recorder, all mounted on a cart. The instrument is pushed along the deck at a constant speed of 1 mph (1.6 km/h). The instrument was evaluated under field conditions and found to be rugged and reliable.

An accuracy within the range ±⅛ in. (±3 mm) is generally obtained from most pachometers in the range 0 to 3 in. (0 to 75 mm). A bias effect, which causes measured values to be less than actual depths of cover, will result if the concrete contains magnetic materials. Some pozzolans, especially certain fly ashes, contain magnetic particles, and many concrete sands contain particles of magnetite. A correction factor can be established by either placing the probe on a sample of the concrete that does not contain any reinforcement, though this is rarely practical for other than new construction, or by coring and measuring the difference between recorded and actual values.

DELAMINATION DETECTION

In Chapter One it was stated that after the steel begins to corrode and before spalls are visible on the deck surface, horizontal cracks, or delaminations, occur at or above the level of the top reinforcing steel. Delaminations need to be detected in a condition survey because they indicate a high level of corrosion activity and represent areas of unsound concrete that must be repaired. It is not uncommon for more than one delamination to occur on different horizontal planes above the reinforcing steel. The separation between the upper layers of the deck concrete causes a dull sound to be heard when the deck surface is struck, thereby enabling delaminated areas to be identified.

Many tools have been devised for detecting delaminations, the first being hammers and iron rods, then chains, and more recently, acoustical methods. In extreme cases, delaminated areas have a slightly darker color than the surrounding areas of sound concrete and may be visible to the naked eye. Research is in progress to enable the detection of delaminated areas by remote sensing through the application of such techniques as thermography (*36, 37*), but such methods are not currently in routine use.

The use of the hammer is tedious and tiresome, because the operator works in a crouched position and covers only a small area of the deck at a time. The iron bar enables the operator to stand upright but is also very time consuming. The chain drag has been found to be accurate, efficient, simple and economical (*38*). One form of the apparatus is constructed from four or five segments of 1-in. (25-mm) chain about 18 in. (0.5 m) long attached to a 2-ft (0.6-m) piece of copper or aluminum tubing by means of a nonmetallic, flexible connection such as rope. A handle is attached to the midpoint of the tube to form a "T." The chain is dragged from side to side in a swinging motion allowing the chains to drag along the surface of the concrete and resulting in a ringing sound. The dull sound emitted when a delaminated area is encountered is easily identified.

Some authorities have found that heavier chains, such as logging chains with a 2-in. (50-mm) link made from ⅜-in. (10-mm) diameter steel, as illustrated in Figure 7, produce more accurate results, especially when the survey must be carried out with interference from traffic noise. A chain approximately 5 or 6 ft (1.5 or 1.8 m) long is swung from side to side along the deck surface, enabling a large area of deck to be covered rapidly. When a delaminated area is located, the length of chain in contact with the deck is shortened and the limits of the delamination can be identified. It has been reported (*39*) that the chain drag will indicate the existence of delaminations in locations where the hammer method will not.

Although the chain drag is a convenient method of determining the location of delaminations, the recording of the observations is tedious. The normal procedure is to mark a grid on the deck and measure the area and the location of the delaminations with respect to the grid lines. This procedure is especially time consuming on a deck that has a large number of delaminations. Consequently, chains are suitable for determining the presence of delaminations during concrete removal operations but less satisfactory when used in a deck condition survey.

The Texas Highway Department, in cooperation with the Texas Transportation Institute, developed a portable electronic instrument for the detection and recording of delaminations on a bridge deck (*40, 41*). The device, known as a Delamtect, is commercially available. The equipment consists of a tapping device, a sonic receiver, and a two-channel pen recorder mounted on a small cart. The tapping device is an oscillating solenoid mounted on a pair of steel-rimmed wheels in contact with the deck sur-

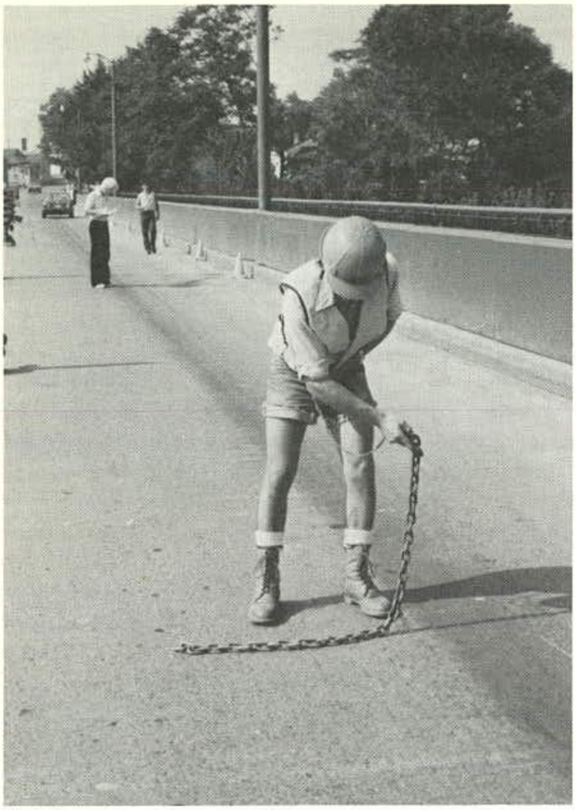


Figure 7. Use of heavy chain for detection of delaminations.

face. The instrument is wheeled across the deck, and the instrument picks up (by means of microphones) and electronically interprets the acoustical signals generated by the instrument and reflected through the concrete. The signals are displayed as two independent traces on the pen recorder. The operator also may wear headphones that will indicate the presence of a delamination so the location may be identified, often by means of spray paint, directly on the deck surface. On each pass, the detector surveys about a 3-ft (1-m) wide path, though wider spacings are sometimes used. The equipment overcomes the workload of mapping out a full grid on the deck, because only the survey lines need be located. The workload is also independent of the number and extent of delaminations on the deck, and the instrument is not influenced by traffic noise. The delamination detector does not locate delaminated areas as accurately as is possible using the chain drag; therefore, it is most useful in conducting general condition surveys, when an overall indication of the deck condition is required, rather than for isolating specific areas requiring repair.

A thickness of asphalt greater than about 1 in. (25 mm) masks the sound of delaminations. Furthermore, even with the Delamtect, a positive response is a measure of a subsurface discontinuity and, without further examination, it is not known if the discontinuity is a delamination in the concrete deck or a lack of bond between the asphalt and concrete.

MEASUREMENT OF CHLORIDE CONTENTS

Chloride ion plays an important role in bridge deck deterioration, because its presence, at a concentration beyond the threshold value, is necessary at the reinforcing steel before corrosion can begin. Chlorides used as deicing agents go into solution and easily penetrate even good quality, uncracked concrete. Measurement of the chloride content of concrete indicates whether one of the conditions for the corrosion of the reinforcing steel is present and yields information necessary for selecting the most appropriate method of bridge deck repair.

A number of questions must be answered before obtaining samples of concrete for chloride analysis and determining the method of chemical analysis. How many specimens should be taken and from which locations? Should the specimen be taken as a core or a pulverized sample? Should the determination be made in situ or in the laboratory? Should the sample represent the whole concrete or just the mortar fraction? Should the total or the soluble chloride content be measured? Although definitive answers to these questions have not been formulated, the factors that have to be considered are discussed below.

Location and Number of Samples

The number of samples required for chloride analysis is determined by the variability of the chloride content within a given bridge deck. The number of samples actually taken is determined by the budget and purpose of the investigation. A survey prior to repair requires more detailed information than a routine deck condition survey. Fewer samples are required from a deck showing no signs of physical distress to confirm that chloride contents are below the corrosion threshold value. The degree of variability has been found to differ from deck to deck because of such factors as variations in concrete quality and the location of the deck drains, which may promote ponding of water and cause higher chloride concentrations in localized areas. As a general rule, it has been recommended (*42*) that at least six samples be taken from a deck to constitute a valid sample in a detailed survey.

Similarly, the location of the samples is a subjective decision. One approach is to take purely random samples; another is to select the locations to try and obtain the maximum variation of chlorides in the deck. For example, samples may be taken from delaminated areas and areas of sound concrete, from areas of good and poor drainage, or from areas of high and low potential measurements. Another approach is to take all the chloride samples from areas of half-cell potentials between $-0.20V$ and $-0.35V$ (*43*), because this is the range of uncertain corrosion activity. Determination of the chloride contents may indicate the possibility of future corrosion in the areas where samples were taken.

Type of Sample

The standard method of obtaining samples for chloride measurement has been to take cores using truck mounted, water-cooled core drills. The diamond core bits are expensive, and the procedure is time consuming. The precise location of the core should be determined through the use of a pachometer to avoid drilling through reinforcing bars. The minimum depth of the reinforcement within approximately a 4-ft (1.2-m) radius of the core should be noted so that the chloride content at the level of the steel can be determined.

The cores must be sectioned and pulverized to provide the powdered samples required for complete extraction of the chloride during the chemical analysis. To avoid contamination of one sample by another, the pulverizer and all tools must be carefully washed between samples with ethyl alcohol or distilled water and permitted to dry. If only the chloride content at the level of the top steel is required, only that section need be cut from the core. If a measurement of the variation of chloride with depth, or a chloride profile, is required, a standard method of sectioning is recommended to enable comparisons between data from different decks. In most decks, the chloride content diminishes rapidly as the distance from the surface increases, and consequently it is advisable to make most of the measurements near the deck surface. One method of sectioning (*44*) is to take a ¼-in. (6-mm) slice at the surface and double the thickness of each successive slice, i.e., chloride measurements are made on sections 0 to ¼ in., ¼ to ¾ in., ¾ to 1¾ in., 1¾ to 3¾ in. (0 to 6 mm, 6 to 19 mm, 19 to 44 mm, 44 to 95 mm), etc. A more convenient method of sectioning is to cut the core into slices ½ in. (13 mm) thick and determine the chloride content of alternate slices, making sure that the slice at the level of the reinforcing steel is included.

An alternative method is to obtain a pulverized sample directly from the bridge deck by means of a rotary hammer (*22*). The drill is fitted with a core bit and a carbide-tipped starter bit positioned inside the core bit. This combination increases the pulverizing action and retains the major portion of the pulverized material inside the core hole until the desired depth is reached. The powdered concrete is collected with a spoon or with a vacuum attached to the drill and placed in a sealed container. The pulverized sample is checked in the laboratory to determine if it will completely pass a No. 50 (300 μm) mesh screen. Occasionally a short period of additional pulverizing will be required.

The rotary-percussion drill fitted with a depth indicator may be used in combination with a vacuum cleaner to obtain samples from specific depths in the deck. The procedure is to locate the position and depth of the reinforcement as indicated for core samples. If the chloride content at the level of the reinforcement is required, a hole is drilled to the depth of the reinforcement minus ¼ in. (6 mm) and cleaned out with the vacuum cleaner. The hole is drilled for a further ½ in. (13 mm) and the sample collected for analysis. This procedure can be modified as necessary to obtain samples from any required depth. When pulverizing, care must be taken not to contaminate samples at the sampling depth by abrading concrete from the sides of the hole, especially near the surface where the chloride contents are highest. This problem can be overcome by reducing the core diameter as each successive sample is taken.

The use of the rotary hammer has the advantages of portability, light weight, speed, and economy. The use of core samples permits the preparation of samples under controlled laboratory conditions and is generally preferred when maximum accuracy is required.

In Situ Measurement of Chloride Content

The Kansas DOT has developed a method for the measurement of the chloride content of bridge decks in situ (*45*). The procedure is as follows. A ¾-in. (19-mm) diameter hole is drilled in the deck to a predetermined depth using a vacuum drill system (*46*). The hole is filled with a borate nitrate solution, and a chloride-ion-specific electrode is inserted. After 90 seconds, the potential across the electrode is measured and converted to chloride concentration using a calibration curve. The chief advantages of the method are that it is quick (approximately three minutes per determination) and the damage to the deck is minimal and easily repaired. The accuracy of the method is approximately ± 0.5 lb Cl^-/yd^3 (± 0.3 kg Cl^-/m^3).

Nature of the Sample

To prevent distortion of the chloride measurements by the presence of large amounts of aggregate in the sample, the mortar fraction of the concrete is sometimes analyzed. Some investigators have separated the coarse aggregate particles by hand prior to pulverizing, though this is not considered feasible on a production basis. An alternative is to drill into a core at the required section until aggregate is encountered. The procedure is repeated until a sufficient quantity of drill dust has been collected.

However, neither of the above methods is in common use and, if aggregate-induced distortions are suspected, a correction can be determined by measuring the weight loss of the sample between 221 and 932 F (105 and 500 C) (*22*). The aggregate experiences only a small weight loss in this range, whereas the cement paste has a significant weight loss. Similarly, a correction can also be made for the moisture content of the sample (*22*). These corrections are not, however, necessary for most bridge deck work, and errors can most easily be avoided by increasing the number of samples.

Measurement of Total and Soluble Chlorides

The method of extracting the chloride contained in a sample of concrete determines not only the quantity measured but also the significance of that value. In general, there are two types of chloride analyses: the measurement of total chloride and the measurement of soluble chlorides. When the significance of the role of the chloride ion in the corrosion of steel in concrete was appreciated, an accurate, reliable method for determining chloride ion concentration in hardened concrete was urgently needed. Such a method was developed by Berman

(*21*) and presented in a procedural form by Clear (*22*). The method employs a wet chemical analysis to determine the total chloride content of a concrete sample. It is then necessary to compensate for the fact that all the chloride is not readily available to the corrosion process. The test method involves dissolution of a powdered sample of the concrete in dilute nitric acid and subsequent potentiometric titration of the chloride ion with silver nitrate solution. The procedure is used by many state highway organizations and has been found reproducible by different operators and laboratories. The accuracy is within 0.5 percent of the chloride present.

More recently, a simplified procedure has been developed which significantly reduces the analysis time per sample (*47*). The major change in the method is the use of the Gran endpoint determination procedure. There is no significant change in accuracy and precision. The method of sampling and testing for total chloride content is now prescribed by AASHTO (AASHTO T260).

The measured quantity of soluble chloride content in concrete is determined by the age of the specimen and the duration and medium of extraction. Ideally, a procedure for the measurement of soluble chlorides would determine the quantity of chloride ions available to the corrosion process. However, the division between free and combined chloride is not well defined, and even water will eventually dissolve all the chloride present in cement paste (*48*). It is therefore necessary to adopt an arbitrary standard procedure for extracting the soluble chlorides and to relate the result to the threshold value that has been determined for the corrosion of embedded steel. A test procedure has been recommended (*49*) that involves the extraction of the chlorides in distilled water by boiling for five minutes and then letting the sample stand for a further 24 hours.

For a detailed description of the method of sampling, the determination of water-soluble chlorides, and the measurement of total chloride content by potentiometric titration or the Gran endpoint determination procedure, the reader is referred to the report by Clear and Harrigan (*49*). The interpretation of the test results is further discussed later in this chapter.

MEASUREMENT OF CORROSION POTENTIALS

The method of measuring half-cell potentials on concrete bridge decks was developed in California (*50*). It has been further investigated and promoted by the FHWA (*51*).

In Chapter One it was shown that corrosion is an electrochemical process and that when corrosion begins anodic and cathodic areas are developed on the reinforcing bars. Corrosion currents flow through the electrolyte from the anodic areas to the cathodic areas. A potential difference, or voltage, exists between the anodic (half cell) and the cathodic (half cell) areas, which may be measured by a voltmeter (*38*). The electrical activity of the half cells changes seasonally and with changes in the electrolyte such that the difference in measured voltage between two unstable half cells is not a good measure of corrosion activity in the deck. For this reason, the potential of the corrosion half cells in the deck is compared with a standard reference half cell, which has a known electrical potential.

Earlier studies in which the procedure was developed used a calomel cell as the reference cell, but the copper/copper-sulfate cell is now preferred. The latter cell is sturdier and easier to use. A copper rod immersed in saturated copper sulfate solution represents a half cell of constant electrical potential. To compare the electrical potential of the standard cell to that of the steel embedded in the concrete, the two must be connected through a high-impedance voltmeter. This is done by making a positive connection to the top mat of reinforcing steel and by providing a moisture path through the concrete between the standard cell and the point at which the potential is being measured. Convention dictates that the potentials be reported as negative values even though the arrangement of the voltmeter and half cell is such that the value of the corrosion potential is read on the voltmeter as a positive number. The reason for this apparent anomaly is that the convention used by the National Association of Corrosion Engineers assigns positive values to the more noble metals, such as gold and copper, and negative values to more active metals, such as iron and zinc, relative to the standard hydrogen half cell, which has a potential of zero. Following this convention, the corrosion potential of steel is more negative than the reference copper/copper-sulfate half cell potential.

A full description of the equipment and a standard test procedure have been published by ASTM (ASTM C876). The procedure requires that where attachment is not made directly to the reinforcing steel, it must be demonstrated that the component to which the lead is attached is directly attached to the reinforcing steel. This is usually done by measuring the resistance between the connection and other metal fixtures on the deck, for example, between expansion joints or deck drains as widely separated as possible. On most bridge decks, the resistance will be less than 10 ohms if good connections have been made.

On bridge decks that have received a seal coat or a membrane, the seal or membrane must be punctured at the point of measurement. This is commonly done by the use of a hand drill fitted with a ½-in. (13-mm) diameter masonry bit. Readings can sometimes be taken through an asphalt wearing course, but unless it can be shown that electrical contact is made through the asphalt, it is good practice to drill holes in the same manner as when a deck seal is used. If the asphalt contains sufficient moisture that there is electrical continuity over wide areas, the applicability of the half-cell method is the same as when the test is used under water, i.e., corrosion activity will be detected but not necessarily its location.

A temperature correction must be applied to the standard cell potential and, under working temperatures of less than 50 F (10 C), 15 percent by volume of alcohol should be added to the contact solution. The test can not be performed when the deck is frozen, because the high electrical resistance of ice prevents completion of the circuit. Although the test procedure can be used at temperatures between freezing and 50 F (10 C), it is recommended that both the deck and the ambient temperature be above 50 F (10 C).

ELECTRICAL RESISTANCE TESTING

A nondestructive electrical method for evaluating the permeability of bridge deck seal coats was developed by Spellman and Stratfull (*52*) and subsequently has been applied to membranes.

The method assumes that where a dielectric material is used as a deck seal, its electrical resistance is a measure of its waterproofing ability. Thus, if the sealant or membrane is porous and water can pass through the pores, the electrical resistance will be low because of the multiple paths available for the flow of current. Conversely, if the deck seal is impermeable to water, the electrical resistance should be infinite. The method is applicable to most sealants and membranes, because few of the commercial products are electrically conductive.

The basic concept of the experimental procedure is to connect one lead of an ohmmeter to the surface of the membrane and the other lead to the deck reinforcing steel. A full description of the equipment and a standard test procedure have been published by ASTM (ASTM D3633). Neither half-cell potentials nor electrical resistance can be measured if the reinforcing steel is coated with epoxy, because the latter is a dielectric material. Most membranes are installed with an asphalt wearing course, and many are also protected by an intermediate layer of protection board or roofing felt. Electrical contact to the membrane is made by placing a moist sponge on the surface of the asphalt. As shown in Figure 8, the electrical circuit is completed by attaching a copper plate to the sponge, with provision for connecting one lead from the ohmmeter.

The location of the test sites has to be chosen carefully. If a random sample of resistance readings is desired, this is usually achieved by means of a grid. However, the test is most commonly used to identify deficiencies in membranes, and experience has shown that the locations most susceptible to leakage are near the curbs and in the wheel paths. Consequently, many authorities take measurements only in the areas where leakage is most probable.

The permeability of asphalt wearing courses varies considerably, and it may take a few hours to wet the asphalt and complete the electrical circuit. The first resistance readings at the test location selected as the checkpoint are taken approximately 30 minutes after wetting the test locations. The operation is repeated at regular intervals until the resistance remains essentially unchanged. At this time, the wetting solution is assumed to have contacted the surface of the membrane. Additional wetting solution is applied to the deck surface between readings as required. Where the readings remain essentially infinite, measurements should be continued for at least four hours.

Some problems have been experienced in the use of the test, mainly because of variations in pavement porosity and moisture conditions (*53, 54*). If the wearing course is dry and incomplete wetting occurs, the apparent resistivity of the membrane will be increased. As initially conceived, the result of the test was a measure of the resistivity of a deck sealant. However, where there is a bituminous concrete overlay, it is widely recognized that the contact area of the wetting solution with the membrane is indeterminate and is a function of time of soaking, moisture content, and permeability of the asphalt and of the

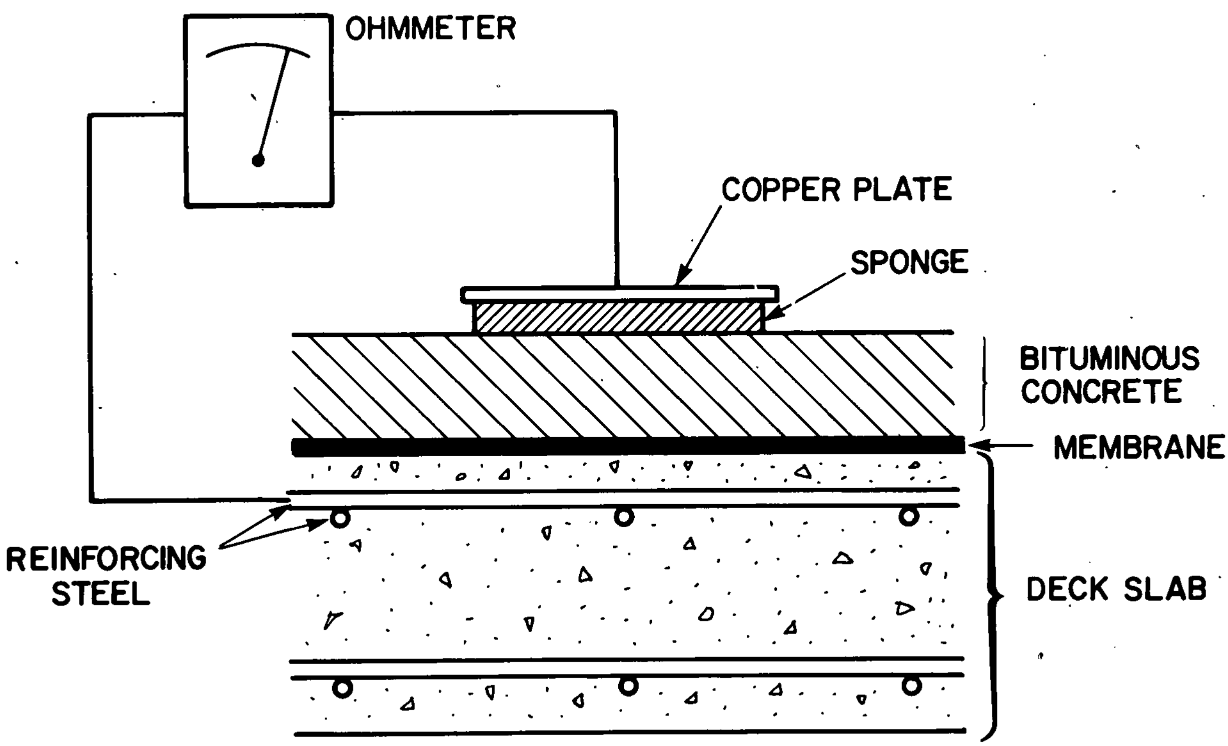


Figure 8. Electrical circuit for the measurement of the resistance of deck sealants.

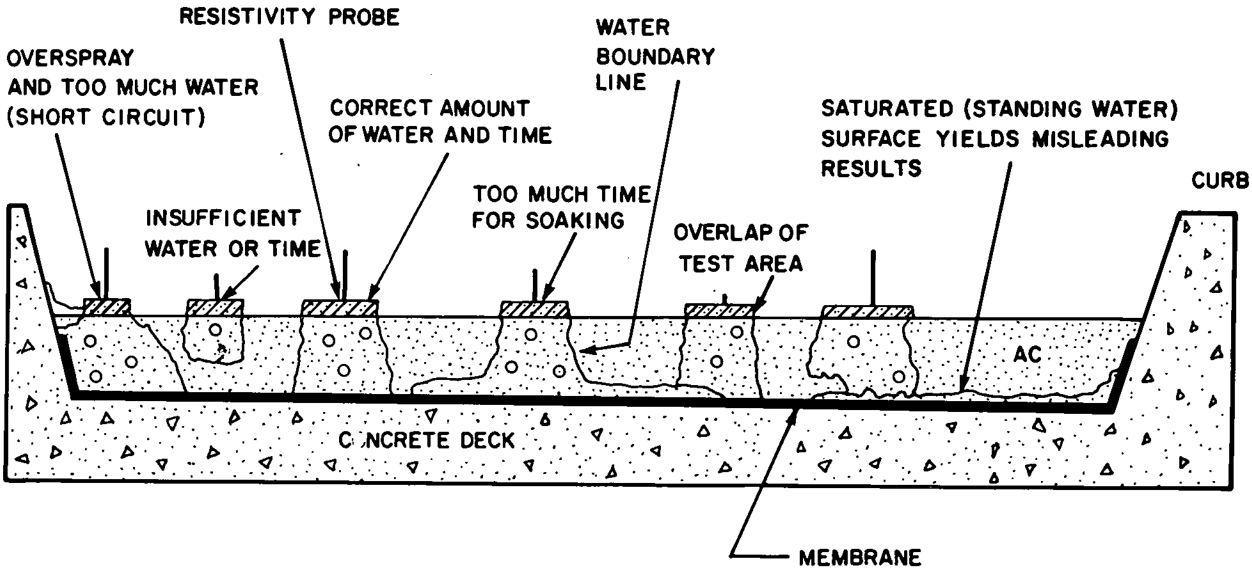


Figure 9. Possible problems in resistivity measurements (43).

longitudinal grade and crossfall of the deck. Some of the problems that can occur in measuring resistivity are illustrated in Figure 9. Because of the difficulty of establishing the area of contact of the solution, some authorities report the values as resistance measurements rather than resistivity. If the asphalt wearing course is moist and water is ponded on the upper surface of the membrane, a low resistance reading may occur because of a short circuit to the reinforcing steel through a deck drain or a steel expansion device. A short circuit may also occur around the edge of the membrane adjacent to the curb.

The Arizona Department of Transportation has developed a procedure that includes the placing of two 1-ft (0.3-m) square aluminum foil sheets on the membrane prior to asphalting (*43*). The sheets are electrically connected to the reinforcing steel, usually at a deck drain. By measuring the resistivity between the deck surface and the aluminum sheets, the time for wetting of the asphalt can be established and the problem of substantially increasing the wetted area can be avoided. Measurements of the resistance between the two sheets on a deck can also be used to determine if low resistivity readings are the result of water on the surface of the membrane. If the resistance between the sheets is low, it is likely that water is present on the membrane, whereas if the resistance is high, the low resistivity readings indicate a permeable membrane. In cases where aluminum sheets are not placed beneath the asphalt, the presence of moisture in the asphalt can be detected by measuring the resistance between probes placed on the surface of the asphalt at widely separated locations on the deck.

Uniformly high or low resistance readings should be viewed with suspicion and thoroughly investigated. Similarly, it is good practice to plot equal resistance contours where sufficient readings are taken and to investigate anomalous areas more closely.

CORE DRILLING AND TESTING

The drilling and subsequent examination of cores is a measure of the quality of concrete. It is a useful supplement to the nondestructive testing of an exposed concrete bridge deck and is vital to ascertaining the condition of an asphalt-covered deck. In the case of asphalt-covered decks, it is often useful to supplement coring by dry sawing to remove sections of asphalt and examine the concrete surface. If dry, as opposed to wet sawing, is employed, information can be gained about the presence of water on the deck surface; this is especially useful in the case of decks that have been waterproofed.

Coring is an expensive procedure compared with other test methods, and the number of cores should therefore be kept to a minimum. On an exposed concrete surface, a general rule is one core for each 2,000 ft^2 (185 m^2) of deck area, with a minimum of three cores. On asphalt-covered decks, three or four times as many cores may be needed because of the lack of corroborating data from other test methods. A thin-walled diamond bit should be used, and the minimum core diameter should be 4 in. (100 mm) to avoid inducing fractures in the concrete during the coring operations. Smaller diameter cores are also used but are less reliable when used to measure the compressive strength of the concrete.

The location of all cores should be noted on the site plan. The majority of the cores should be taken in areas of deterioration or, in the case of asphalt-covered decks, suspected deterioration. Suspect areas are likely to be near the curbs, at cracks in the asphalt, or in areas of poor drainage. Where possible, it is advisable to retrieve the asphalt prior to drilling the concrete deck.

The thickness, condition, and type of membrane present should be noted in the core log, together with an assessment of the bond between the asphalt, concrete, and the membrane. The presence of protection board or roofing felt should also be noted.

On exposed concrete decks, cores should be taken after completion of the nondestructive testing, because they can be especially useful if taken in areas of contradictory data. An example would be in an area of high potential and high cover when the core should reveal if the disparate findings could be attributed to poor quality concrete. This sequence of operations also prevents water from the coring operation interfering with the half-cell potential measurements.

In general, cores should not be taken from the wheelpath areas unless there is sufficient time for the backfill material to cure before the lane is reopened to traffic. Cores must not be taken through prestressing strands. Reinforcing steel should be located by means of a pachometer and avoided by the core drill. Where reinforcing steel is intercepted and the core terminated at the level of the steel, the depth of cover and any corrosion on the steel should be noted. Rarely is it necessary to drill completely through the deck and, if partial depth cores are taken, the task of backfilling is simplified. Where a core is broken into several fragments, the orientation and juxtaposition of the pieces should be indicated, either by a sketch or identification of the individual pieces, except where the deterioration is so extensive that the core has been converted to rubble.

Examination of cores is normally done in the laboratory. The nature and extent of the testing is determined by the purpose of the investigation, the type of deterioration encountered, and the alternative methods of repair. The laboratory evaluation may include one or more of the following: a visual appraisal of the type and degree of deterioration, a petrographic examination to determine the condition of the aggregates and the paste, a measurement of the air-void system, a density and strength test, or a measure of the chloride ion content. Because the testing on each core will vary both qualitatively and quantitatively, it is essential that the laboratory evaluations be performed by qualified personnel.

INTERPRETATION AND SIGNIFICANCE OF RESULTS

Direct Physical Measurements

The results of a visual inspection and of pachometer and delamination surveys are direct physical measurements, and no interpretation of the findings is required.

The presence of delaminated areas indicates that corrosion of the steel has progressed to the point where distress has occurred in the concrete. If remedial action is not taken, delaminated areas may progress to open spalls. Prior to any deck repair, the delaminations must either be removed and the concrete replaced or the hollow plane areas injected with a suitable adhesive to restore the riding quality and structural integrity of the deck.

The significance of the pachometer survey is that there is a well-defined relationship between inadequate cover [less than 2 in. (50 mm) of good quality concrete] and the occurrence of delaminations and active corrosion. Inadequate cover is the single most common cause of spalling. Pachometer readings of less than 2 in. (50 mm) indicate that spalling may occur at some future time if corrosion of the reinforcing steel takes place. Following an extensive survey of bridges in the mid-1960s, Stark (*55*) concluded that all the spalls were associated with steel having less than 2 in. (50 mm) of cover, and usually the cover was less than 1½ in. (38 mm). The relationship between the onset of spalling, concrete cover, and quality of concrete has also been demonstrated in the FHWA time-to-corrosion study in which specimens were stored in an outdoor exposure plot. As illustrated in Figure 5, after 830 daily applications of salt, the clear cover of properly consolidated concrete to protect 95 percent of the reinforcing steel from corrosion was found to be 1.7 in. (43 mm) for a water-cement ratio of 0.40 and 2.8 in. (70 mm) for a water-cement ratio of 0.50 (*26*). It should be noted that the thicknesses of cover reported are minimum values, which are not the same as design or specified cover because there is no allowance for construction tolerances. The effect of construction practices on the specified cover is discussed in Chapter Three.

Chloride Content Corrosion Threshold

The important question to be answered with respect to chloride in concrete is: Is there sufficient chloride available in the concrete to destroy the passivity of the reinforcing steel? The answer to this question gives rise to the concept of a chloride content corrosion threshold, which is defined as the minimum quantity of chloride required to initiate the corrosion of steel embedded in concrete provided that other necessary conditions, chiefly the presence of oxygen and moisture, exist. Establishing a universally applicable corrosion threshold is difficult. The chloride required to initiate corrosion appears to be dependent upon the initial pH of the concrete, the proportion of soluble chlorides present, the quantity of cement, the moisture content, and other factors (*56*).

Lewis (*23*) reported the soluble chloride corrosion threshold to be 0.15 percent Cl^- by weight of cement. Work in the FHWA laboratories showed an average chloride solubility of 75 to 80 percent for hardened concrete subject to deicing salts, and the corrosion threshold was therefore established at 0.20 percent of total chlorides by weight of cement. For a typical bridge deck concrete having a cement factor of 7 sacks per cu yd (658 lb/yd^3 or 390 kg/m^3), the total chloride content threshold is approximately 0.033 percent Cl^- by weight of concrete. This value has been confirmed by field studies including those in California (*42*) and New York (*57*). Although the result of the test method is a chloride content in percent by weight of concrete, chloride values are sometimes expressed in terms of weight of chloride ion, sodium chloride or calcium chloride, per cubic yard (m^3) of concrete. The conversion factors between these various units are given in Table 1.

Unfortunately the soluble chloride content of concrete is not a sensibly constant proportion of the total chloride content. It can vary considerably because of the chloride contents of the ingredients of the concrete mixture. All the materials used in concrete contain some chlorides and,

TABLE 1

CONVERSION FACTORS FOR EXPRESSING CHLORIDE CONTENT OF CONCRETE

Required Units	Method of Conversion	Example(1)
1. percent Cl^- by weight of concrete	Result of test method	0.025
2. ppm Cl^- by weight of concrete	Multiply #1 by 10 000	250
3. percent Cl^- by weight of cement	Multiply #1 by weight of concrete in lb/yd^3 (kg/m^3) and divide by cement content in lb/yd^3 (kg/m^3)	0.15
4. lb Cl^- per cu yd	Multiply #1 by weight of concrete in lb/yd^3 (kg/m^3) and divide by 100	1.0 lb/yd^2 or 0.59 kg/m^3
5. lb sodium chloride per cu yd of concrete (kg/m^3)	Multiply #4 by 1.65	1.65 lb/yd^3 or 0.97 kg/m^3
6. lb calcium chloride (2) per cu yd of concrete (kg/m^3)	Multiply #4 by 2.07	2.07 lb/yd^3 or 1.23 kg/m^3

(1) Assuming a cement content of 658 lb/yd^3 (390 kg/m^3) and concrete unit weight 145 lb/ft^3 (2320 kg/m^3).

(2) Calcium chloride dihydrate (flake calcium chloride), $CaCl_2 \cdot 2H_2O$

in the case of cement, the chloride content varies with the cement composition. Although aggregates do not usually contain significant amounts of chloride (*58*), there are exceptions. There have been reports of natural aggregates with a total chloride content greater than 0.1 percent, of which less than one third is soluble but is not thought to be available to contribute to corrosion (*59*). On the other hand, some aggregates with high chloride content are known to have caused corrosion (*10*).

When considering the probability of corrosion, it is logical to measure only the soluble chloride content of the concrete, rather than the total chloride. Tests for soluble chloride, however, are time-consuming and difficult to control. Conversely, the test for total chloride does not have the above drawbacks. Consequently, the recommended procedure for the measurement of chloride content is to determine the total chloride content and compare the result with the corrosion threshold value of 0.15 percent by weight of cement. Only if the result exceeds this limit is the soluble chloride content of the concrete determined.

Threshold values for corrosion refer to the concrete surrounding the reinforcing steel. In bridge decks, higher concentrations of chlorides are found nearer the surface of the deck and may migrate towards the steel with time. Consequently, a conservative approach is normally taken when assessing the potential for corrosion in a deck slab, even when half-cell measurements indicate the steel is in a passive condition.

It is important to note that the corrosion threshold value is the amount of chloride sufficient to depassivate the steel. Whether corrosion actually begins depends upon the environmental conditions, principally the existence of water and oxygen. The occurrence and rate of corrosion are time-dependent and, once the threshold value is exceeded, do not depend upon a further increase in the chloride content. Consequently there is no relationship between the chloride content of the concrete and the rate or degree of corrosion except that all generally increase with time. Similarly, for chloride contents above the threshold value, there is no relationship between chloride content, the magnitude of half-cell potentials, and the existence of delaminations except for the indirect relationship that all are a function of time.

Half-Cell Potentials

The half-cell potential measures the corrosion activity in the bridge deck (*50, 60, 61*). Corrosion is an electrochemical reaction and, consequently, the rate of the reaction decreases with decreasing temperature. As a result, measured potentials will be lower in cooler temperatures. It is important to recognize that the test measures the corrosion activity at the time of test and that corrosion may be more active at other times of year. For this reason, deck and ambient temperatures greater than 50 F (10 C) are recommended.

In the half-cell survey, the voltmeter reading is a measure of the potential of a complex physical situation that includes potential gradients associated with current flow between anodic and cathodic area superimposed on the potential associated with the conversion of iron to ferrous ions. If a significant amount of current passes through the copper/copper-sulfate half cell, the cell becomes polarized and its potential is no longer a constant +0.316 V at 77 F (25 C). In order to minimize current flowing in the circuit, a high impedance voltmeter (100,000 ohms/volt—or greater) should be used to prevent polarization of the reference half cell. It should also be noted that the potential indicated is the highest potential in the vicinity of the cell and not necessarily the potential of the nearest reinforcing bar. Thus a reading taken over a bar of low potential may be masked by the influence of a bar with a high potenital a short distance away. The same phenomenon occurs if the deck surface is wet, that is, corrosion activity is indicated but its location is not clearly defined. Consequently, readings should not be taken when there is free water on the deck surface.

Empirically, Spellman and Stratfull (*62*) found that measurements of half-cell potentials identified steel that was (a) corroding, or active, if the potential was numerically greater (more negative) than —0.35 V relative to the copper/copper sulfate reference cell (CSE) and (b) not corroding or passive if the potential was numerically less than —0.30 V CSE. These values were subsequently modified by Clear and Hay (*61*), who stated that for potentials numerically greater than —0.35 V CSE there is a 90 percent probability that reinforcing steel corrosion is occurring at the time of measurement. For potentials in the range —0.20 to —0.35 V CSE, the corrosion activity of the steel was found to be uncertain. For values less negative than —0.20 V CSE over an area, there is a 90 percent probability that the reinforcing steel is not corroding in that area. Potentials measured by other than copper/copper sulfate half cells should be converted to the copper/copper sulfate equivalent.

Field work in New York (*57*) has shown better than 90 percent correlation between potentials more negative than —0.35 V CSE and chloride contents above the corrosion threshold value. The correlation between potentials less negative than —0.20 V CSE and chloride contents less than the corrosion threshold value was almost 80 percent. The lower correlation value is also an indication that corrosion does not necessarily begin once the chloride concentration exceeds the threshold value.

Isolated high potential readings cannot be assumed to be totally reliable, because high transitory voltages sometimes occur (*63*). Data from reinforced slabs in an outdoor exposure plot suggested that, on the average, 5 percent of the potentials obtained in a single time survey may incorrectly signify steel corrosion. The point of attachment of the voltmeter has a significant effect on the magnitude of the potentials even when all the reinforcing steel is known to be electrically continuous. Consequently, if measurements are made to record the change in potential of a bridge deck with time, all surveys should utilize the same ground connection.

The numerical value of the potential increases with an increase in the amount of corrosion, but the potential is not a measure of the rate of corrosion (*50*). Similarly, the greater the area of active potentials, the more probable the amount of corrosion because both are time dependent (*64*). The potential associated with corrosion-induced cracks and delaminations is typically more negative than about 0.50 V.

Confirmation of the relationship between half-cell potentials, corrosion activity, and physical distress is well established (*42, 65*). In 1970, the Federal Highway Administration, Region 15, Demonstration Projects Division, began demonstrating and gathering data on half-cell potential measurements. Over a two-year period, 473 bridges were examined in 43 states. The more significant findings (*38*) were that as the area of the deck with potential measurements more negative than —0.35 V increases, the visible signs of deterioration also increase. The probability of severe deck deterioration also increases as the magnitude of the potential increases. Potential measurements greater than —0.35 V were frequently found in areas of bridge decks that appeared to be sound and uncracked. When concrete was removed in these areas on several structures, corroded steel was found. Consequently, even though a bridge appears in good condition, the steel can be actively corroding, although not necessarily to the point of causing concrete cracking and spalling.

If half-cell potentials are measured on a deck containing epoxy-coated reinforcing steel, the results need to be interpreted with care. The time to active corrosion of the steel can be used as an indirect measurement of the time taken by chloride ions to reach the level of the reinforcement. Once corrosion begins, the potential reading is characteristically very high whether the defect is a holiday or a major break in the coating because of the intense activity at a small anode. The rate of corrosion is almost certainly very slow because the cathode is also very small. This phenomenon is also evidence that a high potential reading is not synonymous with a high rate of corrosion.

Electrical Resistance Tests

Experience in the use of the electrical resistance test method had led to the adoption of "bench mark" resistance values to subjectively assess the effectiveness of deck seals. Spellman and Stratfull (*52*) originally suggested that an excellent waterproofing material would have an average electrical resistivity greater than 500 kΩ/ft² (5.4 MΩ/m²), a poor or perforated sealant or membrane would have a resistivity less than 100 kΩ/ft² (1.1 MΩ/m²), and the performance of installations having average resistivity values between these two extremes was considered questionable. Other investigators have established different criteria; those adopted by Van Til et al. (*66*) and Corkill (*67*) are compared with Spellman and Stratfull's criteria in Figure 10. When necessary, values have been changed from resistivity to resistance for comparative purposes. The fact that the test results have been interpreted differently indicates the lack of correlation between measured values and field performance. Some states have adopted criteria that do not rely simply on average values. For example, Oregon considers a membrane to be satisfactory if 80 percent of the resistance readings are greater than 500 kΩ and 100 percent are greater than 100 kΩ (*54*). A membrane is deemed unsatisfactory if 50 percent of the readings are less than 100 kΩ. The performance of membranes with readings between the two criteria is considered doubtful.

Laboratory tests by the Vermont Department of Highways have demonstrated a relationship between resistivity readings and chloride penetration through sealants and membranes (*68*). It was also found, however, that coatings having pinholes and blisters such that the average resistance readings were low were effective in slowing down the rate of chloride penetration. This suggests that readings well below the criteria illustrated in Figure 10 may be acceptable. In 1975 and 1976, core samples were taken from 131 locations on 51 bridges where resistance readings were taken and analyzed for chloride content (*53*). When 500 kΩ was used as the criterion for an effective waterproofing, there was a correlation between resistance readings and chloride intrusion data in approximately 60 percent of the measurements. In other words, high resistance readings were associated with no chloride intrusion and low resistance readings were associated with chloride intrusion about 60 percent of the time. Varying the acceptance criterion above and below 500 kΩ did not significantly affect the reliability factor of the test.

The lack of good correlation between resistance readings and chloride intrusion is an indication of the limitations of both the resistivity test and the sampling procedures for chloride determinations. The resistivity test measures the average value of the resistance over the area wetted by the contact solution, and the extent of this area is unknown. The chloride sample is, however, taken from a point that may or may not coincide with an imperfection in the sealant or membrane. The time at which a membrane failed is not known, and if failure occurred just prior to testing, chlorides may have had insufficient time to penetrate the concrete. Other events that may cause erroneous measurements of resistivity, such as short or incomplete circuits, were discussed earlier in this chapter.

The electrical resistance test has generally been accepted as an indicator of the effectiveness of a deck seal or membrane. When readings can be taken directly on the surface of the material, the presence of pinholes and other imperfections is indicated. When the test method is used in the field on bridge decks with an asphalt wearing course, the test method is considerably less reliable unless the asphalt is removed at the test locations to expose the surface of the membrane. The test method is less precise than the other test procedures described in Chapter Two. The method is more susceptible to error, and interpretation of the results is subjective. In the absence of other data, it is, however, a useful method of assessing sealant or membrane performance.

The test method should not be used on decks containing epoxy coated reinforcement. Although even a perfectly coated bar does not have a very high resistance, the test method is not sufficiently reliable to be useful with the addition of another variable.

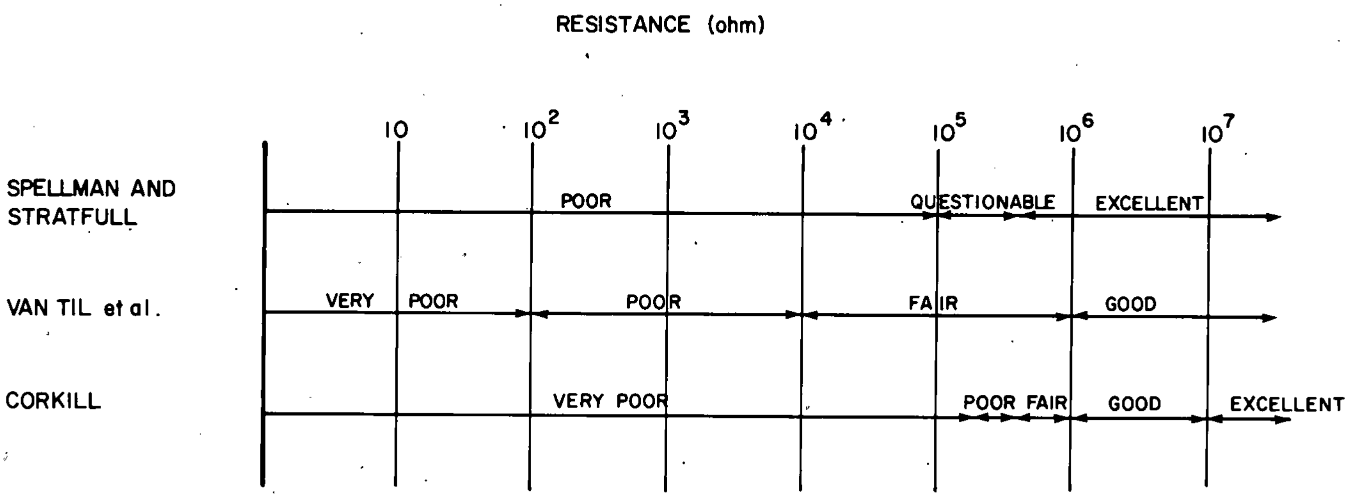


Figure 10. Comparison of criteria for effectiveness of deck sealants and membranes.

SUMMATION

The techniques described for evaluating the condition of bridge decks should be used in a discriminating manner to acquire the necessary information at minimum cost. Although guidelines can be developed, the details of any investigation have to be determined in accordance with the purpose of the study and the nature of the site.

A visual examination is the first stage of any condition survey. Where corrosion of the reinforcing steel is possible, the visual survey must be supplemented by physical testing, because the early stages of corrosion induced deterioration can be deceiving. On exposed concrete bridge decks, a survey using one of the delamination detecting devices will reveal subsurface fractures. A half-cell survey measures the extent and location of corrosion activity in a deck and also indicates the areas where further distress may occur. The results are normally plotted with respect to grid lines marked on the deck at 4- or 5-ft (1.2- or 1.5-m) intervals. The delamination and half-cell surveys are nondestructive and relatively quick to perform. These tests may be supplemented by coring to determine the condition of the concrete and to confirm the data obtained by the nondestructive testing. Samples for chloride analysis may be taken either by coring or by obtaining a pulverized sample from the level of the reinforcing steel by means of a rotary hammer drill. A pachometer is used to determine the location and depth of the reinforcing steel prior to coring or taking pulverized samples. A complete pachometer survey will reveal those areas of inadequate cover, where further distress may be anticipated, and where care may be needed if the use of rotary scarifying equipment is envisaged in deck repair operations.

The evaluation techniques are components of an overall evaluation system and not competitive options. No one device should be used to make an evaluation. Each technique has a specific purpose and certain limitations. The interrelationship of the results of the different tests has been confirmed a number of times (*50, 58, 61*).

There is a high probability that corrosion is taking place if the half-cell potentials in an area are greater than —0.35 V and the chloride content at the level of the reinforcing steel is above the corrosion threshold value. Corrosion is also associated with inadequate cover, but may occur with adequate cover where the deck concrete is of an exceptionally poor quality. Consequently, the use of the test methods as part of an overall condition survey enables anomalous readings from any one test method to be identified and the overall condition of the bridge deck to be ascertained.

Evaluating the condition of asphalt-covered decks is more difficult than if the deck surface is exposed concrete. Delamination and pachometer surveys are generally not satisfactory unless the asphalt wearing course is no more than about 1 in. (25 mm) thick and well bonded. Greater emphasis has to be placed upon visual examination, especially on the underside of the deck, and considerable experience is needed on the part of the inspector. More coring, often supplemented by dry sawing, is required than for exposed concrete decks. Half-cell potentials are a valid measurement of corrosion activity in the deck, providing that the asphalt is dry and holes are drilled through to the concrete surface. Where a seal coat or membrane has been placed on the deck surface, its effectiveness can be measured by the electrical resistance test method, provided that the circuit can be properly established at each test location.

CHAPTER THREE

TECHNIQUES FOR NEW CONSTRUCTION

INTRODUCTION

The hostile nature of the bridge deck environment was discussed in detail in Chapter One. Given these exposure conditions, the achievement of a maintenance-free bridge deck is a difficult goal. The imperfections of man and materials combine to make the goal even more distant. The solution to the problem of achieving bridge deck durability lies in a systems approach. Durable decks can be achieved only through careful design, proper selection of materials, and good construction practices. Improvements in any of these three areas may mitigate the problem but, independently, cannot provide the solution. Chapter Three discusses measures that can be taken at the time of design and construction that will help achieve a durable bridge deck.

DESIGN PRACTICES

It is not the sophistication of the structural analysis that primarily determines the durability of a bridge deck, but the detailing practices. Basic deck geometry does have an effect upon deck deterioration, particularly cracking. The incidence of cracking increases with span length (*8, 69, 70*), with angle of skew (*69*), and on continuous structures (*8, 70*).

The highway profile and crossfall influence the adequacy of drainage from the structure. Insufficient slopes make construction of decks without localized depressions or "bird-baths" difficult. Water, containing deicing salts, ponding in these areas accelerates the ingress of chlorides and promotes scaling of the concrete. Deterioration in the gutter areas is common on flat or almost flat bridges, because dirt and debris collect in these areas and trap brine on the deck (*71*). Sometimes deck drains are not provided because surface run-off cannot be discharged into streams or travelled areas beneath the bridge. In such cases, provision must be made for the water to run off the structure.

The size and location of deck drains need careful selection. A small number of large drains that do not clog with debris and that are less susceptible to freezing in winter is more effective than a larger number of small drains. Not only must the water be drained from the deck, but provision must be made to remove the water from the site. Deck drains that discharge on columns and designs that permit run-off to collect around columns or abutments can be worse than no drains at all. Water discharging through expansion devices can cause serious damage to abutments, bearings, and bearing seats. Unnecessary joints should be eliminated and, even when the joint is designed to be fixed or sealed, it is good practice to design the structure with pedestals for the bearings and without horizontal surfaces on piers and abutments on which water may collect.

The thickness of deck slabs is typically in the range of 7 to 10 in. (180 to 250 mm) depending upon the deck span. Span-to-thickness ratios in the range 10 to 20 are usual. Although there is no clear evidence that deterioration is more likely to occur in a thinner, as opposed to a thicker, deck slab (*72*), it has been suggested that once deterioration has started, it is likely to progress more rapidly in thinner decks (*71*). A conservative approach to deck thickness is prudent. A thicker deck allows more latitude in placing steel to achieve the design cover.

The depth of cover over the top reinforcing steel, as specified in the deck design, is the most significant factor contributing to the durability of the deck (*26, 55, 73*). An investigation into the effect of changes in design on the resistance of bridges to corrosion spalling has been undertaken by the Kansas DOT (*74*). It has been estimated that by increasing the cover from 2 in. (50 mm) to 3 in. (75 mm) and also by decreasing the water-cement ratio of the concrete from 0.44 to 0.35, the life of a deck can be tripled.

Measurements of the cover on bridge decks have shown that distribution of readings is approximately normal and the standard deviation is about ⅜ in. (10 mm) (*38, 75*), although exceptions have been noted (*76, 77*). The specified cover must account for construction tolerances. For example, if the anticipated standard deviation is ⅜ in. (10 mm) and 95 percent compliance with a minimum clear cover of 2 in. (50 mm) is acceptable, the average, and therefore the design or specified cover, must be approximately 2⅝ in. (67 mm). If 99 percent compliance is required, the specified cover must be 2⅞ in. (73 mm). After taking into consideration typical construction tolerances and specified clear covers of 2 in. (50 mm) for the top steel and 1 in. (25 mm) for the bottom steel, the minimum thickness for a deck containing 2 mats, each consisting of a #4 (13-mm diam.) and a #5 (16-mm diam.) bar, the minimum practical, though not necessarily the minimum desirable, deck thickness is 8 in. (200 mm) (*71*).

Deck reinforcement is currently determined by the AASHTO Standard Specifications for Highway Bridges, which contain empirical equations to represent the Westergaard analysis of bridge deck behavior. Laboratory tests (*78*) and field tests (*79*) have shown that for deck slabs with a span-to-thickness ratio of 15 or less (providing there is adequate restraint of the slab) the ultimate strength is considerably greater than that assumed under current design practices because of the development of membrane and arching effects in the slab. If the deck slab is designed to take advantage of this strength enhancement, the amount of steel in the deck slab can be reduced by up to two-thirds. This permits smaller bars to be used; consequently, larger cover-to-diameter ratios are attained and durability of the slab is increased. Implementation of these findings will be determined by the length of time needed to revise existing design codes.

CONSTRUCTION PRACTICES

A complete discussion of all the factors that comprise good construction practice is beyond the scope of this report. Details of bridge deck construction, including the requirements for preconstruction planning, inspection, falsework and formwork, reinforcement, concrete materials, placing, consolidating, finishing, and curing, are contained in recommended practice reports prepared by the American Concrete Institute (*71*) and the Roads and Transportation Association of Canada (*80*). The selection of materials and the relationship of deck deterioration to construction practices were also included in the first synthesis report on bridge deck durability (*1*). Discussion in this report is limited to the achievement of the specified cover with high-quality concrete. The reader is referred to the above reports for further information on other construction practices affecting bridge deck durability.

The quality of the concrete is of the utmost importance in determining the durability of a bridge deck. Careful attention must be paid to the selection of mixture proportions to keep the water-cement ratio to an absolute minimum. Several states have adopted a maximum water-cement ratio of 0.44, which corresponds to 5 gallons per 94-lb bag of cement. The American Concrete Institute recommends (*10*) a maximum water-cement ratio of 0.40 and a minimum cover of 2 in. (50 mm). Where local materials preclude the use of this water-cement ratio, a maximum water-cement ratio of 0.45 may be used provided the minimum cover is increased to 2½ in. (65 mm). Many mixtures are oversanded, thereby unnecessarily increasing the water demand and hence the water-cement ratio. The mixture should be proportioned by the trial mixture method utilizing actual job materials.

For many years, an attitude has prevailed that if the requirements for the specified strength are satisfied, the deck will perform adequately. It is much more difficult to design for durability. The most important factors that determine the durability of concrete are the selection of good quality materials and the provision of a low water-cement ratio and air entrainment. Not only must the air content comply with the specification, but the parameters of the air-void system must be within recommended limits.

Concrete is considered to have an adequate air-void system if, when tested in accordance with ASTM C 457, the spacing factor is no greater than 0.008 in. (0.20 mm), the specific surface is greater than about 600 in.^{2}/in.^{3} of air-void volume (24 mm^2/mm^3), and the number of air voids per inch of traverse is more than twice the numerical value of the percentage of air in the concrete (*71*). If the water-cement ratio is appropriate to the exposure conditions, even with 7 or 8 percent air content, the strength of the concrete will rarely be a consideration (*81*), especially in thin slab-on-beam structures. The water-cement ratio determines not only the rate of penetration of chlorides through the concrete but also affects bleeding and plastic shrinkage.

The in-place quality of the concrete is determined by its porosity, which is controlled by the initial water-cement ratio, the degree of consolidation, and the degree of hydration, which is a function of the adequacy of curing and the age of the concrete.

Bridge deck durability is also controlled by the cover over the reinforcing steel. The specified cover must be adequate, and construction procedures must ensure that the specified cover is achieved. The reinforcing steel has to be placed accurately and firmly secured against displacement during the placing of concrete in the deck. This is done by tying the steel frequently at intersection points and providing rigid support systems for each mat of reinforcement. A safe rule is to tie every second bar intersection with wire of not less than 16 gage (1.6 mm diam). If possible, all top mat intersections should be tied. Each mat of steel should be held down and, where shear connectors are present, it is good practice to tie both the top and bottom mats to the shear connectors.

The use of a mechanical finishing machine is highly desirable and, except on the smallest decks, is essential to provide an acceptable riding surface on exposed concrete decks. Consideration should be given to specifying a machine that is heavy enough that it strikes off the concrete at the required grade and stiff enough not to deflect and reduce the cover. Rail supports should be located directly on the main structural members. The structure will deflect during the placing operation and allowance must be made for anticipated settlement and camber. Deflections should be calculated and control points checked during the placing operation. The spacing of the rail supports should also be specified to ensure that the rails do not deflect between the supports under the weight of the finishing machine.

A "dry-run" procedure is necessary to check for clearance between the screed on the finishing machine and the reinforcing steel. Flexible strips or "tell-tales," equal length to the specified clear cover, are normally attached to the bottom of the screed. As the machine traverses the deck, any necessary adjustments are made to the reinforcement to provide the minimum specified cover.

The importance of checking and careful planning of the deck placing procedures cannot be over-emphasized. A preconstruction conference should be held to discuss the method of deck construction (*82*). As a minimum, the conference should review the method of supporting the reinforcement, the procedure for checking the steel in-place, the rate of concrete placement, personnel and equipment to be used, type of finish, details of the curing, contingency plans for adverse weather conditions, and methods to be used to assure conformity with the specified water-cement ratio (*83*). The method of consolidation is also crucial because improper consolidation not only accelerates damage from frost and corrosion of the reinforcement (*28*) but also completely negates the benefits of using a low-cement ratio concrete.

The significance and finality of the placing operation is well-described in the first synthesis report on deck durability (*1*):

> The casting of a concrete slab takes only a few hours but requires many days of preparation. The cost of the freshly mixed concrete is only about 10 percent of the total slab cost. Yet the placing of that material at that time is an essentially irreversible act creating enormous pressures on those involved in the decision-making process.

It might also be added that the superstructure concrete generally accounts for only 23 percent of the total bridge concrete, but the superstructure concrete accounts for 99 percent of the bridge concrete maintenance costs (*81*).

NONCORROSIVE REINFORCING STEEL

The susceptibility to corrosion of the reinforcing steel commonly used in bridge decks is not significantly affected by its composition, grade, or level of stress (*27, 84*). Natural weathering steels do not perform well in a concrete containing moisture and chloride (*10*). Stainless steel bars are manufactured in South Africa (18 percent Cr, 8 percent Ni) and England (18 percent Cr, 10 percent Ni, 3 percent Mo). The bars have been used only in special applications, especially in the attachment of cladding panels on buildings. Even with domestic production, their use in bridge decks would be uneconomical.

Stainless-steel-clad reinforcing bars are being tested in the FHWA time-to-corrosion study. The cladding (17.6 percent Cr, 10.3 percent Ni) was achieved by application of a layer of stainless steel to a steel bloom prior to rolling into reinforcing bars; the result was a surface layer of Grade 316 stainless steel approximately 20 mils (0.5 mm) thick. After 20 months of daily salting, red rust staining and corrosion-induced concrete cracking were present on both the slabs containing the stainless-clad bars and the control slabs containing conventional black reinforcing steel, although the amount of deterioration was less for the slabs with stainless-clad bars. Testing is continuing to determine whether corrosion of the clad bars was confined to black steel corrosion at defects in the coating or whether corrosion of the cladding occurred.

Pennsylvania used the stainless-clad bars in both mats of one span of an experimental bridge deck in 1976. No findings are yet available.

COATED REINFORCING STEEL

An alternative and more economcial solution than noncorrosive reinforcing bars is to apply a coating to conventional reinforcing steel. A stable coating isolates the steel from contact with oxygen, moisture, and chloride ions, thus preventing corrosion. The concept has the merits of simplicity and ease of implementation, because it requires very little change in construction procedures.

The selection of suitable coatings has been the subject of several investigations (*30, 85-88*). The coating must be easily applied, be durable in the service environment, not impair the structural properties of the steel, and be economical. If the coating is to be applied in a plant, it is essential that the coated bars be easily transported without damage. A dielectric coating is preferable to completely isolate the steel from the potentially corrosive environment in the concrete.

Nonmetallic Coatings

Numerous nonmetallic coatings have been evaluated, including coal-tars, epoxies, asphalts, urethanes, vinyls, and rubbers, and the characteristics of these materials have been summarized (*30, 87*). A detailed investigation of nonmetallic coatings was completed by the National Bureau of Standards under the FHWA contract research program. In the course of the investigation (*86, 89*), 47 nonmetallic coatings were evaluated: 21 liquid and 15 powder epoxies, 5 polyvinyl chlorides, 3 polyurethanes, 1 polypropylene, 1 phenolic nitrite, and 1 zinc-rich coating. Four powder-epoxy coatings were found to be the best candidates for protecting steel reinforcing bars from corrosion. The optimum thickness of the epoxy coating was found to be 7 ± 2 mils (0.18 ± 0.05 mm) with respect to corrosion protection, bond strength, creep characteristics, and flexibility. Performance criteria were developed for the evaluation of similar coatings and, subsequently, formal requirements for the prequalification of organic coatings for reinforcing bars were prepared (*90*).

The process of coating reinforcing steel developed from the application of epoxy coatings to pipe used by utility companies and the petroleum industry (*91*). The bars are first heated, often by open flame, to approximately 450 F (230 C). The primary heat treatment aids in the removal of mill scale, rust, and grease. The bar is blast cleaned to a near white finish by grit or shot. It is then heated in an oven until the bar temperature is constant at the temperature required for application of the epoxy powder, usually 400 to 450 F (200 to 230 C). The reinforcing bar is passed through an electrostatic spray that applies the charged dry epoxy powder on to the steel. The epoxy melts, flows, and cures on the bar, which may be cooled in air or by water quenching.

Once the bar has cooled, it is tested with a holiday detection device that electrically examines it for minute cracks or pinholes in the coating. If a holiday is detected, the area is marked and the holiday subsequently touched up with a liquid epoxy that is compatible with the powdered-epoxy coating.

The four coatings identified as most suitable by the laboratory evaluation were recommended for use in the National Experimental and Evaluation Program (NEEP) Project Number 16, Epoxy Coated Reinforcing Steel. The first installations of epoxy-coated bars in bridge decks were in 1973. The main difficulties encountered in implementing use of the coated bars were damage to the coating during transportation and handling, and cracking due to inadequate preparation of the bar and to bending after coating. Initially, the bars were not heated before entering the shot mill, with the result that the blast cleaning was inadequate. Addition of the primary heat treatment and additional blast cleaners on the production line has solved the problem of inadequate preparation.

Damage during transportation and handling can be minimized by increasing the frequency of supports during shipping to prevent bar-to-bar abrasion and by using padded bundling bands and nylon slings for loading and unloading. Bars in place are not damaged by workmen walking them, but supporting chairs and tie wires need to be plastic-coated to prevent cuts in the coating. Although the coatings were designed to withstand fabrication after application, cracking was observed in the bend areas on some projects (*91*). Many coaters now can coat prefabricated reinforcement, and this has eliminated the problem. The use of straight bars, however, is preferred, because this not only facilitates the coating but the transportation and placing of the bars.

Most specifications require that all damaged or exposed areas such as sheared ends, cracks, cuts, or holidays be patched with the approved liquid-epoxy repair material. Initially an attempt was made to obtain a coating completely free from defects, but the touch-up work required to do this was both tedious and expensive. Tests included in the FHWA Time-to-Corrosion Study showed no failure after 35 months of exposure to severe corrosive conditions of specimens fabricated with epoxy-coated bars in which major nicks and cuts were deliberately made in the coating (*92*). This work has lessened the concern that a small exposed area of steel would be susceptible to intense corrosion activity and has enabled the specifications for patching to be relaxed. Increased emphasis has been placed on proper handling procedures, and coating repair is neither required after fabrication unless damage exceeds 2 percent in straight areas or 5 percent in bent areas nor after placing unless the damage exceeds 3 percent of the coated area.

By March 1976, the number of prequalified coatings had increased to 10 and the number of states participating in the NEEP survey to 19 (*90*). By the fall of 1977, 17 states had adopted the use of epoxy-coated bars as a standard construction procedure in some structures and 9 others had installed coated bars on an experimental basis. As the use of epoxy-coated bars has substantially increased and relaxation of the specification has been possible, the cost of the bars has decreased. In 1974, the additional in-place cost of the coated bars was reported as $0.39/lb ($0.86/kg) (*93*), in late 1975 as $0.25 to $0.30/lb ($0.55 to $0.66/kg) (*90*), and on most contracts in 1977 the premium was $0.15/lb (0.33/kg). Prices quoted are typically for #6 (19-mm diam.) bars. The premium for the use of epoxy-coated bars per square foot (m^2) of deck area will be further reduced if the quantity of deck reinforcement is decreased in accordance with the findings discussed earlier in this chapter. In the United States, 38,000 tons (34,000 Mg) of epoxy-coated reinforcing steel had been used up to the end of 1978.

It is common practice to coat only the top mat of steel in the deck and those bars in other elements, such as curbs and barrier or parapet walls, requiring protection against deicing salts. The use of epoxy-coated bars assures protection against corrosion of the reinforcing steel. Use of coated bars does not, however, permit relaxation of the construction specifications for the remainder of the deck. The clear concrete cover over the bars should be at least 2 in. (50 mm) to minimize the possibility of corrosion at any defects in the coating. The use of a high quality, air-entrained concrete is essential to ensure durability of the deck slab.

There are no reliable, nondestructive performance evaluation tests for inplace epoxy-coated bars. Alternating current resistance measurements are feasible provided that wiring connections are made to the bars at the time of construction (*94*). Electrical potential readings can be used to indicate the time to initial corrosion, and several states are routinely monitoring installations of epoxy-coated bars, though no deficiencies in performance have yet been reported.

Metallic Coatings

Metallic coatings for the protection of steel reinforcing bars fall into two categories: sacrificial and nonsacrificial. In general, metals that are higher than iron in the electrochemical series, such as zinc and cadmium, give sacrificial protection to the iron. If the coating is damaged and the steel is exposed, a galvanic couple is formed in which the zinc or cadmium becomes the anode and steel becomes the cathode. Where no break occurs, the coating acts as a barrier to protect the steel. The nonsacrificial coatings are formed from metals more noble than iron, such as nickel and copper. These metals can protect the steel only when the coating is unbroken. If the steel is exposed, it will become anodic, the coating will act as the cathode, and rapid corrosion of the steel may result. However, the activity of metals in the highly alkaline environment of concrete is not necesarily determined by their relative position in the electrochemical series. It has been suggested, for example, that zinc will not always be sacrificial to steel because of the insulating corrosion products that may form on the zinc surface and the slight reversal of polarity that may occur (*95*).

With any metallic coating, there is always the possibility that it may corrode because of anodic and cathodic areas forming, even if the steel is not exposed, through a mechanism similar to that described in Chapter One for uncoated steel in concrete. The uncertain life of the coating and the possibility that corrosion of the coating will cause distress in the concrete are the most serious limitations on the use of metallic coatings for reinforcing bars.

Copper has been found to corrode rapidly in an alkaline chloride environment (*85*) and, consequently, is not a suitable coating. The corrosion resistance of nickel is high in alkaline chloride solutions, and even if breaks occur in the nickel coating, corrosion of the steel is not appreciably accelerated (*85*). Nickel-coated bars, produced by hot-rolling nickel-coated steel billets, have been tested over an 11-year-period in a marine environment (*88*). The results showed that the nickel coating was effective in delaying, and in some cases preventing, corrosion of the bars. The economic viability of nickel-coated bars is uncertain, and the bars are not commercially available.

Of the sacrificial coatings, cadmium has been identified as a suitable coating on the basis of laboratory work (*96*), but field performance data are not available. Conversely, zinc-coated, or galvanized, bars have been the subject of numerous laboratory (*85, 95, 97, 98, 99*) and field studies (*100-103*). The hot-dip galvanizing process consists of pickling the steel to clean it and then immersing it in a kettle of molten zinc. The zinc is metallurgically bonded to the steel, and the coating consists of an outer zone of pure zinc and a number of transition zones of zinc-rich alloys encasing the underlying steel. The thickness of the coating is usually not less than 3.4 mils (0.086 mm). In fresh concrete, zinc reacts with the alkalies in portland cement to release hydrogen gas. Traces of chromate will passivate the zinc surface, and galvanized bars are normally dipped in a chromate bath to prevent hydrogen formation around the bar in the concrete.

Results of the performance of galvanized reinforcing bars in concrete have been conflicting. In laboratory work, Cornet and Bresler (*97*) found that cracks developed later and grew more slowly over galvanized reinforcement than over black steel. Hill, Spellman, and Stratfull (*95*) found that in concrete typical of the quality used in bridge deck construction, galvanized steel caused cracking earlier than untreated steel.

It is known that the zinc coating will corrode in concrete (*88, 104*) and that intensive pitting can occur under conditions of nonuniform exposure in the presence of high chloride concentrations (*99*). Because of the possibility of galvanic action between galvanized bars and uncoated steel in the same structure, it is normal practice to coat all the steel in a deck slab. This includes the use of galvanized tie wires and either galvanized or plastic chairs. The necessity to coat all the steel in a deck slab when using a metallic coating contrasts with the use of inert nonmetallic coatings where only the steel in chloride-contaminated areas of the deck need be coated.

The length of time during which the zinc will afford protection to the steel under field conditions is uncertain. In an attempt to determine a useful life for galvanized bars, a study was initiated in 1974 to investigate the condition of those decks containing galvanized reinforcement that had been in service the longest and exposed to either a marine environment or deicing salts (*102*). Although none of the decks were showing visible signs of distress, the chloride contents of the concrete at the level of the reinforcing bars were low, making it difficult to reach firm conclusions on the effectiveness and life-span of the galvanizing. One of the difficulties in establishing the performance of galvanized bars is that there does not appear to be a half-cell potential value that discloses the activity of galvanized steel in concrete. Consequently, performance is determined by taking core samples, measuring the chloride ion concentration at the level of the steel, and determining the thickness of the zinc coating remaining on the bar. Over a six-year period, accelerated field studies in Michigan (*103*) have shown that galvanizing will retard the formation of delaminations and spalls but will not prevent them, especially where there is only shallow cover to the reinforcement.

Galvanized reinforcing bars were placed in bridge decks in many states on an experimental basis in the early and mid-1970s, and additional in-place costs as low as $0.14/lb ($0.31/kg) were reported (*90*). In view of the uncertain effectiveness of galvanized bars in providing long-term protection against corrosion-induced damage, in 1976 the Federal Highway Administration limited installations containing galvanized bars to a maximum of three bridge decks per state (*105*). There are more than 200 decks in the United States containing galvanized reinforcement, most of which are in Pennsylvania, and these will continue to be monitored. It is anticipated that this policy will remain in effect until definite conclusions can be drawn from laboratory studies and from existing field installations.

CORROSION INHIBITORS

A corrosion inhibitor is an admixture to the concrete used to prevent the corrosion of embedded metal. The mechanism of inhibition is complex, and there is no general theory applicable to all situations.

The effectiveness of numerous chemicals as corrosion inhibitors for steel in concrete (*27, 61, 106, 107*) has been studied. The compound groups investigated have been primarily chromates, phosphates, hypophosphites, alkalies, and fluorides. Some of these chemicals have been suggested as being effective; others have produced conflicting results in laboratory screening tests. Many inhibitors that appear to be chemically effective produce undesirable effects on the physical properties of the concrete, such as causing a significant reduction in compressive strength. More recently, calcium nitrite has been reported to be an effective corrosion inhibitor (*108*), and exposure plot studies are continuing.

Admixtures used to prevent corrosion of the steel by "waterproofing" the concrete, notably silicones, have been found ineffective (*61*).

Preventing corrosion through the use of a chemical admixture to the concrete is appealing because of its simplicity and its negligible effect on design and construction practices. Although corrosion inhibitors may be useful in the future, considerably more research is required to establish long-term performance before widespread use can be contemplated.

CHAPTER FOUR

TECHNIQUES FOR NEW CONSTRUCTION AND REPAIR

INTRODUCTION

A number of approaches to solving the bridge-deck problem can be applied either at the time of new construction or during repair of an existing deck. Such systems involve the placing of a sealant, membrane, or overlay, or a combination of the former on the deck surface. The construction techniques, except for surface preparation, are the same for repair work as they are for new construction.

This chapter discusses the design, construction, inspection, and performance of sealants, impregnants, membranes, and overlays. Discussion of the methods and extent of concrete removal and the performance of these systems on chloride-contaminated decks is contained in Chapter Five.

SEALANTS

Several investigations have been undertaken to determine the effectiveness of sealants in improving the durability of concrete. Many of these studies (*109-113*) have included the evaluation of linseed oil, which has been widely used by highway agencies to prevent deterioration of concrete pavements and bridge decks. Some states routinely apply two coats of 50/50 mixture of boiled linseed oil and either mineral spirits or kerosene to new construction. In some cases there are further applications at regular intervals, especially in the first few years after construction. The results of the investigations differ in the value of the linseed-oil treatment. The main reason for the differences is that the results are compared with a control mixture that is of an arbitrary quality and may or may not be air-entrained. Many of the early investigations were concerned only with the resistance of concrete to scaling, and it has been demonstrated that linseed oil treatments are effective in reducing the scaling of improperly air-entrained concrete.

The depth of penetration of linseed oil depends upon the quality of the concrete and its moisture condition at the time of application. Penetration depths in the range 1⁄16 in. (1.5 mm) (*114, 115*) to 1⁄8 in. (3 mm) (*116, 117*) have been reported. Linseed-oil emulsion curing compounds have been found to penetrate up to 1⁄4 in. (6 mm) (*118, 119*). Nevertheless, the life of the treatment is limited in any area subject to traffic wear, especially where studded tires or chains are used. If an asphalt wearing course is placed, the linseed oil substantially reduces the bond between the asphalt and the concrete (*115*).

Only recently has the effectiveness of linseed-oil treatment in preventing the corrosion of steel in concrete been investigated. The FHWA time-to-corrosion studies (*22, 26*) and a study on a bridge deck in Vermont (*120*) have shown that linseed-oil treatment will retard chloride penetration but not prevent it. Furthermore, the treatment is effective for only a few years. Half-cell measurements on the bridge deck showed that the steel was actively corroding in about 9 percent of the deck area within 5 years of construction. It has also been shown (*115, 120*) that linseed oil is ineffective in resisting moisture penetration of concrete.

Treatment of concrete with linseed oil is inexpensive, less than $0.10/ft² ($1.08/m²) and, as such, offers relatively cheap insurance against scaling where the quality of the concrete is marginal. It is not, however, a substitute for proper air entrainment and good finishing and curing practices. Its benefits are short lived on exposed deck surfaces unless it is regularly renewed. It will not prevent corrosion of the reinforcing steel. Where linseed oil treatment is used, a period of air drying should precede application and concrete temperatures should be above 50 F (10 C) to hasten drying. The deck surface will be slippery when the linseed oil is first applied (*121*), and sand is not effective in improving the skid resistance during the drying period (*172*).

Numerous other sealants have been investigated, including a wide variety of resins, petroleum products, silicones, vegetable oils, and other organic materials (*111, 123-126*). Some of the products were clearly of no benefit, and none of the others could compete with linseed oil treatment when both effectiveness and economics were considered.

Epoxies have, perhaps, been the most widely promoted of the alternative materials, both as penetrating sealants and as surface seal coats for the prevention of deck deterioration. Penetration into the deck is comparable to that of linseed oil (*127*), and the products are considerably more expensive. Experiences with epoxy resin seal coats in Kansas over a number of years were unsatisfactory (*128*), and laboratory studies have shown that thin epoxy seal coats are not impermeable (*115*). An interim report on NEEP Project 12, "Bridge Deck Protective Systems" (*129*), concluded that epoxy seals have not been satisfactory and recommended against their continued use in experimental projects.

IMPREGNANTS

Many of the deficiencies of sealants can be overcome if the depth of the sealant's penetration is increased, thereby increasing the life of the treatment. Furthermore, if all the pores of the concrete are permanently filled with an impermeable material, chloride is prevented from reaching the reinforcing steel. Deep penetration can only be achieved if the concrete is first dried and then soaked with a low-viscosity sealant for several hours. Most of the development of deep-sealing techniques has occurred as a result of research in polymer-impregnated concrete.

The idea of filling the voids of hardened concrete with a monomer and polymerizing in situ originated in the Bureau of Reclamation in 1965 (*130*). Considerable research was undertaken over the next several years to define the properties of polymer-impregnated concrete (*131-134*) and to develop concrete polymer materials for use in bridge deck construction and repair (*135, 136*).

Laboratory studies have demonstrated that polymer-impregnated concrete is strong (*130, 137*), durable (*130*), and almost impermeable to deicing salts (*26, 138, 139*). For maximum polymer loading, the production of polymer-impregnated concrete consists of the following processes:

1. Casting and curing concrete using normal procedures.
2. Drying the concrete to remove all the evaporable water.
3. Vacuum soaking the concrete in a low-viscosity monomer under pressure.
4. Polymerizing the monomer in the voids of the concrete and simultaneously preventing evaporation of the monomer.

Polymerization, the process by which the individual molecules of the monomer are caused to join together to form a plastic, can be accomplished by the use of gamma radiation or chemical initiation. Only the latter method is feasible for field applications. The common initiators, which are the organic peroxides and azo compounds, decompose under the action of heat or a chemical promoter. This decomposition generates free radicals, which then cause the monomer units to join together in a chain reaction known as addition polymerization. The rate of polymerization can also be increased by the use of multifunctional monomers.

Work has also been undertaken to develop monomer-initiator-promoter systems that polymerize at ambient temperature (*140*). Although the monomer systems developed have the advantage that no external source of energy is required during the polymerization cycle, there are difficulties in obtaining predictable polymerization times and in matching the monomer saturation time to the onset of polymerization. In practice, this means that if the monomer-initiator-promoter system has a relatively short gel time, the viscosity becomes too high for the impregnant to penetrate the concrete to the required depth.

The effectiveness of any particular polymer impregnation depends upon the degree to which the ideal processing conditions are compromised. Adequate drying is essential to the achievement of good mechanical properties (*141*). Techniques have been developed for partial polymer impregnation, in which only one surface of the concrete is dried and impregnated, often to a depth of about 1 in. (25 mm). Partially impregnated slabs also have excellent resistance to penetration by chlorides (*26, 142*).

The choice of monomer is determined by a complex interplay between the requirements of viscosity, vapor pressure, rate of polymerization, safety in handling, physical properties, and cost. The monomer most widely used in highway applications is methyl methacrylate, often with the addition of 5 percent by weight of a cross-linking agent, trimethylolpropane trimethacrylate.

All full-scale applications of polymer-impregnated concrete have been experimental and have involved partial, sometimes referred to as surface, impregnation. Typical procedures (*135*) commence with the cleaning of the deck, preferably by sandblasting. The deck is then dried using open-flame, infrared, or space heaters. After the deck has cooled, a thin layer of dry sand is placed on the deck surface and the sand is saturated with monomer containing an initiator. The monomer, cross-linking agent, and initiator must be stored with care and mixed in small batches just prior to use. The initiator should be stored in a refrigerated container. Where the grade of the deck requires it, the monomer must be ponded in the sand by the construction of dikes. The saturated sand is covered with polyethylene sheeting to prevent evaporation of the monomer, and the deck is allowed to soak overnight. If the sand becomes dry, additional monomer is added. The monomer is polymerized by maintaining the deck temperature between 140 and 175 F (60 and 80 C) for 2 hours using steam, ponded hot water, or forced-air heaters. Open-flame or infrared heaters are not recommended for polymerization because the monomer is flammable and there is danger of explosion. Water added to the sand inhibits evaporation of the monomer and bonding of the sand to the deck. Any sand that does become bonded must be removed where this reduces the quality of the riding surface.

The first application of polymer impregnation on a bridge deck was a one-year-old deck in Austin, Texas, in 1973 (*143*), and the objective of polymerization to a depth of 1 in. (25 mm) was achieved. A full-scale surface impregnation was completed on a new deck in Denver, Colorado, in 1974 (*144*). The only problem identified was the development of map cracking in the bridge deck; further research and field trials have not yet succeeded in eliminating this problem. Further development of the impregnation procedures in Texas (*145*) led to a contract for the impregnation of two new structures. Bid prices on the two structures ranged from \$0.56 to \$1.67/ft² (\$5.97 to \$17.92/m²) on a 48,000 ft² (4470 m²) deck and from \$1.11 to \$5.56/ft² (\$11.95 to \$59.74/m²) on a 8,700 ft² (810 m²) deck. The decks were completed in 1978 and the effectiveness of the treatment is being evaluated.

In NCHRP Project 18-2, "Use of Polymers in Highway Concrete," techniques for deep polymer impregnation were investigated. The procedures developed approach more closely the ideal laboratory processes and include more complete drying of the deck, monomer penetration under pressure, and longer soak times (*136*). Small-scale field trials were completed but there have been no full-scale installations.

A feasibility study of using polymer-impregnated, prestressed panels for deck construction found the concept technically feasible, but costs were substantially greater than conventional deck construction, and the findings were not implemented (*146*).

Considerable research effort has been expended in developing concrete polymer materials for highway applications. At the present time, there have been few full-scale installations, and the techniques cannot be considered operational. Some of the disadvantages of polymer-impregnated concrete are that the monomers are expensive and volatile and the process includes several lengthy procedures. Consequently, attempts have been made to identify other materials that can be used to fill the pores of the concrete and that preferably do not require a polymerization cycle.

Among the materials that have been evaluated are sulfur (*147, 148*) and linseed oil (*149*). Although sulfur eliminates the polymerization cycle, it must be maintained at between 120 and 160 F (49 and 71 C) during the soak period, and field processing presents environmental difficulties.

Small-scale field trials have demonstrated the feasibility of impregnating concrete with linseed oil to a depth of 2 in. (50 mm) or more, but the procedure is both lengthy and relatively expensive, and its effectiveness in preventing corrosion of embedded reinforcing steel is unknown. Preliminary cost estimates were \$3.53/ft² (\$37.85/m²) exclusive of profit (*149*). There have been no full-scale installations of either sulfur or linseed oil impregnation.

POLYMER OVERLAYS

Polymer materials have been used as both patching materials and thin overlays on bridge decks. The difference between these systems and surface sealants is that the polymer is extended through the use of fillers, usually fine aggregate, and the polymer mortar is applied in a thickness of approximately ½ in. (13 mm). The filler reduces costs, imparts skid resistance and reduces the coefficient of thermal expansion of the polymer. The requirements for thin overlays are the same as for any other overlay (*150*), viz.: (a) low permeability to chlorides, oxygen, and moisture; (b) durable and resistant to wear; (c) good skid resistance; (d) good bond to concrete; (e) thermally compatible with deck concrete; (f) easy to apply; (g) capable of bridging existing and new cracks; and (h) inexpensive.

The potential advantages of a thin overlay are that a minimum of material is used, thereby minimizing costs and additional dead load on the structure. Thicknesses up to ½ in. (13 mm) can usually be accommodated without modification to the expansion joints or building up the approaches, which results in significant cost savings.

Epoxy and polyester mortars have been the materials most widely used as thin overlays, and they have been shown to be effective in preventing moisture penetration under laboratory conditions (*115*). The electrical resistance test is a particularly good measure of the permeability of a polymer overlay because most polymers are dielectric materials. In the field, the deck surface must be clean and sound. Epoxies, and most other polymers, have a low tolerance to moisture and cold conditions, requiring a dry substrate and temperatures of at least 40 F (4 C). Thorough mixing of the polymer and activator are necessary before the filler is added. Improper mixing of the two components of the polymer has been a common source of field problems. In general, relatively small batches should be mixed and applied promptly. The sand must be surface dry, otherwise the overlay will not harden. Workmen should use skin cream or rubber gloves and wear protective clothing when working with epoxies and some other polymers because of the potential for skin sensitization and dermatitis (*151*).

A bond coat of neat polymer is usually applied ahead of the polymer mortar. One of the major construction problems has been blistering of the overlay before it has cured. Blistering is caused by the expansion of air, and sometimes water vapor, within the concrete after application of the overlay. It commonly occurs in membranes and is discussed later in this chapter in that context.

A number of successful installations of polymer overlays have been reported (*152, 153, 154*), but the majority of installations have lasted only a few years (*155*). In the early 1960s, polyester overlays containing chopped glass fibers were installed (*156*), but their use was discontinued after a few experimental installations. Most of the overlays have been used to rehabilitate deteriorated decks, except where they have been used on newer decks for the sole purpose of providing adequate skid resistance. Even the concept of a thin overlay has been challenged because it is doubtful whether a thin overlay will withstand heavy traffic loadings (*157, 158*). Neither is it realistic to specify a nominal thickness of ½ in. (13 mm) because of irregularities present in most bridge decks. If a minimum thickness of ½ in. is required, the average thickness will be much greater, thereby significantly increasing the material costs. Other problems have been rapid wear (*159*), lack of flexibility at low temperatures (*66*), differential thermal expansion, and autogeneous shrinkage (*157*), which cause cracking of the overlay. The coefficient of expansion of epoxy mortar may be five times that of conventional concrete (*160*). In view of the mixed, though generally unsatisfactory experience with polymer overlays, further installations must be considered highly experimental.

CONCRETE OVERLAYS

Concrete overlays may be applied as the second stage of construction on a new deck, as preventative maintenance on a deck that has been open to traffic for a short time but was built without a deck protective system, or in the rehabilitation of existing, deteriorated decks. Two-stage construction is not new; it was used in some parts of the country more than 40 years ago as a matter of convenience, sometimes with a membrane between the two lifts of concrete. In two-stage construction, the first lift of concrete is placed to cover the top of the reinforcing steel and the overlay usually is placed a few days later, though there have been cases where the second-stage concrete has been placed before the first stage concrete has set.

Several potential advantages of concrete overlays can be identified:

1. The overlay can be tailor-made to provide the required thickness of highstrength concrete and maximum durability at the deck surface.
2. Properly-proportioned and consolidated concrete is effective in retarding the penetration of chloride ions.
3. The overlay is an integral component in the load-carrying capacity of the deck.
4. The use of a concrete overlay permits vapor exchange between the concrete and the environment, preventing the build up of vapor pressure that occurs beneath an impermeable membrane.
5. The overlay is thermally compatible with the base concrete and absorbs less solar radiation than an asphalt overlay.
6. A smooth riding surface can be provided because minor irregularities in profile and crossfall can be corrected and dead-load deflections are minimal.
7. High-quality aggregates can be incorporated in the concrete mixture to provide good wear and skid resistance at little additional cost.
8. In new construction, the overlay assures adequate cover to the reinforcing steel. Furthermore, the cover is free from cracks directly over the reinforcing bars.
9. In repair work, the overlay will fill in areas of concrete removal without the need for a separate placing operation. Work can proceed while part of the deck remains open to traffic.

The viability of using concrete overlays has been demonstrated in the laboratory by casting overlays on vibrating beams to simulate the effect of vibration caused by traffic in an adjacent lane (*161*). Epoxy, latex, cement paste, and mortar have all been investigated as bonding agents (*161, 162*) and found to be satisfactory provided that the base is properly prepared. Sand or water blasting is considered to be the minimum treatment required to ensure a clean, sound surface, free from contaminants and laitance. In two-stage construction, resin curing compounds should not be used on the first stage concrete because they can be difficult to remove by blast cleaning and increase the possibility of contamination. Several years ago, etching of the base with dilute hydrochloric acid followed by vigorous brushing and neutralization by flushing with ammonia, caustic soda, or water was a common method of surface preparation (*163, 164, 165*). The addition of chlorides to the concrete was a questionable practice, and the difficulty of disposing of the surface run-off has caused those techniques to be superseded by mechanical methods of surface preparation.

There has been some controversy as to whether a bridge deck should be wetted prior to applying a bonding agent. Latex bonding agents should be applied to a prewetted surface (*166*) because the water assists penetration of the latex particles into the deck concrete and prevents the latex from rapid drying, which would result in film formation and impairment of bond strength. For epoxy bond agents, the concrete surface should be dry. Laboratory studies with cement paste and mortar bonding agents have shown that the bond strength is not significantly affected by the moisture condition of the base (*162, 163*), though slightly higher bond strengths are achieved if the base is dry.

In the field, the use of epoxy bonding agents has been limited by their expense and variable performance, especially where the epoxy has contained a volatile component. Cement paste and mortar bonding agents have been used satisfactorily on both dry and prewetted decks. Ideally, the deck should be kept wet for several hours before placing the overlay and the contact surface allowed to dry prior to actual concrete placement. This prevents the hardened concrete from drying out the bonding layer or the weakening of the bond because of excess water on the deck surface. The decision to prewet the deck also involves a practical consideration. Not only is wetting an extra operation, but excess water must be removed from the deck. The additional cost is offset by the extra control that is necessary to prevent the grout from drying when applying a bonding agent to a dry deck prior to placing the concrete. On existing decks, where steel is exposed and the deck surface is irregular, a dry deck is preferable because the water promotes rapid oxidation of sandblasted bars and it is difficult to remove puddles of water from depressions in the deck surface.

The reason that adequate bond strength can be achieved in a number of different ways is that, provided the deck is properly prepared, bond strengths are considerably in excess of the maximum shear stress at the bond line. The horizontal shear at the interface between a 7-in. (180-mm) thick uncracked slab and a 2-in. (50-mm) thick overlay has been estimated to be 64 psi (440 kPa) under an AASHTO H20 wheel load plus impact (*161*). Other work has indicated that a bond strength as low as 40 psi (280 kPa) may be adequate for an overlay (*167*). Shear bond strengths measured in the laboratory and using any of the previously mentioned bonding agents are typically in the range of 350 to 500 psi (2.4 to 3.4 MPa). This is not to suggest that bonding procedures can be compromised. It simply indicates that, provided the base concrete is clean and sound and good construction procedures are employed, the bond strength between the base course and a concrete overlay has a considerable safety factor.

Several different types of concrete have been used as concrete overlays including conventional quality portland cement concrete (*155*) and concrete containing steel fibers (*168*). Although steel fibers improve the flexural strength and fatigue resistance of concrete, they do not improve its durability or resistance to chloride penetration. Applications to bridge decks have been few in number (*169*) because the fibers are relatively expensive and the properties of the fibrous concrete cannot be fully utilized, though the fibers may be of benefit on cracked decks. Laboratory studies (*170*) have indicated that the use of shrinkage-compensating concrete may reduce corrosion by reducing cracking of the concrete. The benefits are, however, not likely to be significant. The overwhelming majority of concrete overlays on bridge decks has consisted of low-slump, dense concrete, polymer modified concrete, or internally-sealed concrete.

Low-Slump Concrete Overlays

The use of low-slump concrete as a repair material was originally proposed for pavement repairs (*163*) and was developed for patches and overlays on bridges in the early 1960s at several locations (*164*), but especially in Iowa (*171*) and Kansas (*172*). The widespread use of low-slump overlays in Iowa has led to the process frequently being described as the "Iowa Method." Initially, overlays were no more than ¼ in. (32 mm) thick (*173*), but most agencies now specify a nominal thickness of 2 in. (50 mm) because the cost is not substantially affected by the thickness of the concrete.

The procedure comprises the application of a very low water-cement ratio, dense, portland cement concrete overlay. When used in bridge repairs, the essential steps are as follows: (a) remove the existing deteriorated concrete; (b) scarify the concrete surface to remove ¼ in. (6 mm); (c) sand or water blast the concrete surface and the exposed reinforcing steel; (d) apply a mortar bonding agent to the dry deck surface; (e) place the low-slump concrete overlay; (f) cure the concrete using burlap and water.

In two-stage construction, steps a and b are eliminated; in a delayed second-stage construction, in which the deck has been open to traffic, step a is omitted.

The scarifying is done to remove all contaminants, such as oil drippings and linseed oil, from the surface concrete. The blast cleaning, which is normally done not more than 24 hours before placement of the concrete, removes the rust from the surface of all exposed reinforcing bars and thoroughly cleans the concrete, as illustrated in Figure 11. The blasting operation also removes particles that may have been loosened or cracked by the chipping or scarifying. Sand blasting is generally more satisfactory than water blasting, where environmental considerations permit, because the latter promotes oxidation of the exposed steel unless all excess water is removed from the deck. When used, sand from the blasting operation is blown off the deck with oil-free compressed air.

The mortar bonding agent consists of equal parts of sand and cement mixed to the consistency of stiff cream. The grout is scrubbed into the dry deck surface, as shown in Figure 12, to ensure good penetration. The rate of application of the grout must be carefully controlled to prevent drying prior to applying the overlay. A maximum distance of 5 ft (1.5 m) or a period of 10 minutes ahead of the concrete placement are typical figures used in specifications.

The concrete must be properly air-entrained and proportioned to have a very low water-cement ratio, usually around 0.32. This is achieved by a cement factor of ap-

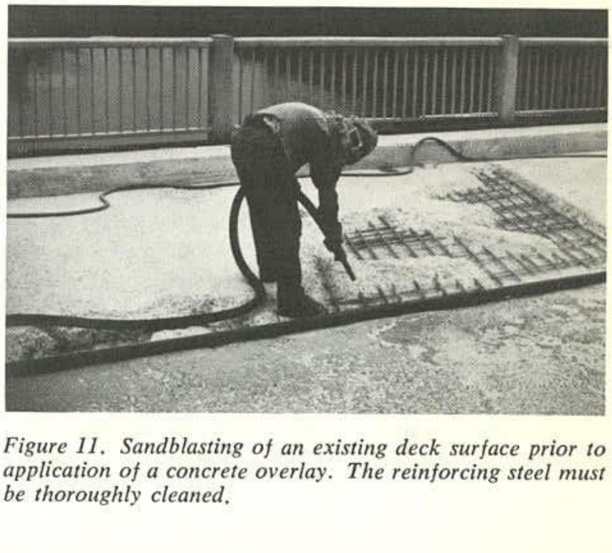


Figure 11. Sandblasting of an existing deck surface prior to application of a concrete overlay. The reinforcing steel must be thoroughly cleaned.

Figure 12. Application of a mortar bonding grout to a dry deck. The rate of application is controlled to prevent premature drying of the grout.

Figure 13. Placement of a low-slump concrete overlay.

proximately 800 lb/yd^3 (480 kg/m^3) and sufficient water to produce a maximum slump of 1 in. (25 mm). A water-reducing admixture, often also acting as a mild retarder, is frequently used. Most highway agencies prohibit the use of transit mixers because it is difficult to mix and discharge concrete of such a low slump, and the rate of application is such that the delay between mixing and placing is unacceptable. Consequently, site mixing using stationary paddle mixers or mobile continuous mixers has been used. Both methods have been found satisfactory, though the latter has a higher production capacity.

The concrete is transported to the point of placing using either hand or power buggies, distributed across the deck by hand, and compacted and screeded to the required elevation by a finishing machine designed for use with low-slump concrete mixtures. Such machines are much stiffer and heavier than conventional finishing machines and are usually equipped with two screeds, at least one of which has considerable vibratory capacity. The machine is supported on heavy adjustable rails, which, when carefully adjusted, will result in an overlay that has a smooth ride. A typical placing operation is illustrated in Figure 13.

As discussed in Chapters One and Three, good consolidation of the concrete is of paramount importance because even concrete that has a low water-cement ratio and is poorly consolidated is rapidly penetrated by chloride ions (*26*). The degree of consolidation is frequently checked with a nuclear density gauge; the requirement is a minimum consolidation of 98 percent of the compacted unit weight of the concrete.

The concrete behind the deck finishing machine is hand finished only where necessary to close up any surface irregularities. It is then transversely grooved using a tined rake to impart skid resistance properties to the overlay. Difficulties in providing the required depth of groove in the stiff concrete have proven to be one of the major deficiencies in low-slump concrete overlays.

Wet burlap is placed on the overlay as soon as possible without damaging the concrete surface, often within 20 minutes of placing. Curing compounds are not used because the thin overlay is susceptible to both plastic and drying shrinkage, especially when placed on a hot deck. Furthermore, water must be supplied to hydrate the cement because of the low initial water content of the concrete, and the wet burlap provides a cooling effect due to evaporation. A minimum curing period of 3 days is required. If the temperature is higher than 85 F (29 C), a number of agencies require the work to be done at night. Then not only are temperatures lower, but usually there is less wind, and consequently evaporation rates are only a fraction of those in daytime conditions.

The edge of the concrete that is not restrained, for example along the centerline of a deck on the first placing operation, has a tendency to slough away under the action of the finishing machine and not be properly compacted. It is common practice to increase the width of the placement by at least 2 in. (50 mm) and saw-cut remove the excess concrete the day after placing.

Low-slump concrete overlays use inexpensive materials but the placing operation is labor-intensive and requires the use of specialized equipment. The success of the overlay is dependent upon good quality control and inspection procedures. Some states have determined that the control necessary is beyond the capabilities of local contractors and inspection staff; therefore, they have dismissed the use of low-slump concrete overlays.

The procedures described, with slight variations, have been used widely in the United States in the past few years, especially for bridge deck repairs. By the fall of 1977, approximately 600 concrete overlays had been constructed on primary and interstate bridges in Iowa. In 1977, costs were in the range \$1.60 to \$4.10/ft^2 (\$17.20 to \$44.10/m^2), with an average of \$2.40/ft^2 (\$25.80/m^2) in new construction (including sand blasting), and in the range \$2.40 to \$9.60/ft^2 (\$25.80 to \$102.20/m^2), with an average of \$3.50/ft^2 (\$37.60/m^2), on repair work. The latter figures include scarifying and sand blasting but exclude removal of deteriorated concrete and ancillary work such as expansion joint modifications and raising the bridge approaches. Nineteen other states have installed low-slump concrete overlays, and many have adopted their use as a routine procedure. Generally good performance has been reported (*158, 173, 174*) dating back to 1965. Local bond failures have been reported, but these have been ascribed to inadequate surface preparation (*158*) or premature drying of the grout (*174*). The overlays are susceptible to cracking, especially on continuous structures, a characteristic common to all rigid overlays.

Polymer-Modified Concrete Overlays

Polymers have been used to modify the properties of concrete for many years (*175*), and the effects of numerous modifiers have been investigated. The main difference between polymer-modified concrete and conventional concrete modified by the use of admixtures, some of which are polymers, is that dosages are much larger than normal admixture dosages. Polymer-modified concrete has been classified as either premix or postmix polymerization.

In the postmix polymerization process, monomer is incorporated in the fresh concrete (the essential difference from polymer-impregnated concrete) and polymerized during or after hardening of the concrete. The reasons for the process's limited success are as follows:

1. Many organic polymers react to form acids in the alkaline environment of the cement paste.
2. The acids inhibit the hydration of the portland cement.
3. Most organic polymers are not soluble in water and consequently produce a nonuniform polymer-modified concrete.

Premix polymerization is the incorporation into the fresh concrete of polymer emulsions, which are polymerized before being added to the mixture. The water of suspension in the emulsion hydrates the cement, and the polymer enters the structure of the concrete and provides supplementary binding due to the adhesive and cohesive properties of the polymer. In general, this results in concrete having good durability and bonding characteristics, properties that are well-suited to use in bridge deck overlays. The structural properties of polymer-modified concrete do, however, vary considerably depending on the type and amount of polymer, the type of aggregate, the cement factor, and the water-cement ratio. Polymer emulsions have been in use for a number of years, and the concrete is more commonly known as latex-modified concrete.

Polymeric latexes are a colloidal dispersion of synthetic rubber particles in water. The particles are stabilized to prevent coagulation, and antifoaming agents are added to prevent excessive air entrapment during mixing. Latex-modified concrete is conventional portland cement concrete with the addition of approximately 15 percent latex solids by weight of the cement. Styrene-butadiene latexes have been most widely used in highway work and are the only type approved for federal-aid work. A latex additive consisting of a blend of saran and styrene-butadiene has also been the subject of several studies. The saran contains vinylidene chloride and vinyl chloride and is not approved by the FHWA, though its use has been permitted on a few experimental installations. More recently, acrylic latexes have been investigated. Field use has been limited but is expected to increase in the future.

The construction procedures for latex-modified concrete parallel those for low-slump concrete. Surface preparation, whether on new or existing decks, is exactly the same as for low-slump concrete. The principal differences in construction procedures are as follows:

1. The deck must be kept wet for at least one hour prior to placing the overlay.
2. A separate bonding agent is not always used.
3. The mixing equipment must have a means of storing and dispensing the latex into the mixture.
4. The latex-modified concrete has a high slump.
5. The concrete is not air-entrained.
6. Conventional deck finishing equipment may be used.
7. A combination of wet and dry curing is required.
8. The thickness of the overlay is usually slightly less than for low-slump concrete.

The latex-modified concrete has been produced almost exclusively in mobile, continuous mixers fitted with an additional storage tank for the latex. The latex modifier should always be maintained between 45 and 85 F (7 and 29 C). This may present serious difficulties, especially during the summer months, and may necessitate night placing operations. Hot weather also causes rapid drying of the concrete, which makes texturing difficult and promotes shrinkage cracks. The shelf life of the latex may preclude storage from one construction season to the next.

The bond coat consists of the mortar fraction of the latex-modified concrete and is usually produced directly from the continuous mixer by cutting off the stone content from the mix. The slurry is broomed into the deck surface.

Mixture proportions differ from low-slump concrete in that the proportion of fine aggregate by weight of the total aggregate is usually much higher. The cement content is approximately 650 lb/yd^3 (390 kg/m^3). The most significant difference, however, is in the consistency; typically, the slump is 5 ± 1 in. (125 ± 25 mm). Although the slump is high, the water-cement ratio is low, usually about 0.35, with a maximum of 0.40. Air entrainment is not required for resistance to freezing and thawing, and the air content, which is mainly entrapped air, is limited to 6½ percent. The entrapment of excessive amounts of air during mixing is one of the most serious problems in the construction of latex-modified concrete overlays and has been reported in a number of field applications. The excessive air content not only reduces the flexural, compressive, and bond strength of the overlay, but also increases its permeability to deicing salts at air contents greater than about 9 percent.

Placing operations are straightforward except on steep grades and crossfalls where the latex-modified concrete tends to flow after being struck off at the required elevation. Finishing machines with conventional vibratory or oscillating screeds may be used (*176*), though a rotating cylindrical drum is preferred. A typical placing operation is illustrated in Figure 14. The ease of placing the highly workable concrete means that manpower requirements are significantly less than for low-slump concrete. Hand finishing is comparable to low-slump concrete overlays, but the concrete accepts a groove much more readily, and there is little difficulty in achieving groove depths of from ⅛ in. to $\frac{3}{16}$ in. (3 to 5 mm). The time of application of the texture is crucial, because if the grooves are made too soon, they collapse, and if the texturing is delayed beyond the time the latex film forms, the film is destroyed. Once destroyed, the film does not form again (*177*), and cracking often results.

Wet burlap must be applied to the concrete as soon as it will be supported without damage. After 24 hours, the burlap is removed and the overlay permitted to air dry for a period of not less than 72 hours. The initial period of wet curing is necessary for the hydration of the portland cement and to prevent the formation of shrinkage cracks; the period of air drying is necessary to permit the latex to dry out and the latex particles to coalesce and form a continuous film. It is film formation within the concrete that gives the concrete good bond, flexural strength, and low permeability. The film forming properties of the latex are temperature-sensitive and develop very slowly at temperatures less than 55 F (13 C). Curing periods at lower temperatures may need to be significantly extended, and placing at temperatures less than 45 F (7 C) is not recommended.

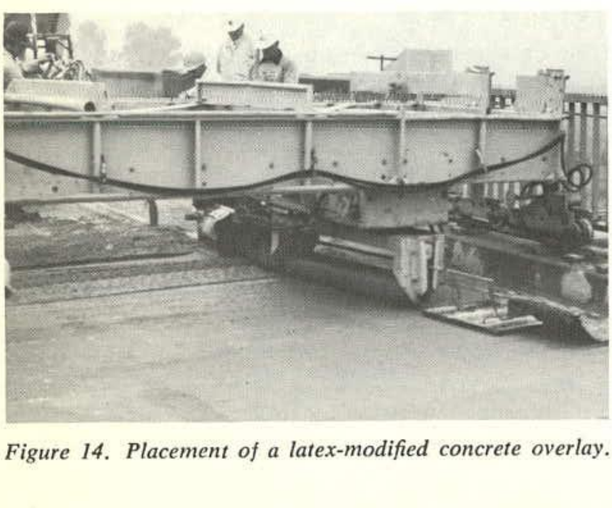


Figure 14. Placement of a latex-modified concrete overlay.

High material costs and the superior performance of latex-modified concrete in chloride penetration tests have led to latex-modified concrete overlays being thinner than most low-slump concrete overlays. Latex-modified overlays ¾ in. (19 mm) thick were used a few years ago (*155, 174*), but thicknesses of 1¼ or 1½ in. (32 to 38 mm) are now more common. Many agencies permit the use of low-slump or latex-modified concrete as an alternate at the contractor's option. The only difference between the two alternates is that the latex-modified option is often ½ in. (13 mm) thinner than the low-slump option. This method of bidding also circumvents the difficulty of specifying a proprietary product, which arose because initially only one brand of latex was approved for use in federal-aid work and contractors were licensed by the manufacturer of the latex. This situation no longer exists, as other latexes have been given FHWA approval.

The use of latex-modified concrete has been more widespread than low-slump concrete, and a number of states prefer the system because of its ease of application. In some states, the system of licensing contractors led to good quality control practices, which simplified the problem of the highway agency in obtaining high-quality workmanship, though this was not always the case. Although latex-modified and low-slump concrete are designed to fulfill the same purpose, the essential difference between the two systems is that low-slump concrete uses inexpensive materials but is difficult to place and requires sophisticated finishing equipment. Conversely, latex-modified concrete utilizes expensive materials but requires less manpower and is placed by conventional equipment.

By the end of 1977, 24 states had installed latex-modified concrete overlays on either new or existing decks. Latex-modified concrete overlays were first used in 1957 (*166*), though the majority of installations are less than 5 years old. Performance has generally been satisfactory, though extensive cracking and some debonding have been reported (*178*), especially in overlays ¾ in. (19 mm) thick (*155, 174*). Numerous cracks, which develop shortly after placing, have also been observed (*165, 176*). Most of these cracks, which do not extend through the overlay and are not progressive, have been attributed to rapid, initial shrinkage. The cracking is most prevalent in latex-modified mortar overlays because the drying shrinkage of the mortar is about double that of latex-modified concrete (*179*).

The relative costs of latex and low-slump overlays depend upon the historical development of the market in each state. The result is that where there is a history of low-slump concrete, this is the least expensive, while in areas where strong licensees were established, latex-modified concrete may be more competitive. By the end of 1977, the overall trend was for latex-modified concrete to be slightly more expensive, though this situation could change rapidly with the introduction of more latexes to the marketplace.

Internally Sealed Concrete Overlays

The process of internally sealing concrete consists of incorporating fusible polymeric particles in a concrete mix followed by fusion of the additive after the concrete is cured. Fusion is accomplished through the application of heat and causes the additive to flow into the micropore structure of the concrete, effectively sealing the concrete against the ingress of moisture and chemicals. Although internally sealed concrete is a polymer modified concrete, it is not readily classified as either a premix or postmix polymerization procedure. Polymeric particles are added to the concrete at the time of mixing, a characteristic of the premix process, but postmix fusion is also required.

Laboratory work identified the most promising additive to be a 25/75 blend of montan and paraffin wax in the form of 180 to 850 μm (No. 80 to No. 20 sieve size) spherical particles added to the concrete at the rate of 3 percent by weight, which corresponds to approximately 8 percent by volume (*180*). The wax in the concrete was found to be uniformly distributed after the concrete was heated to 185 F (85 C) (*181*).

The wax particles are prepared by blending the waxes in a reaction kettle and spraying the molten wax through fine nozzles into the top of a 35-ft (11-m) high chamber. The particles cool and solidify as they fall and are screened prior to storage. Particles failing to meet the required mesh size are recycled. Beads from the initial production run in 1975 were found to cause cracking in nonair-entrained concrete because of the solid volume expansion of the wax during the heating cycle. This problem was overcome by injecting compressed air into the wax during manufacture to produce beads with an average porosity of 8 percent (*182*).

Both air-entrained and nonair-entrained, internally sealed concrete were found to be almost impermeable to deicing salts and to have excellent frost durability (*180, 182*). The compressive strength of nonair-entrained, internally sealed concrete was found to be similar to that of conventional concrete with 6 percent air. Internally sealed, air-entrained concrete had a strength equal to conventional concrete with 11 percent air.

In view of the good durability of the nonair-entrained concrete, most installations of internally sealed concrete have been without air entrainment. However, where loss of strength is not a factor, usually where an overlay is used, air entrainment offers good insurance against scaling in areas where improper sealing occurs and also against cracking induced by beads with insufficient porosity (*158*).

Mixture proportioning follows standard procedures with the additional step of replacing 2.1 cu ft of sand per cu yd (0.078 by volume) or a combination of sand and stone by the wax beads. Trial mixtures are used to establish the water content to produce a slump of between 2½ and 4 in. (60 and 100 mm). The water-cement ratio should not exceed 0.55 (*183*). Conventional admixtures may be used. The beads are shipped in moisture-proof containers and must be stored at temperatures less than 120 F (49 C) at all times.

Several procedures have been used for adding beads to the mixer, but the recommended sequence is to ribbon batch the dry ingredients, add the wax beads, and finally add the water. About 10 minutes of mixing, or 85 revolutions, have proven sufficient for thorough mixing and uniform distribution of the beads.

Concrete containing wax beads is similar in appearance, handling, and placing characteristics to conventional bridge deck concrete. Internally sealed concrete is most appropriately used in a 2 or 3-in. (50 to 80-mm) thick overlay on a conventional concrete deck, although it has been placed full depth on some experimental projects. In such cases, no attempt has been made to heat treat the entire deck, and neither is such a procedure necessary. Two-course construction uses fewer beads and is potentially more economical. Two different methods have been used: applying a bonding agent and placing the overlay on the hardened deck concrete, or placing the overlay and the deck concrete simultaneously, with the overlay lagging the base by about 30 minutes. In both cases, conventional construction procedures and equipment are used.

Wet curing must be used because membrane curing compounds interfere with the removal of moisture prior to and during the heat treatment (*183*). A minimum of seven days moist curing is required.

Heat treatment is the final step in the construction process and can be done any time after the design strength of the concrete is achieved, but preferably at least 21 days after placement. For sealing, the deck must be heated to 185 F (85 C) at the desired depth of sealing, usually 2 in. (50 mm).

Heating may be accomplished by either a single pass infrared heater or an electric blanket heating system. If the single pass heater is used, the concrete must be dry before heat treatment begins. The rapid rise in surface temperature quickly seals the surface of the concrete, and blistering and cracking may occur if moisture is trapped within the concrete. A one- or two-man-operated, 6- by 15-ft (1.8 $\times$ 4.6 m) unit will heat treat approximately 45 ft^2 (4.2 m^2) of bridge deck per hour.

The electric blankets provide a much slower heat rise, which minimizes thermal gradients in the deck. This method can be used regardless of the moisture condition in the concrete because there is ample time for excess moisture to escape during the heating cycle, which takes from 8 to 12 hours, depending upon such factors as concrete moisture content, ambient temperature and wind speed. The progress of the heating cycle is monitored by thermocouples installed in the deck. The blankets that have been used on projects to date are prototype, folding, stainless steel pads developed by the Federal Highway Administration. The pads cover a total of about 1,200 ft^2 (110 m^2) and require power up to 130 W/ft^2 (1400 W/m^2). Power is supplied by a 550 V, three-phase generator of not less than 150 kW capacity. The pads are connected to the generator through a distribution panel and are covered with insulation to minimize heat losses.

One of the difficulties that has been encountered is the development of hairline cracks adjacent to curbs or barrier walls during the heating operation because of the restraint imposed by these members (*158, 183*). In some cases, vertical cracks in the parapet or barrier walls have been observed. Where feasible, it is recommended that heat treatment precede the placement of curbs and barrier walls. Where this is not possible, heating blankets should be used to partially heat the curbs and walls to reduce thermal stresses.

The first internally sealed concrete overlay was placed in Oklahoma in 1976. Up to the end of 1978, a total of 14 internally sealed concrete overlays had been constructed. All the installations were on new decks and considered experimental. The overlays appear to be performing well, and no problems, other than the initial cracking and the cracking that developed on some continuous structures after opening to traffic, have been identified. Measurements have shown that the skid resistance of the deck is not significantly changed after melting the wax.

Laboratory testing and field demonstration projects have shown internally sealed concrete to be technically feasible, but its economic feasibility and practicality remain doubtful using present procedures. Research is continuing in an attempt to improve the speed and efficiency of the heat treatment process and also to identify different additives, which may eliminate the heat treatment entirely. The degree of success achieved in this ongoing research will largely determine the number of future applications of internally sealed concrete.

WATERPROOFING MEMBRANES

Membranes have been used extensively for many years in parts of North America, especially the New England States, and Europe to prevent bridge deck deterioration. Their use increased dramatically with the advent of the policy in 1972 requiring that a deck protective system be applied to all federal-aid structures. Some of the alternative deck protective systems, such as epoxy-coated bars, were in the development stage, and others, such as concrete overlays, required expertise by the contractor and a level of inspection that was not readily available. Consequently, the installation of a membrane was one of the most convenient methods of complying with the FHWA requirements.

The rapid expansion of the market for membranes led to the introduction of numerous products, and this, in turn, caused difficulties for highway agencies because performance criteria were not identified. Products of different materials and quality have been used and, as may be expected from such diverse origins, field experience has been highly variable. No other deck protective system has polarized the opinion of highway agencies as strongly as membranes.

The requirements for the ideal waterproofing system have been defined as follows (*150, 184*): (a) easy to install; (b) good bond to substrate and to the wearing course; (c) compatible with all the system components including substrate, protective layer, wearing course, adhesives, and prime coat; (d) maintain waterproofing qualities under service conditions, notably temperature extremes, vehicular loading, crack bridging, and aging; and (e) inexpensive.

The wide variety of products that have been developed to satisfy these requirements makes generalizations of product characteristics and performance difficult. Any classification system is arbitrary, though one of the most useful is the division between the preformed sheet systems and the liquid or applied-in-place materials (*150*). The relative merits of these two groups of products are presented in Table 2.

The overwhelming majority of waterproofing systems cannot be used as the riding surface of the deck and require an asphaltic concrete wearing course. Many products also require the use of an intermediate protective layer between the membrane and the wearing course to prevent damage during installation of the hot mix and to resist puncture of the membrane by aggregate particles under service conditions. The most common forms of protective layers are roofing felt and asphalt-impregnated protection boards, typically ⅛ in. (3- mm) thick.

The requirement for a separate wearing course has an effect on the performance of the bridge deck and may dictate the economic life of the membrane. When the wearing course requires replacement, rarely is it possible to overlay it with an additional lift or to remove the existing wearing course with the membrane intact, so the membrane must also be replaced. The advantages and disadvantages associated with the use of the asphalt wearing course are summarized in Table 3.

The economic life of the wearing course is determined by its thickness and the service conditions at the bridge site, especially traffic volumes. Wear of asphaltic concrete has been particularly rapid where studded tires are permitted, and some states limit the use of asphalt wearing courses to bridges on secondary highways because of their short life on primary routes.

The number of cycles of freezing and thawing and the distribution of temperature within a bridge deck have been measured on a structure with a membrane and a 2-in. (50-mm) thick wearing course (*185*). It was found that the bridge deck had approximately 50 percent more frost cycles than the abutting roadway and that a temperature differential of 5 to 10 F (3 to 6 C) between the bottom of the bituminous concrete and the top of the deck slab was not uncommon. The abrupt change in temperature at the membrane, together with the difference in the coefficients of thermal expansion between portland cement and bituminous concretes, makes it difficult to achieve good bond at the interface.

Several studies have been undertaken to evaluate membranes in both the laboratory (*66, 68, 185, 186, 187*) and the field (*53, 54, 57, 67, 188, 189*). Phase I of NCHRP Project 12-11, "Waterproof Membranes for Protection of Concrete Bridge Decks," investigated the effectiveness of 147 waterproofing systems available at the time the work commenced in 1970 (*66*). The study also included the field examination of 25 systems previously installed and the development of laboratory characterization and performance tests. Following a series of eliminations, five systems, all of which included preformed sheeting, were

TABLE 2

CHARACTERISTICS OF MEMBRANES

Applied-in-Place Materials	Preformed Systems
Difficult to assure the quality of two-component materials and products which are hot applied.	Quality of material controlled under factory conditions.
Careful field inspection required to control thickness of membrane and detect presence of pinholes.	Thickness and integrity controlled at the factory.
Usually applied in one course by spray or squeegee. No laps required.	Labor-intensive installation, especially if not self-adhesive. Laps necessary.
Application independent of deck geometry.	Difficult to install on curved or rough decks.
Bonding not usually a problem if substrate properly prepared. Systems usually self-adhesive.	Cured sheets may be difficult to bond to substrate, protection layer and at laps.
Installation not affected by deck details.	Vulnerable to quality of workmanship at critical locations such as curbs, expansion joints and deck drains.
Blisters and blowholes easily repaired in self-sealing materials.	Blisters must be repaired by puncturing and patching.
Tend to be less expensive.	Tend to be more expensive.

selected as the most promising for detailed evaluation under service conditions. Specifications for the materials and methods of construction were prepared.

Other laboratory investigations have also indicated the superiority of preformed membranes (*186, 187*), which may be expected to perform better than applied-in-place materials in simple screening tests. Several states have developed prequalification tests for membranes, but there has been little uniformity in either test methods or acceptance criteria. The most common tests have been for permeability, crack bridging capability, bond to concrete, and durability in the service environment. The last includes resistance to damage by heat and impact (to simulate placement of the wearing course) and resistance to slow penetration by aggregate in the wearing course. Permeability has generally been adopted as the most important criterion, though it has been found that liquid-applied membranes offer substantial protection against chloride intrusion even when pinholes and bubbles occur in the coating (*68*). Unfortunately, laboratory evaluations cannot simulate the effects of inadequate surface preparation, poor workmanship, and adverse weather conditions. Neither can they duplicate the effects of vehicular loading, which creates high transient pressures if water exists in confined voids at the interface with either the top or bottom of the membrane (*190*).

NEEP Project No. 12 was initiated in 1971 to encourage the evaluation of a wide range of membranes on an experimental basis. The five systems identified as the most promising in the NCHRP study were included in the NEEP project. The cost of these products was found to be high, and installation difficulties were experienced in some instances. Generalization of field performance is more difficult than for product characteristics because of differences not only between the products themselves but also differences in the quality of workmanship, weather conditions at the time of installation, design details, and the service environment. Experiences have covered the whole spectrum from satisfactory performance in some systems (*53*) to dramatic failures where the membrane has had to be removed (*54, 190*), sometimes before the deck was open to traffic. Because of this tendency to slip, membranes should not be used on grades greater than 4 percent or in areas subject to rapid acceleration, deceleration, or turning

TABLE 3

RELATIVE MERITS OF A BITUMINOUS WEARING COURSE

Advantages	Disadvantages
Provides a smooth riding surface.	Adds dead load and is not a structural component.
Reduces stress concentrations on the concrete slab from vehicular loading.	Deterioration of the concrete cannot be detected until serious distress has occurred.
	Must be replaced periodically, typically 5 to 15 years.
	Bituminous concrete is both permeable and porous, trapping brine on the surface of the membrane.
	Asphalt absorbs solar radiation more rapidly than concrete, thereby increasing the number of frost cycles in the winter months.
	Asphalt is difficult to compact at curbs, joints and deck drains and is therefore most porous at these critical locations of the waterproofing.
	If leakage occurs through the membrane, water is trapped on the deck and deterioration is likely to be accelerated.
	Bonding of the membrane and wearing course to the deck is difficult because of the different coefficients of expansion of asphalt and concrete.
	Cracks in the asphalt may be reflected through the membrane.
	Additional cost factor.

movements (*191*). Where membranes have been applied to existing decks, surveys have indicated a continuation of corrosion activity if all the chloride contaminated concrete is not removed (*57, 67, 192*). The field performance of membranes has been highly variable, and unsatisfactory examples have been found in all the investigations.

Several studies have indicated that leakage is most prevalent at curbs and expansion joints (*53, 188, 189, 190*). These areas are particularly difficult to seal effectively with preformed membranes. There are also several examples of lower than average resistance readings in wheelpath areas (*66*) and a significant decline in the effectiveness of membranes during the first few years after installation (*189*).

Blistering has been identified as the greatest single problem in satisfactorily waterproofing a bridge deck (*193*) and affects both preformed sheets and applied-in-place materials. Blisters are caused by the expansion of air in the concrete after application of the membrane, a phenomenon sometimes known as outgassing. Water or water vapor is not a necessary requirement for blister formation but can be a strong contributing factor (*194*). The vaporization of even a small quantity of water contained in the concrete produces a substantial volume of water vapor and exerts a significant pressure on the membrane. Blisters may take several forms ranging from numerous pinholes to single blisters that may cover a square foot (0.1 m^2) or more.

A distinction is sometimes drawn between blisters and blowholes. Blowholes occur at the time of installation of applied-in-place membranes; blisters may develop in any kind of membrane several hours after installation. Blowholes may be formed by the rapid expansion of vapors in the concrete during the application of hot-applied products or during the curing of cold-applied products because of rising ambient temperatures, decreasing atmospheric pressure, or the increase in deck temperature caused by increased absorption of solar radiation upon application of a black or dark-colored membrane. Consequently, the possibility of blisters occurring in any situation is determined by the porosity and moisture content of the concrete (*187*) and the atmospheric conditions.

Membranes can be placed without blowing if atmospheric and substrate conditions are favorable. One solution to the blistering problem is to ensure that the deck temperature is higher than ambient temperature at the time the membrane is applied and, where applicable, during the curing period of either the membrane or its adhesives. One proposed method (*194*) of satisfying this requirement involves the application of a black prime coat to the deck, which is then allowed to heat the deck by solar radiation. The membrane is then applied after the maximum deck temperature has been reached. When the membrane is cured, the bond to the deck should be sufficient to resist vapor pressures from within the concrete. Sealing the deck prior to applying the membrane to prevent blowhole formation is possible but not practical (*194*).

Blisters, as distinct from blowholes, are more common with prefabricated membranes and usually occur in areas of poor adhesion. They may result from air pockets trapped beneath the membrane or from water vapor pressure developing at the interface between the deck and the membrane as a result of moisture migration through the deck. Blisters may occasionally be caused by the vaporization of solvents used in prime coats and adhesives. To obviate this danger, systems with critical air curing times for any component should not be used (*191*). Blisters may also result from solar radiation on the membrane, in which case the risk of blister formation can be reduced by minimizing the delay between membrane and asphalt placement. The rapid expansion of vapor during placement of the hot mix can result in blister formation which, in turn, may cause cracking of the asphalt. Air pockets trapped beneath a protection layer will also have the same effect. Except for one unusual case (*193*), a 2-in. (50-mm) thick asphalt wearing course has been found sufficient to prevent blisters occurring after the hot-mix has been placed, though a 3-in. (75-mm) thickness is preferable. Three-quarter in. (20 mm) of sand asphalt has been found to have insufficient dead load to prevent blistering (*193*).

An alternative method of preventing blisters is to allow the vapor pressure to disperse through a venting layer beneath the membrane. Venting layers, which may take the form of a perforated sheet of bituminous felt; an open-weave glass or polypropylene fiber; dimpled, coated aluminum or copper sheeting; or similar material, depend upon controlled debonding of the membrane. In the United Kingdom, the practice is to seal the edges of the venting layer (*193*) to prevent the ingress of water, and the venting layer acts to disperse local pressures. In Germany and Switzerland, venting pipes through the deck are frequently provided (*195*). The disadvantages in the use of venting layers are increased cost, that partial debonding may lead to slipping, and that water may spread from a puncture in the membrane to any part of the deck. Venting layers have not received widespread use in the United States, though a vented membrane has been developed for experimental evaluation (*196*). Tennessee first used a sandwich system of waterproofing in 1975 to prevent blistering, and the system appears to be working satisfactorily. A 1-in. thick (25-mm) asphalt base course is placed on the deck, followed by a sheet membrane and a 1 or 2-in. thick (25- or 50-mm) asphalt wearing course. The base course not only acts as a venting layer to dissipate vapor pressures but also acts as a leveling course that prevents puncturing of the membrane on rough decks.

The performance of membranes can be improved considerably through improvements in design details and workmanship. The surface of the deck must have a smooth texture and be free from serious irregularities. A surface tolerance that does not permit a departure of more than ⅜ in. (10 mm) when tested with a 10-ft (3-m) straightedge and a maximum texture depth of ⅛ in. (3 mm) have been suggested (*190*). Depressions greater than ⅜ in. (10 mm) should be patched and, on extremely rough decks, such as may occur on a deck which has been in service, a sand-asphalt leveling course may be applied.

The deck surface should be sand or waterblasted to remove surface laitance, curing membranes, and other contaminants. The deck should be dry and free from dust at the time of membrane application to improve adhesion and lessen the risk of blister formation. Priming the deck not only aids in bonding but will also bind any dust particles on the deck. A protection board or roofing felt [typically a 65-lb (3.2 kg/m^2) felt] may be used to prevent damage to the membrane during starting, turning, and stopping the paving equipment and to prevent puncture of the membrane by aggregate particles in the wearing course under the action of traffic. The protection board must have a low absorption to prevent separation of the board under frost conditions and must not contain solvents that may vaporize and cause the edges of the board to curl prior to applying the wearing course. Poor bond between the protection board and the wearing course has resulted in rapid failure of wearing courses less than 2 in. (50 mm) thick (*174*). Priming the protective layer to improve bond is desirable. Although many systems are installed with a protective layer, one survey has indicated that a protective layer is not required to extend the service life of all membranes (*53*).

Leakage at curbs and expansion joints can be prevented by sealing the edges of the membrane at these locations, either by placing the membrane up the face of the curb or joint or by forming a sealed joint at these locations. It is also important to recognize that the asphalt wearing course is permeable and provision must be made for drainage from the surface of the membrane. This is often accomplished through vertical slots in the deck drains.

Although the results of field surveys to evaluate the performance of membranes have been conflicting, all have revealed deficiencies in at least some installations. There is some doubt as to the long-term performance of membranes, especially in view of the limited life of the asphalt wearing course. Despite serious limitations because of problems in construction and serviceability, some membranes have been found to increase the service life of bridge decks and to be cost effective (*53, 190*). Consequently, the use of membranes is permitted on federal-aid structures as a less preferable alternate to epoxy-coated bars and low-slump and latex-modified concrete overlays (*83*).

CATHODIC PROTECTION

The use of cathodic protection to prevent steel corrosion in pipelines has been well established since the mid-1930s. Cathodic protection has also been installed on large diameter, prestressed concrete water pipelines in several countries. It has also been used in such specialized applications as the protection of steel reinforcement linear plate in nuclear reactor containment vessels. The application of cathodic protection to bridge decks is relatively new and presents greater difficulties than pipeline protection because of the lack of a suitable conductive environment for the anode (*197*), and the necessity to prevent overprotection.

The theory of cathodic protection of steel in concrete is to apply sufficient direct current in the proper direction so that corroding anodes on the steel are prevented from discharging ions. Thus, the current discharging anodes become current receiving cathodes; hence the term cathodic protection.

To accomplish the requirement that all the existing anodes on the steel be made to receive current, the half-cell potential of all the steel must be made more negative than the most negative of the anodes. The most anodic half-cell potential of corroded steel in corrosion-cracked concrete measured by Stratfull (*50*) was —0.67 V (relative to the copper/copper sulfate half-cell, CSE). For steel pipelines, the empirical criterion for cathodic protection is that the steel must be made more negative than —0.85 volt CSE. Stratfull has recommended that for the satisfactory cathodic protection of steel in concrete, the potential of the steel should be no less than —0.85 volt CSE and no more than —1.10 volt CSE to avoid overprotection (*39*). Laboratory studies undertaken as part of NCHRP Project 12-13, "Cathodic Protection for Reinforced Concrete Bridge Decks" (*198*), demonstrated that corrosion was stopped at a steel polarized potential of —0.77 volt CSE and that hydrogen gas bubbles began to form at the steel surface at —1.17 volt CSE in a high pH solution. These values confirm the absolute range of potentials required for effective cathodic protection without any allowance for a factor of safety. It has also been reported that cathodic protection should be effective for steel in calcium hydroxide solutions containing chlorides at potentials of about —0.71 volt CSE (*199*), and even —0.50 volt CSE has been suggested as being sufficient (*200*).

There are two methods for applying cathodic protection: galvanic anodes and impressed current. In the galvanic anode system, a metal electrode that is higher in the electromotive series than the metal to be protected is connected to it. The driving potential for the current is the potential difference between the metal and the anode which is sacrificed. Zinc and magnesium are the most suitable anodes for the protection of steel in concrete.

The limiting factor in the use of sacrificial anodes is their low driving voltage, which means that, because concrete has a relatively high resistivity, numerous anodes are required. Overprotection is, however, not possible, and the galvanic system is maintenance-free until the anodes are consumed. Laboratory studies (*198*) found the sacrificial anode approach to bridge deck protection was worthy of further study. Requirements for spacing, surrounding materials, and installation methods were suggested on the basis of analog and prototype studies. A field study program was initiated under NCHRP Project 12-13A, "Field Evaluation of Galvanic Protection for Reinforced Concrete Bridge Decks." This program includes the evaluation of the effectiveness of a zinc anode galvanic protective system on an actively corroding bridge deck by placing different configurations of zinc anodes in the deck over the top layer of reinforcement. By the end of 1977, the system had been installed on a bridge in Illinois and periodic monitoring begun.

The impressed current or external power system is illustrated in Figure 15. Because the current flow does not depend upon the relative potentials of the anode and the metal being protected, the anodes may be selected for their durability and conductivity. High-silicon cast iron

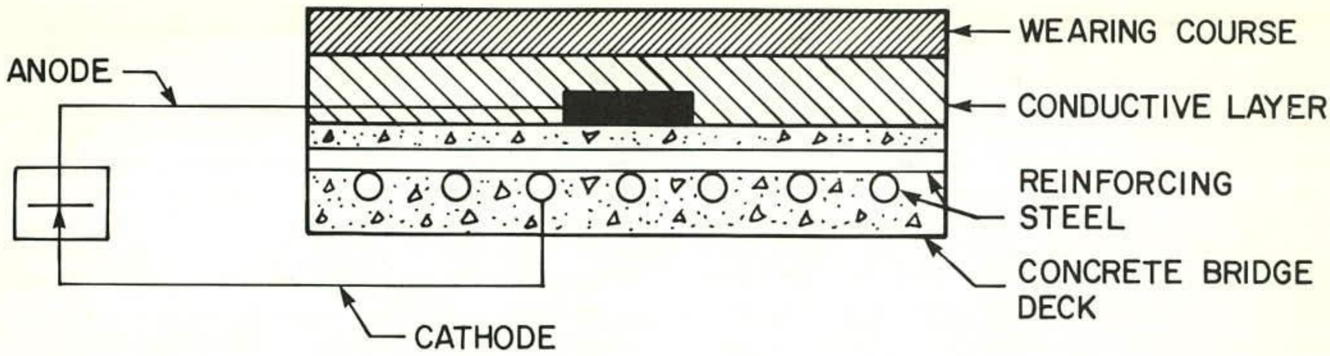


Figure 15. Impressed current cathodic protection circuit.

or graphite is most commonly used. The impressed current may be provided from batteries or by a DC rectifier operating on AC line voltage. The voltage and current of each anode can be individually controlled to maintain the required potential of the reinforcing steel through the use of a simple instrumentation panel. The location of a rectifier and control panel beneath a bridge deck is shown in Figure 16. There is a danger of overprotection with the impressed current system, and periodic monitoring is required to ensure that the polarized potential of the steel is maintained within the prescribed limits. The steel reinforcement in the bridge deck must also be electrically continuous. Because this has been found to be the case in all installations to date and, because of tying at intersection points required by construction specifications, this may not be a problem in practice.

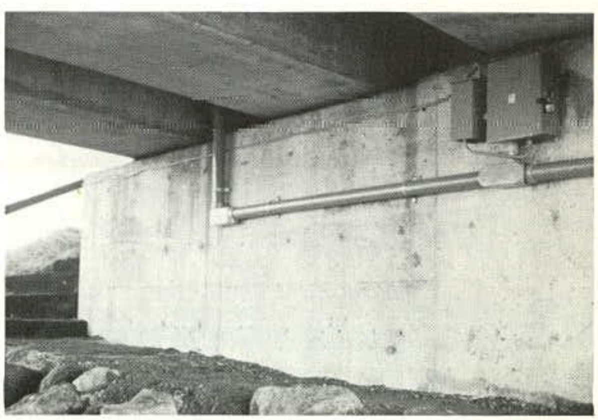
Although the passage of a low electrical current through concrete has no detrimental effect on the concrete itself, hydrogen released at the cathodic steel surface may result in loss of bond between the concrete and the steel. This effect was first demonstrated by National Bureau of Standards tests published in 1913 (*29*) in which a definite softening of the concrete near the cathode was observed. Subsequent reports have confirmed this effect but have differed in determining the threshold values of applied voltage or current at which it becomes significant. More recently (*198*), extensive bond studies showed that the application of a cathodic protection current to rebars can result in a decrease in bond strength. This reduction would be slight under the anticipated conditions of cathodic protection applied to a bridge deck and is not considered to be a problem in practice, providing that overprotection is avoided by limiting the maximum polarized potential of the steel to −1.10 volt CSE.

The application of cathodic protection to bridge decks has been pioneered by Stratfull at the California Department of Transportation. Stratfull's first attempt at cathodic protection of a bridge structure was in 1958 (*201*) when the system was applied to concrete beams in one of the San Francisco Bay structures. The system appeared to be functioning effectively when the structure was replaced one year later.

More recently, Stratfull has shown (*39*) that to protect the top mat of reinforcing steel in a bridge deck with a practical number of anodes, each anode must be placed in an electrically conductive overlay on the concrete surface. This provides essentially equal resistance between all the bars and the power source. Coke has been in use for many years as an anode backfill material in the pipeline industry, and a mixture of coke breeze with an asphalt binder was developed for use as the conductive layer on a bridge deck. The first time this concept was applied on a bridge was an experimental installation in 1973.

Prior to installing the cathodic protection, delaminated areas were repaired by injection. Thirty-six iron-alloy anodes were fastened to the deck using an epoxy adhesive. The purpose of the adhesive was not only to secure the anodes during paving but also to prevent current flowing directly from the anode to the steel, thereby creating localized areas of high potential. A coke-breeze mixture consisting of 85 percent coke aggregate of ⅜ in. (10 mm) maximum size and 15 percent 85-100 penetration asphalt was applied to the deck. This was followed by a nominal 2-in. (50-mm) thick conventional wearing course for a total thickness of approximately 5 in. (130 mm).

It was found that the deck area of 3,300 ft^2 (310 m^3) could be adequately protected using 7 anodes with an initial driving voltage of 1.65 volt and a total current of 1 amp. Subsequent measurements indicated that the minimum polarized potential criterion of −0.85 volt CSE was not obtained, though corrosion detection devices indicated corrosion stopped at power levels of about 2 watts (*202*).



The corrosion detection devices were steel strips embedded in concrete containing 10 percent calcium chloride by weight. The cathodic protection was deemed to be effective when corrosion of steel strips ceased. The steel strips were also used to demonstrate that polarization of the steel is maintained for several days after the power is switched off. This confirms that the system is not susceptible to sudden or undiscovered loss of power if, for instance, the control panel or rectifier is damaged by lightning or vandalism.

Based on the experience gained in the first installation, cathodic protection was applied to three additional structures in California in 1974, with four more added in 1975 (*202*).

Installations elsewhere (*203, 204*) have been patterned on the California prototype. Extensive tests of the impressed current system using a soffit anode arrangement proved this concept to be uneconomical because of the close anode spacing required for adequate current distribution (*197*).

Up to the end of 1977, there have been six installations of cathodic protection in Ontario, where the system has been the subject of continuing development. Two of the more significant advances have been the installation of all electrical hardware such that the wearing course and conductive layer can be replaced without damage to the electrical circuitry (*205*) and the development of a conductive mixture with a stability comparable to conventional bituminous concretes (*206*). The electrical hardware is protected by recessing all the anodes and voltage and corrosometer probes in the deck surface as shown in Figure 17. The wires for the hardware are also recessed and carried in saw cuts to the curb, where all the wires are cast in a concrete strip along the base of the curb face.

The disadvantages of the mixture of asphalt and coke breeze, which had a high void content and a low stability, were the danger of rutting under heavy traffic, difficulty of compaction at temperatures over 160 F (71 C) (*204*), and that brine accumulating in the pores may cause stripping of the asphalt and accelerate deterioration of the concrete. It was found that by including fine and coarse aggregates as well as coke in the mixture, a conductive mixture could be produced with other characteristics comparable to conventional mixtures. Typical mixture proportions were stone/sand/coke breeze in the ratios 40/15/45, with an asphalt content of 15.5 percent by weight of the total aggregate. The resistivity of such a mixture was found to be 3.0 ohm-cm compared with 1.4 ohm-cm for a mixture of 80 percent coke breeze and 20 percent asphalt.

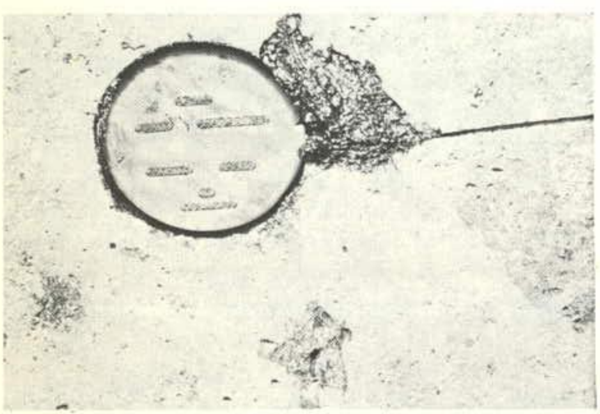


Figure 17. Anode recessed in concrete deck surface to permit future replacement of conductive layer and wearing course.

The use of cathodic protection on bridge decks has shown that the conductive mixture is an effective method of distributing current across the deck and that fewer anodes are needed than were included in prototype installations. Typically, anode spacings of 50 ft (15 m) are sufficient to provide uniform current densities.

One of the difficulties that has developed is in control of the rectifiers. A continuous feedback system was originally proposed (*39*) whereby the potential on the deck would be sensed by half-cells, which would control the rectifier to maintain the potential within the range —0.85 volt CSE to —1.10 volt CSE. It has been found that half-cells are unreliable in the bridge deck environment (*207*) and do not measure the polarized potential of the steel, which is the criterion for cathodic protection, but simply record the voltage on the deck surface from the rectifier. The polarized potential of the steel is determined not only by the rectifier output but also by the conductivity of the concrete and the depth of cover. The conductive mixture also acts as a half-cell, and its voltage must be taken into account when measuring polarized potentials on the deck (*207*). Rectifiers with current control are an alternative to voltage control, but suffer the serious limitation that, because the conductivity of the concrete changes seasonally, the upper limit of polarized potential may be exceeded if periodic adjustments are not made. Voltage probes in the deck do give reliable readings and, because of the uniform distribution of voltage that has been observed in existing installations, it may well be that constant voltage rectifiers can be used and the separate controls for each anode eliminated.

In 1977, the first installation in California in 1973 and the first in Ontario in 1974 were exposed. In both cases it was found that there had been no further corrosion of the reinforcing steel in cathodically protected areas except where delaminations had been repaired by epoxy injection. The layer of epoxy had electrically insulated the underlying concrete, and the experiences illustrate the need for either an electrically conductive epoxy or the necessity to remove and replace all unsound concrete prior to applying cathodic protection. In California, where part of the deck was waterproofed with a coal tar emulsion and fabric system, corrosion continued beneath the membrane, resulting in significant areas of new deterioration. In the Ontario installation, where one side of the deck was cathodically protected and the other paved with a dense, asphaltic concrete, corrosion spalling developed on the unprotected side of the deck. In both Ontario and California, there was no deterioration of the concrete from brine trapped on the deck by the coke breeze, but the concrete was known to have a good air void system. Neither was there any evidence of rutting or stripping in the coke breeze mixture.

One of the factors that has restricted the number of installations of cathodic protection is that it requires expertise for design, construction, inspection, and monitoring that does not exist in many highway agencies. Consultants are now available to undertake these tasks. The installation of the electrical hardware must be checked very carefully prior to applying the conductive layer to ensure that it does not short circuit to the reinforcing steel. The conductive mixture must also be applied so that it is insulated from all the steel in the deck including deck drains, expansion joints, and exposed reinforcing bars. This is usually done by covering exposed bars with epoxy and surrounding deck drains and expansion joint assemblies with conventional, nonconductive bituminous concrete.

The source of power for structures in locations where electrical power is not available is an added complication. Solar cells are considered feasible in such situations, and they have been used in a demonstration project (*208*). Vandalism is a potential problem, however, not only to the solar cells but to the distribution panel and rectifier in any installation.

The economic life of the conductive mixture and its long-term effects on the concrete deck slab are uncertain. Because the purpose of the conductive layer is to conduct the current uniformly across the deck and impress it on the reinforcing steel, a membrane, which is a dielectric material, cannot be used to protect the concrete. A necessary precaution is to ensure that the concrete is sound and properly air entrained.

Although some states have installed cathodic protection on new structures in demonstration projects, the need for periodic monitoring does not satisfy the requirement for a completely maintenance-free deck. It is, however, the only method, short of removal of all chloride contaminated concrete, that is sure to stop active corrosion in a bridge deck. As such, it has application as a method of deck repair. Its use has been largely experimental, and design, materials, and construction procedures are the subject of continued improvement. Cathodic protection is permitted on federal-aid work where it is economically viable (*83*). The cost of cathodic protection is highly variable, depending chiefly on the local availability of the coke breeze and the willingness of local batch plants to interrupt normal operations to produce the conductive mixture. Costs can be comparable to alternative repair methods in some parts of the country, and figures in the range \$2.10 (*204*) to \$4.40 (*205*) per sq ft (\$22.60 to \$47.30/m^2) have been reported.

CHAPTER FIVE

TECHNIQUES FOR REPAIR

INTRODUCTION

Repair of a deteriorated bridge deck is a complex process of planning and execution. The fact that the deck is in need of repair means that it has deteriorated in the service environment. Consequently, the remedial action needs to be more than restoring the deck to its original condition because the deterioration would inevitably occur again. The rehabilitation must result in a structure with improved durability, but there are fewer options in selecting a deck protective system for repair work than for new construction. A decision has to be made on how much concrete should be removed from the deck and what effect the repairs will have on the future performance of the deck. Furthermore, repair work is often carried out while maintaining traffic. The weather, traffic volumes, and budget may place serious operational constraints on the decisions of the bridge maintenance engineer.

Chapter Five discusses factors that influence the selection of the rehabilitation scheme. It also discusses the materials, procedures, and performance of the methods available for bridge deck repair.

REPAIR STRATEGIES

The first decision to be made in planning and designing a bridge deck rehabilitation scheme is whether the deck should be repaired or replaced. If the bridge condition survey indicates deterioration that cannot be repaired, for example, cracking caused by reactive aggregates or severe frost damage, or if the deterioration is so extensive that the deck can be replaced more economically than it can be repaired, then clearly the deck requires replacement. The majority of decks do not fall in these categories, and the decision to replace, repair immediately, or undertake continued maintenance and to repair at a later date becomes much more complex. It is common practice to repair a deck only when the riding quality becomes unacceptable. This is unfortunate because preventive maintenance or repairs made before the riding quality is affected, for example, when delaminations first occur, may be significantly more cost effective (*209*). The most appropriate solution for any particular site can only be reached by following a systematic decision-making process that includes a condition survey of the structure to determine the cause and extent of the deterioration (*210*) and a technical and economic analysis of the alternative rehabilitation schemes.

Although this synthesis is concerned with the durability of bridge decks, bridge deck repairs cannot be undertaken without an evaluation of the condition and load-carrying capacity of the remainder of the structure. If the structure is found to be functionally obsolete or deficiencies are noted in other components that limit its future life, then the deck rehabilitation strategy must be compatible with the life of the whole structure.

The factors that influence the selection of the repair method are 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 repairs on traffic flow.
3. The type, size, condition, 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 the repairs and the availability of funds.

The repair schemes in most widespread use are to patch or inject the bridge with adhesives where temporary repairs are appropriate and to use membranes, concrete overlays, or cathodic protection where more permanent repairs are required. The relative advantages and disadvantages of the more permanent schemes are shown in Table 4. It is the

TABLE 4

RELATIVE MERITS OF REHABILITATION METHODS

Rehabilitation Method	Advantages	Disadvantages
Concrete Overlay (Low-slump or latex-modified concrete)	1. Structural component of deck slab 2. Relatively impermeable 3. Relatively long service life 4. Well-suited to repair of badly spalled or scaled decks 5. Many qualified contractors	1. Not suited to decks with complex geometry 2. Cannot bridge moving cracks 3. Difficult to provide good skid resistance 4. May not stop active corrosion
Membrane with Bituminous Concrete Wearing Course	1. Bridges moving cracks 2. Relatively impermeable 3. Provides good riding surface 4. Applicable to any deck geometry 5. Many qualified contractors	1. Performance highly variable 2. Will not stop active corrosion 3. Service life limited by wearing course 4. Nonstructural component of deck slab 5. Not suitable for grades in excess of 4 percent
Cathodic Protection	1. Stops active corrosion 2. Can be used on decks with moving cracks 3. Provides good riding surface 4. Applicable to any deck geometry	1. Presence of wearing course may accelerate deterioration of the concrete 2. Nonstructural component of the deck slab 3. Continuing maintenance procedure 4. Limited performance history 5. Service life limited by wearing course 6. Specialized contractor and inspection required 7. Electrical power source required

interrelationship of the factors listed above and the merits of the alternative repair schemes that determine the most suitable rehabilitation strategy.

The importance of a structure is dependent upon the volume of traffic at the site and the availability of alternative routes. Thus, repairs to a structure on an urban freeway, where closure of the bridge would divert traffic on to city streets, must be given greater priority than repairs to a small structure in a rural location where the diversion of traffic would not cause as great a hardship. Traffic volumes also influence the choice of repair method because they determine the proportion of the deck that may be closed at any one time and dictate the hours available for the repair work. In extreme cases, work may be done only at night and the repaired areas must be reopened to traffic before the morning rush hour. Such conditions may require the use of expedient methods rather than those that are the most desirable. For example, it may be necessary to rehabilitate the deck in short sections using rapid-hardening patching materials followed immediately by application of a membrane and paving. Under normal conditions, the installation of concrete overlays, cathodic protection, and concrete patching followed by waterproofing and paving all take about the same length of time to complete because concrete removal operations and ancillary work such as expansion-joint and deck-drain modifications are often common to all schemes. Similarly, constraints on temperature and precipitation at the time of installation are approximately the same for all three systems.

Unusual deck geometry, such as large skews or changing superelevation, which results in the crown line not being parallel to the centerline of the roadway, may exclude the use of concrete overlays, particularly low-slump concrete placed with machines having transverse oscillating screeds. The high workability of latex-modified concrete makes it difficult to install on structures with steep grades and crossfalls. Membranes should not be used on grades greater than 4 percent or in areas subject to rapid acceleration, braking, or turning movements.

The amount and type of deterioration on a deck have a very significant effect on the selection of the repair method. The time and cost of repairing a spalled deck with a membrane or cathodic protection increase with the extent of the deterioration because separate deck repairs are required prior to applying the protective system. Where the load capacity of the structure is a factor, this may also exclude the use of a bituminous wearing course. Conversely, concrete overlays are well-suited to badly deteriorated decks because areas of concrete removal are replaced during the paving operation and, because the overlay is a structural component, the load-carrying capacity of the deck slab is increased.

Active cracks generally preclude the use of concrete overlays because they are very susceptible to reflection cracks, which may limit their service life. Recently, a thin overlay consisting of woven glass fiber in an acrylic latex-modified cement binder has been used in experimental construction to rehabilitate a badly cracked deck (*158*). The system exhibits a multiple fracture failure mechanism that enables it to span moving cracks and retain its structural integrity and impermeability. The fiber-reinforced composite has also been installed on cracked decks beneath low-slump and latex-modified concrete overlays, but no performance data are currently available.

The quality of the concrete in the deck slab requires careful study and, if cathodic protection is contemplated, the air-void system must be examined using the linear traverse or point count methods. The bituminous overlay is permeable and, if the concrete is not properly air-entrained, deterioration of the concrete may be initiated, even if the exposed concrete deck slab is free from scaling, because the bituminous overlay places the concrete in a more severe environment.

The most difficult problem in planning a repair strategy is to determine the amount of concrete to be removed from the deck and the effect of placing a membrane overlay on a deck slab if all the chloride contaminated concrete is not removed. At the present time, the only way to be assured of a permanent bridge deck repair is to remove all concrete that contains chlorides in excess of the corrosion threshold value and then prevent further applications of deicing salts from gaining access to the reinforcing steel (*22, 210*). However, deck replacement or the removal of concrete below the top mat of steel are expensive operations and, in view of the number of structures involved, beyond the resources of most highway agencies. Consequently, considerable effort has been expended to identify other methods of deck rehabilitation and, in 1976, the Federal Highway Administration permitted the use, in federal-aid work, of experimental cost-effective reconstruction in which not all the chloride contaminated concrete is removed if it is otherwise sound (*83*). Requirements for evaluating the condition of decks were prepared: Categories of deck conditions were suggested; and concrete overlays, membranes, and cathodic protection were approved for use in projects designated as experimental cost-effective reconstruction. Some states have adopted slightly different criteria for the selection of a repair method that are more appropriate to local conditions and include factors such as the volume of traffic (*211*).

Although the monitoring of installations is a requirement for participation in the program for cost-effective reconstruction, little data are yet available on the effects of the approved systems on continuing corrosion in bridge decks. It is known that if a bridge deck is patched, corrosion will continue and may be accelerated because the patches have a different chloride, oxygen, and moisture content than the remainder of the deck and strong corrosion cells may be established (*65, 210, 212*).

The effect of overlays on continuing corrosion of the steel is less well established. An initial reduction in corrosion activity has been recorded (*42, 213*) on structures in which spalled and delaminated areas have been patched and the entire deck has been waterproofed and paved, but continuation of corrosion activity beneath membranes has also been reported (*42, 57, 67, 192*). There are also numerous other cases where continued corrosion activity beneath membranes has been monitored, but the results have not been published.

A similar trend has been noted for latex-modified concrete overlays (*178*). In some cases, there has been an initial reduction in half-cell potentials (*214*) and, in others, corrosion activity has continued (*215*) or increased within a short period after overlay placement (*213*). Evidence of a reduction, though not necessarily a cessation, of corrosion activity beneath low-slump overlays is more positive (*158, 213, 216*), and unpublished data from Iowa indicate half-cell potentials have continued to decrease during a 3-year period after overlay placement on structures that have been monitored. The long-term performance of concrete overlays in Iowa has been good, even where chloride-contaminated concrete was not removed (*173*), suggesting that corrosion activity slows down, even if it does not cease.

Under laboratory conditions, corrosion activity gradually decreased when concrete overlays (low-slump, latex-modified, internally sealed, and conventional quality concrete) were placed on reinforced concrete slabs to which a chloride content in excess of the corrosion threshold value had been added at the time of mixing (*158*). A significant reduction in corrosion activity in the field is to be expected immediately after placing an overlay because the highest corrosion potentials coincide with the spalls and the delaminated areas. Concrete is removed from these areas prior to overlay, and the exposed reinforcing bars are cleaned by sand blasting. The reduction in corrosion activity may be masked by rapid rust formation on sand-blasted bars, especially if the deck is wet down prior to overlay placement. This phenomenon may account for some of the conflicting trends in corrosion potentials following the application of latex-modified concrete overlays to salt-contaminated decks. The laboratory studies, however, show that, even though the chloride level may be above the threshold value, corrosion will not necessarily continue. The mechanism by which the corrosion activity diminishes is not clear.

It is difficult to generalize when discussing bridge deck repair strategies and it is especially so with respect to costs. Traditionally, costs have been at the top of the list in selecting the method of repair. The cost analyses have to be made not only for local conditions, which will determine such factors as the availability of materials and contractor expertise, but also for the individual structure. Wide variations in costs can be expected for the same method of repair applied to different structures depending upon the size and location of the structure, traffic volumes, other work included in the same contract, scheduling, and the overall volume of construction work at the time of bidding. Life-cycle costs are especially difficult to predict because the service life of each repair method has to be assumed. The service life, however, can be no more than an estimate because the recommended systems have been in use for less than 15 years and, in some cases, less than 5 years. In addition, user costs, that is, delays to the motoring public while repairs are carried out, should also be taken into account. Common practice is to consider the time needed to execute each repair strategy, but rarely is an attempt made to calculate user costs directly in dollars.

Despite the difficulty in determining service life, there is sufficient evidence to indicate that a rigid concrete overlay will perform much more satisfactorily than a membrane when applied to an actively-corroding deck from which the chloride-contamined concrete is not removed. A survey of 149 latex-modified concrete overlays in Ohio, West Virginia, Michigan, and Kentucky (*178*), some of which were more than 15 years old, found that, despite local debonding and cracking, performance was generally satisfactory. This study, together with experience on low-slump concrete overlays, particularly in Iowa, Kansas, and British Columbia, suggests that the economic life of a rigid concrete overlay is at least 15 years. Conversely, delamination and spalling have been recorded beneath membranes within 5 years of application to a salt-contaminated deck.

Estimated costs must be calculated for each repair contract, and the repair strategy must be selected for each structure in the contract on the basis of an evaluation of the structure and an analysis of the alternative repair strategies. Rarely is there an ideal repair strategy, and the method selected will, almost invariably, be a compromise solution that is technically acceptable and economically viable. In some cases, however, economic considerations may be secondary because an inexpensive scheme that is inappropriate, in an engineering sense, should clearly not be implemented. It is also obviously incorrect to apply the same repair strategy to all structures. Local conditions and experience must be included in the decision-making process to determine the most appropriate repair strategy for each individual structure.

CONCRETE REMOVAL

Estimating the quantity of concrete to be removed prior to repairs is not an easy task, especially if it is intended that only unsound concrete be removed. Substantial overruns have not been uncommon. Overruns can make contract administration difficult and result in substantial claims by the contractor. Estimating errors can be minimized by a thorough condition survey as close as possible to the time the work is executed. Where, by necessity, the survey is done the year prior to awarding the repair contract, it is common to increase the estimated quantities by an arbitrary amount, usually 10 to 25 percent, to account for continued deterioration.

A clean, sound surface is required for any repair operation, and the absolute minimum of concrete to be removed is all concrete that is physically unsound, including all delaminated areas. If an overlay is to be installed, a minimum of ¼ in. (6 mm) is removed from the entire deck surface, usually by mechanical scarifiers. The removal of greater quantities of concrete depends upon whether the contract is considered to be permanent or cost-effective reconstruction. In the former case, concrete is removed to below the level of the top mat of reinforcing steel wherever the chloride content exceeds the corrosion threshold value or active corrosion potentials are recorded.

It is common practice to restrict the size of hammers used in concrete removal to prevent damage to otherwise sound concrete. A typical restriction is the use of a 30-lb (14-kg) maximum size jackhammer above the top reinforcing steel and a 15-lb (7-kg) maximum size chipping hammer below the reinforcing steel. A typical removal operation is shown in Figure 18. If only patches are to be installed, as for example, prior to placing a waterproofing membrane or cathodic protection, undercut sawing of the edges of the patches is recommended. If a concrete overlay is to be applied, sawcutting is unnecessary and the edge of the areas of removal should be chipped at about 45 degrees to prevent pockets of entrapped air when placing the overlay.

In experimental, cost-effective reconstruction, concrete is only removed beneath the reinforcing bars where it is physically unsound or if the bond between the concrete and the steel is broken. A useful rule of thumb is to remove the concrete to below the bar in those areas where more than half the perimeter of the bar is exposed after chipping to sound concrete. Where it is necessary to chip below the bar, a clear space of ¼ in. (6 mm) plus the maximum size of the aggregate to be used in the repair concrete must be provided. It is also usual to expose bars that are heavily rusted or where there are heavy rust deposits in the concrete adjacent to the bar. However, the temptation to engage in "rust chasing," or the exposure of all bars showing signs of rust, is to be avoided.

Loose bars should be tied at each intersection point to prevent relative movement of the bars and of the concrete in the repair under the action of traffic in adjacent lanes during the curing period.

Many payment schemes have been devised for concrete removal including area, volume, and even volume of concrete replaced. Payment for concrete removal is often a contentious issue between the contractor and the highway agency, because the process is expensive, but the quantities are small and difficult to measure. Areas are usually irregular, and depths are variable. Because costs increase substantially for removal beneath the reinforcing bars, one successful approach is to divide concrete removal into three pay items: (a) scarifying the deck to a depth of ¼ in. (6 mm), (b) removing concrete to the level of the reinforcing steel, and (c) removing concrete to below the level of the reinforcing steel. In all cases, the pay item is square feet (m^2).

The final stage in preparing the existing concrete surface is blast cleaning of the concrete and the exposed steel. After completion of this operation, the concrete should be carefully inspected and aggregate particles that have been cracked or fractured by scarifying or chipping should be removed to a sound surface.

PATCH REPAIRS

A distinction is sometimes made between temporary repair patches and permanent repair patches. Temporary repairs become permanent if they remain in place. However, because of the progressive nature of the corrosion process, there is no such thing as a permanent patch for the repair of a spalled deck. Thus, the distinction is more a question of intent than reality.

Temporary repairs are made in situations where rapid restoration of the riding quality of the deck is required or where available funds or weather conditions preclude the use of other treatments (*209*). Temporary repairs usually involve filling the spalled areas with a repair material with no significant surface preparation prior to placement. The work is often carried out by state maintenance forces, and

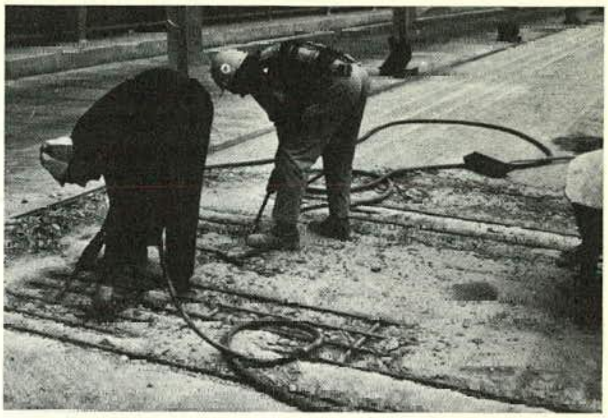


Figure 18. The use of small hammers during concrete removal to prevent damage to the deck slab.

cold-mix asphalt is the material most commonly used. The patches may be required to last only through the winter months and may have to be replaced several times. Hot-mix asphalt is sometimes used to provide a more durable repair.

Permanent repairs are often used to maintain an adequate deck surface until such time as a reconstruction can be undertaken. On a spalled deck, patches are not a permanent solution in themselves. Even so-called permanent patches may last no more than two or three years, or they may accelerate deterioration of the bridge deck concrete adjacent to the patches.

The area to be patched is normally defined by visual observation, sometimes supplemented by delamination detection. The basic steps in the repair process are:

1. A saw cut is made to a depth of 1 or 2 in. (25 or 50 mm) around the spall or delamination.
2. The deteriorated area within the saw cut is removed with chipping hammers.
3. The exposed reinforcing steel is cleaned by wire brushing or sand- or waterblasting.
4. A bonding agent is applied.
5. The repair material is placed and cured.

Featheredging the patching material should always be avoided. Sharp edges, at least 1 in. (25 mm) deep, should be formed by jackhammers or, preferably, by saw cutting. When saw cutting, it is advantageous to tilt the saw blade to key in the patch by making it wider at the bottom than at the deck surface. This can be done by running one wheel of the saw on a plank placed on the deck.

The existing concrete is removed to sound concrete. Some states require removal to below the top mat of reinforcing steel, but practices vary widely from state to state. In areas of very badly deteriorated concrete, full-depth removal may be necessary. In such cases, forms must be attached to the soffit of the deck.

Surface preparation and the selection of the bonding agent are discussed in Chapter Four. For the repair material, in addition to conventional portland cement mixtures, there are a multitude of commercial products available (*217*). Many of the products are, in fact, portland cement concretes with a high cement factor and, not infrequently, a chloride accelerator (*212*). There are also numerous other formulations including those with binders of gypsum, calcium aluminate cement, magnesium phosphate, epoxy, polyester, and other polymers. All are designed for rapid hardening, often under winter conditions and are sometimes referred to as "quick-set" materials. When using proprietary products, the bonding agent is often the binder used in the repair material.

Many of the commercial products are expensive. The quantity of material used in patching is, however, relatively small, and the most expensive component of the operation, especially in urban areas, is often the cost of traffic protection. Consequently, the use of expensive materials can be justified on the grounds of both safety and economy if the patches can be opened to traffic soon after placement. Polymer concrete has been used successfully for rapid repairs to an urban freeway where the high cost of the materials was offset by the inordinately high cost of the traffic protection (*218, 219*). Patches that are well-supported by the underlying deck should be cured until the material has a compressive strength of at least 1,000 psi (7 mPa) before opening to traffic. For full-depth patches, the minimum strength should be at least 3,000 psi (21 mPa).

Many states have experimented to some degree with quick-set materials and opinions vary as to their cost-effectiveness (*209, 217*). There is, in fact, little to be gained by a detailed comparison of individual products because no materials can be expected to last for more than a few years on a corroding deck. Continuing deterioration within and around a patched area is illustrated in Figure 19. The placing of a patch in a bridge deck changes the chloride, oxygen, and moisture content around the steel in the patched area. If the patch contains chlorides, a strong anodic area may be created and corrosion may begin in the patch soon after placement. The opposite is true when concrete is removed from a spalled and delaminated area of a salt-contaminated deck and replaced by material that does not promote corrosion of the reinforcing steel. The new patch creates a differential environment corrosion

Figure 19. Continuing deterioration in and around a patched area on a bridge deck.

cell and becomes a strongly cathodic area, which induces rapid deterioration in adajcent, unrepaired areas of the deck. It is not uncommon to witness an island of repair material in a sea of deteriorated concrete within a few months of placing the patch.

Theoretically, if the steel in a patched area is insulated, it cannot participate in galvanic activity except as a conductor, and the patch has a neutral effect on continued corrosion of the steel (*210*). Insulating the patch may be accomplished either by using an epoxy bonding agent on both the concrete and the steel prior to placing a portland cement concrete patch or by the use of a dielectric repair material such as an epoxy or polyester mortar. The practice was used in California but now has been discontinued. There is no evidence that epoxy and polyester resin formulations are any more durable than other materials (*209*), and corrosion will continue in other areas of the deck. Consequently, once corrosion-induced deterioration is observed in a bridge deck, it cannot be solved by patching. Patches are only useful to restore the riding quality of a deck until more permanent methods of rehabilitation can be scheduled. The service life of all repair materials is limited, and selection should be based on convenience and the overall cost of the patching operation.

On decks where patches are needed to repair other than corrosion-induced deterioration, for example, a localized area of heavy scaling or fire damage, a more permanent repair is feasible. In such cases, portland cement concrete is the preferred repair material, where weather and traffic conditions permit, because it is the most compatible with the remainder of the deck.

INJECTION REPAIRS

The injection of epoxy resin as a method of repairing cracks or filling voids in structures has been known for many years (*151, 220*). The technique was first applied to the repair of delaminations in bridge decks by the Kansas State Highway Commission in 1964, and the equipment and procedures have been significantly improved since that time (*46, 221*). In some circumstances, injection repairs can be a cost-effective method of extending the life of a bridge deck before permanent repairs are made. The advantages of the process are that it is relatively simple and can be carried out with minimum disruption to traffic.

The procedure as developed and used in Kansas consists of the following steps: (a) identifying delaminated areas by sounding hammers or a chain drag; (b) sealing potential leakage points within the delaminations with epoxy paste; (c) locating the steel using a pachometer; (d) drilling holes 2 to 3 in. (50 to 75 mm) deep to miss the steel and ensure that the bottom of the hole is below the delamination; (e) injecting epoxy into the delamination under pressure using motor-driven pumps; (f) scraping up excess epoxy and sprinkling exposed epoxy with sand.

The holes are drilled using hollow-stemmed carbide-tipped drill bits connected to a vacuum cleaner. Drilling dust is thereby removed from the drill hole and cannot block access to the delamination. Pumping pressure is controlled by a variable speed control on the pumps and is determined by the area and thickness of the crack and the viscosity of the epoxy. Normal operating pressures are 20 to 40 psi (140 to 280 kPa). The two components of the epoxy resin are brought together and efficiently mixed immediately ahead of the injection nozzle. This system maximizes the pot life of the epoxy, minimizes material wastage, and aids in cleaning the injection apparatus. Less-sophisticated equipment, such as caulking handguns or pressure pots similar to those used in paint spraying operations, has been used for epoxy injection in other applications (*222*) but not widely on bridge decks.

Coring has shown that the epoxy injection is effective and achieves good crack penetration (*46*). The technique is suited to the repair of bridges in which delaminations have developed but have not progressed to open spalls, providing that the delaminations are free from dirt. It is also preferable that the concrete be dry. The epoxies normally used for injection will tolerate the presence of moisture, although bond strengths are reduced. Epoxy injection is no more than a continuing maintenance method of extending the life of a bridge deck before permanent repairs are made, because it does not prevent the subsequent development of further delaminations in injected areas at a different depth below the deck surface. Epoxy injection should not be used prior to the installation of cathodic protection because the epoxy insulates the underlying steel from the cathodic protection circuit.

Polymers other than epoxies have been injected into bridge decks to repair delaminations but with mixed success (*39*), often because the viscosity was so low the material flowed out of the cracks before polymerizing. Many of the polymers under investigation have less tolerance of moisture than epoxies, and prior drying of the deck may be necessary (*135*).

Polymer rebonding of highly deteriorated and delaminated concrete has been successfully undertaken in field trials (*135*) on a deck in an advanced stage of deterioration due to frost action. The techniques are similar to those of polymer impregnation of new decks except that, where necessary, the underside of the deck must be sealed to prevent monomer loss through cracks. Any asphalt on the deck must be removed because it acts as an inhibitor to the polymerization reaction. Although small-scale field trials have shown the techniques to be effective, the costs are high. Drying, impregnation, and polymerization have been estimated to cost \$6.50/ft² (\$70/m²) for a badly deteriorated deck (*144*). The system is a potential alternative to deck replacement, but further development work and performance data are required before such techniques become routine field processes.

CHLORIDE REMOVAL

A possible method of halting the corrosion of existing bridge decks is the neutralization or removal of chlorides contained in the concrete. The possibility of flushing away the salts by application of water to the underside of the deck has been investigated in the laboratory but was found ineffective and impractical because an adequate flow of water through the concrete could not be achieved (*223*).

An electrochemical method has been developed (*223, 224*) in which the chloride ion migrates through and out of the concrete under the action of an electric field applied between the reinforcing bars, which are made cathodic, and anodes attached to the deck surface. The method found to be most satisfactory on the basis of laboratory tests was to apply 100 V DC voltage for a 12 to 24-hour period using a platinized titanium anode and calcium hydroxide as the electrolyte. Platinized titanium was chosen to fulfill the requirements of low anode polarization characteristics, good corrosion resistance, and mechanical strength. An ion exchange resin was used to collect the chloride ions and prevent the evolution of chlorine gas. The procedure was effective in removing chloride ions from around the reinforcing bars, but the overall efficiency was low, largely because of the presence of hydroxyl ions in the concrete which, like the chloride ions, have a negative charge.

An undesirable side effect of the method is that temperatures around 200 F (93 C) are generated in the concrete, which may induce cracking. The effect of the treatment on the porosity of the concrete was not measured. To be effective, the treatment must be followed by deck sealing, for example by polymer impregnation or the application of a membrane, to maintain the steel in a passive condition.

A field test was undertaken in 1975 on a small section of a bridge deck in Ohio. The only difficulty was in ponding the ion exchange resin on the deck. The test was successful in removing up to 90 percent of the chloride from above the steel and 88 percent of the chloride from concrete immediately adjacent to the steel. No cracks were introduced into the deck by the heat treatment. Potential measurements showed that steel that was actively corroding became passive after treatment. The passive condition has remained for three years even though the deck has been subject to routine salting in winter and has been protected only by a single application of linseed oil. Further study is required to explain this performance.

A parallel investigation has been undertaken by the Kansas DOT using potentials up to 220 V DC and current densities of about 2 A/ft² (22 A/m²) with a copper screen as an anode (*225*). This procedure was also found to be effective in removing chloride ions, but the permeability of the concrete was also significantly increased. Screening tests were conducted to identify compounds that would simultaneously impregnate the concrete as the chlorides are driven out and polymerize in situ. A field trial was carried out on a section of an old deck using furfuryl alcohol as the impregnant. Although the monomer was found to penetrate the concrete readily, polymerization was incomplete, with the result that the concrete later disintegrated and use of furfuryl alcohol was discontinued.

At the present time, neither of the above methods is practical because the equipment used in the field trials is not suitable for treating a full-size deck slab. Neither have the costs of the treatment been estimated accurately, and considerably more work will be required to develop the methods to the stage at which they are economically and practically viable.

CONCLUSION

The repair of a deteriorated bridge deck is a much more complex process than the construction of a deck protective system on a new deck. Within the constraints of budget, work force, traffic control, and weather, the bridge maintenance engineer must choose the most cost-effective treatment for the remaining service life of the structure. Periodic, preventive maintenance, or repairs made before the riding quality of the deck is affected, can, in some circumstances, be a very cost-effective approach to bridge maintenance. The factors influencing the decision as to the repair method are so numerous that use must be made of the evaluation techniques now available. The technical and economic implications of each of the alternative repair schemes must then be carefully assessed.

Many of the repair methods now employed have only been used for about five years. Though these techniques have resulted in the ability to repair decks that would have been replaced in the past, research is continuing into the effect of repairs on future deck performance. Consequently, current methods of repair can be expected to be subject to continuing improvement.

Not only is bridge deck repair difficult, there is always a compromise between what is desirable and what is attainable within the existing constraints. Greater emphasis has to be placed on the design, selection of materials, and the construction of bridge decks to prevent premature deterioration. Money now spent on repairs would have been far more beneficial if spent to achieve a higher quality of construction. As the poet, Proudfit, wrote almost a century ago, and highway engineers have discovered more recently, "Nature abhors imperfect work."

CHAPTER SIX

FUTURE RESEARCH

Past and present research on bridge deck durability has been described in the first five chapters of this report. A comprehensive list of active and recently-completed research projects has been prepared by the FHWA (*92*).

Although much progress has been made in understanding the basic mechanism of corrosion and in the development of test methods, materials, and construction practices for deck protective systems, a considerable volume of research is required before it can be said that economical solutions to the problems of bridge deck durability are readily available. Among the areas in which further study is needed are those given in the following list. The list includes activities that are extensions of current projects and some new studies that have already been proposed (*92*).

FUNDAMENTAL STUDIES

1. Further work is needed to define the conditions under which reinforcing steel in concrete will corrode. A better understanding of the mechanism by which chloride ions depassivate the steel is necessary. Once the chloride corrosion threshold is exceeded, it appears that moisture, rather than oxygen, may be the controlling factor in determining the onset of corrosion. If this proves to be true, the minimum moisture content required to support the corrosion reaction needs to be ascertained.

2. The role of concrete quality and cover in determining whether spalling of the concrete inevitably follows corrosion of the reinforcing steel needs to be determined. There are numerous bridge decks in which active corrosion potentials are known to have existed for several years but which have in excess of 2 in. (50 mm) of good quality concrete cover and no physical distress has occurred. It is not known whether the corrosion activity in these decks has stabilized or whether delamination and spalling will occur at some time in the future.

3. Monitoring the long-term performance of many of the deck protective systems now being applied to salt-contaminated decks, especially concrete overlays and membranes, must be given a high priority. Such studies should be directed to better understanding the corrosion of reinforcing steel under service conditions, to defining the factors affecting the performance of the various deck rehabilitation schemes, and assessing the service life of the latter. It is particularly important to determine if the initial reductions in corrosion activity beneath rigid overlays will be sustained or whether this is a temporary phenomenon. A complete explanation of the different effects of concrete overlays and waterproofing membranes on corrosion activity is also needed.

4. The chloride content of the ingredients of concrete is highly variable, especially for natural aggregates from different sources. Work is required to identify the factors that determine whether the chlorides are available to contribute to the corrosion process when the materials are used in concrete.

TEST METHODS

The needs for further development of test methods fall into three general categories: (a) new test procedures to measure factors influencing the onset and rate of corrosion, (b) nondestructive and in situ test methods to replace some existing procedures, and (c) more rapid determination of some parameters currently measured by other methods.

Specific areas in which new or modified test methods are required are:

1. A nondestructive method of measuring the rate of corrosion. Existing half-cell tests determine the presence of corrosion activity but give no indication of the rate of the reaction. Because it is the rate of corrosion that measures the effectiveness of a deck protective system and determines the length of time before physical distress occurs, development of this test procedure is a high priority.

2. There is currently no test method to measure the oxygen concentration in concrete. Recent research suggests that the level of oxygen in concrete will normally be sufficient to support the corrosion reaction, though this needs to be confirmed.

3. Further work is needed to refine the existing methods and develop new methods of measuring the chloride content of hardened concrete in situ.

4. A procedure is needed to measure the concrete mixture ingredients' chloride content available for corrosion. This test method can not be developed until the criteria for availability have been established in fundamental studies.

5. An improved method of determining the permeability of concrete to both water and chloride ions would be of considerable benefit in screening new systems and in quality assurance testing of routine construction. Although there have been attempts to measure the permeability of concrete for many years, none of the test methods available is completely satisfactory.

6. Improved methods of measuring the degree of consolidation of concrete are required. Existing nuclear devices have limited accuracy, and only random sample measurements can be taken. The new method should be capable of traversing the deck and be calibrated to provide a continuous readout of the degree of consolidation.

7. Current bridge evaluation techniques need to be refined. Remote sensing techniques offer promise in that they permit rapid evaluation of the deck and, if the equipment is air borne, do not require partial closure of traffic lanes. This increases safety and has the potential to reduce costs, especially for freeway structures in urban areas where traffic control operations are very expensive.

8. Techniques for evaluating the condition of asphalt covered decks are rather crude and rely principally upon coring and partial removal of the asphalt by sawing. Methods that will quickly and nondestructively determine the condition of the concrete beneath an asphalt concrete wearing course are urgently needed.

DEVELOPMENT OF MATERIALS

The need for materials development continues in the following areas:

1. The area of materials development that has potential for the greatest impact on bridge deck durability is corrosion inhibitors that can be added to the fresh concrete at the time of construction. Advances have been made, but to date a chemical additive has not been identified that can be guaranteed to prevent corrosion of embedded steel without short or long-term negative effects on the other properties of the concrete. If such a material were economically available, it would have a dramatic effect upon the deck protective systems now in use because of the ease with which its use could be implemented.

2. There is a continuing need to identify new materials and to modify existing materials to make them technically superior and more economical. Such areas include: (a) beads for internally sealed concrete that are less expensive and preferably do not require heat treatment; (b) other coatings, both metallic and organic, for reinforcing bars and improved methods of application; (c) other latex modifiers for concrete; and (d) development of a thin, conductive topping for use in impressed current cathodic protection systems—one that would not require a separate wearing course.

3. The evaluation of superplasticizing admixtures to determine their usefulness in bridge deck construction and repair will probably be undertaken in the near future. A more basic study is also needed to try to negate the high rate of slump loss often experienced when using such admixtures.

4. Measurement of the consumption rate of zinc in concrete is necessary to identify the service life of galvanized reinforcement.

5. Materials should be developed that would be suitable for bridge deck overlays no more than ½ in. (13 mm) thick.

6. The search for an economical, noncorrosive deicer will no doubt continue. Extensive research has already been undertaken in this area, and the chances of a significant breakthrough appear slight.

CONSTRUCTION PRACTICES

Construction practices need to be modified to ensure that the design cover is achieved and to prevent premature deterioration caused by improper installation of deck joints and drains. More specific requirements are:

1. The development of improved methods of fixing the steel and placing the concrete in the deck to guarantee the design cover. Such methods should also lead to an improvement in the riding quality of the deck surface. One approach to this problem is to increase the use of precast components in deck construction.

2. Improved methods of calculating the initial settings and the installation of expansion devices.

REPAIR PRACTICES

The ideal solution to the problems of bridge deck deterioration is a material that could be placed on a deck surface if corrosion of the steel began that would neutralize the effect of all chlorides in the concrete. Other requirements for the material are that it be inexpensive, readily available, easy to apply, and have no negative effects on the concrete or the environment! This "magic powder" has, however, proved most elusive. While the search continues, other necessary developments, which are more readily attainable, are listed below:

1. Techniques for the rapid repair of deteriorated decks.
2. Establishment of criteria for concrete removal in

terms of its chloride content and corrosion activity of the steel.

3. Refinement of polymer impregnation techniques to increase penetration and reduce costs, preferably by eliminating the heating and drying cycles.

4. Improved methods of internally sealing concrete.

5. Refinement of design requirements, equipment, and installation procedures for galvanic and impressed current cathodic protection systems.

6. Improved methods of removing chlorides from bridge decks and determination of their long-term effects on the corrosion activity of the reinforcing steel.

7. Methods of rehabilitating decks with active cracks. This is one area that has received very little attention to date and yet restricts the use of some rehabilitation schemes, especially rigid overlays. Active cracks include those that move with changes in temperature, in which case the response is relatively slow, and those that move under live loads, in which case the response is almost instantaneous.

8. It is often difficult to estimate quantities for inclusion in a repair contract, and there have been numerous examples of cost overruns and serious problems in contract administration on the part of both the highway agency and the contractor. There is considerable scope for the development of alternative bidding methods and conditions of contracts for bridge deck repair work.

REPAIR METHODOLOGY

Bridge deck deterioration and rehabilitation is a complex subject. Several alternative repair schemes must be evaluated prior to selecting the most appropriate repair method for an individual structure. The criteria for selection of repair schemes need refinement, and manuals for use by design teams need to be prepared covering all facets of deck investigation and contract preparation.

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APPENDIX A

CURRENT DECK CONSTRUCTION AND REPAIR PRACTICES

USE OF DEICING SALTS

Only 2 of the 48 states responding to TRB's September 1977 Survey on current practices of bridge deck construction and repair do not use deicing salts. In some cases the use is very limited and, in many of the southern and western states, deicers are used in only part of the state. In some states, where snow and ice occur infrequently, bridge decks are not salted when salt is applied to adjacent sections of the pavement. Elsewhere, salt use is widespread and an essential component of the winter maintenance program.

QUALITY OF CONCRETE

The majority of states in the snow belt areas have recognized the protection offered reinforcement against corrosion by the provision of additional amounts of cover of high-quality concrete. Many states have modified specifications in the past two years to reduce the maximum permissible water-cement ratio of the concrete and to increase the depth of cover. In some cases, the level of inspection has been increased to ensure that the specified cover is achieved.

PROTECTIVE SYSTEMS FOR NEW DECKS

In accordance with FHWA directives, states are now providing positive protection against corrosion-induced deterioration on federal-aid system bridges. Most states are also using the same methods on state funded projects. Forty-six of the respondents have installed protective systems on bridge decks; details of the number of installations of each of the different systems used are given in Table A-1.

In the majority of states, more than one protective system has been used. In some cases this is a reflection of the different climatic conditions within the state; in others, policies change with the funding of the work or the AADT of the highway. The multitude of protective systems in use is also a reflection of the fact that most deck protective systems have been developed since 1971, and many systems are still considered experimental. These experimental installations are being monitored and policies revised as experience is gained with each system. There is no reason to expect that policies will be any less subject to change in the near future.

In some states, double protective systems such as epoxy-coated bars and a concrete overlay have been installed on structures that have been identified as critical components of the highway network, usually on urban freeways.

States were asked in a survey conducted by the U. S. General Accounting Office (GAO) in September 1977 to state the preferred protective system for new deck construction. Thirty-seven states responded and the results are given in Table A-2.

The simplicity of concept, ease of implementation, and the existence of specifications and approved products accounts for the popularity and widespread use of epoxy-coated bars. The use of galvanized bars has been curtailed by the FHWA limit on experimental structures in federal-aid projects. Concrete overlays have been used extensively in some states as the second stage in two-stage construction, but their use has been limited by the availability of specialized contractors and engineering and inspection expertise within state highway organizations. An equal number of states have installed low-slump and latex-modified overlays, sometimes at the contractor's option on the basis of competitive bids. Ten states have installed internally sealed concrete overlays. All the installations are considered experimental. None of the states are proposing to adopt this system as a standard policy for deck construction.

The use of waterproof membranes has polarized the states more than any of the other systems. States are sharply divided as to the merits of placing bituminous concrete wearing courses on bridge decks. A number of states have installed membranes on an experimental basis and have stopped their use because of unsatisfactory per-

TABLE A-1

PROTECTIVE SYSTEMS ON NEW DECKS

Protective System	Standard Procedure	Experimental Installation	Total
Concrete cover ≥ 3"	5	4	9
Sealants[1]	8	5	13
Epoxy-coated bars	17	9	26
Galvanized bars	1	10	11
Low-slump concrete overlay	10	10	20
Latex-modified concrete overlay	12	8	20
Internally sealed concrete overlay	0	10	10
Membrane	19	14	33
Cathodic protection	0	5	5

(1) Not for protection of reinforcing steel against corrosion.

TABLE A-2

PREFERRED PROTECTIVE SYSTEM IN DECK CONSTRUCTION (GAO SURVEY)

System	No. of Responses
Epoxy-coated bars	20
Low-slump concrete overlay	4
Latex-modified concrete overlay	1
Membrane	7
Cathodic protection	1
Other	4

formance. Generalizations regarding the performance of membranes are, however, difficult because of the large number of products available and the sensitivity of some products to conditions at the time of installation and to workmanship. Many states have installed the four products currently approved by the FHWA; others, particularly the New England states, continue to use the built-up systems that have been in use for several years.

There have been few installations of cathodic protection systems on new decks. Although one respondent to the GAO survey indicated cathodic protection is the preferred method of protecting new decks, all existing installations are considered experimental.

DECK REPAIR METHODS

Forty-four of the 48 respondents to the survey are currently involved in deck repair programs. The number of states using each of the repair methods is given in Table A-3.

Some of the applications included in the responses to the survey as deck repair methods have been installed as a second stage of construction because the deck was originally built without a deck protective system.

The protective systems used in the repair of bridge decks parallel the practices on new decks within the individual states. Techniques such as patching, injection of polymeric materials, and the application of surface sealants are generally used for temporary repairs or on decks with only a small amount of deterioration. Where a more permanent repair is required, or where the deck is badly deteriorated, two systems are in widespread use: (a) the application of a concrete overlay; (b) repair of the deteriorated areas of the deck followed by application of a membrane. Each of these two methods has been used in 28 of the 48 states responding to the survey, though the GAO survey has indicated that concrete overlays are preferred by most states. Thirty-six states responded to the GAO question, and the results are given in Table A-4.

TABLE A-3

REPAIR METHODS ON EXISTING DECKS

Repair Method	Standard Procedure	Experimental Installation	Total
Deck replacement	5	0	5
Patching[(1)]	5	6	11
Injection repairs	3	2	5
Sealants	4	5	9
Normal slump concrete overlay	4	0	4
Low slump concrete overlay	15	3	18
Latex modified concrete overlay	17	7	24
Membrane	22	6	28
Cathodic protection	0	9	9

[(1)] Does not include patching prior to further treatment (e.g., application of a membrane).

A greater number of states have installed latex-modified concrete overlays than low-slump concrete overlays. This is a reflection of the historical development of each system and the more widespread availability of contractor expertise for the installation of latex-modified concrete overlays. On badly deteriorated decks, a concrete overlay is often the only alternative to deck replacement. No internally sealed concrete overlays have been placed on existing decks.

As with new deck construction, states are sharply divided on the use of membranes for repair. In many of the states where membranes have been used, many different products have been placed.

Cathodic protection has been installed on structures in nine states, and all the installations are considered experimental. Although the performance to date has been generally favorable, more widespread use has been restricted by the necessity to use technology not familiar to most highway engineers, the practical difficulty of obtaining an economical conductive mix in some parts of the country, and policies in some states which require exposed concrete bridge decks.

TEST METHODS

Forty-three of the 48 respondents to the survey supplement the visual examination of bridge decks by at least one of the physical test methods listed in Table A-5.

TABLE A-4

PREFERRED PROTECTIVE SYSTEM IN DECK REPAIR (GAO SURVEY)

System	No. of Responses
Low-slump concrete overlay	8
Latex-modified concrete overlay	14
Membrane	9
Cathodic protection	1
Other	4

Although physical testing is in widespread use, including the measurement of chloride contents and half-cell potentials (tests not used routinely prior to 1971), the extent of use varies considerably. Many states employ some of the tests only in research projects or on federal-aid projects to comply with the requirement that 10 percent of "experimental cost-effective restorations" be monitored.

TABLE A-5

NUMBER OF STATES USING PHYSICAL TEST METHODS IN BRIDGE DECK EVALUATIONS

Test Method	No. of States Using Test Method
Chloride content	36
Half-cell potentials	37
Delaminations	42
Depth of cover	20

Most states use only the delamination survey, supplemented by visual examination in the field at the time of repair, to determine the quantity of concrete to be removed from a deteriorated deck. Common practice is to remove only the concrete that is physically unsound. A few states determine the area of concrete removal from the results of the half-cell potential survey. Rarely is a conscious attempt made to remove all the concrete containing more than 2 lb Cl^- per cubic yard of concrete, though some states require the removal of concrete to below the level of the top steel in all delaminated areas. Where taken, chloride analyses and, to a lesser extent, half-cell potential measurements are used to assess the over-all condition of a structure rather than determine the specific areas of concrete removal. Chloride contents may be used to determine the need for repair and to influence the selection of the repair method.

A summary of the responses to the questionnaire is given in Table A-6. Some of the data were incomplete or inaccurate, and consequently Table A-6 contains errors and omissions. The table is, however, sufficiently reliable to be a useful indicator of practices current at the time of the survey.

TABLE A-6

TRB SURVEY OF CURRENT PRACTICES OF DECK CONSTRUCTION AND REPAIR (SEPTEMBER 1977)

STATE	Deicers Used	Spec. Min. Cover	Max. W-C Ratio	PROTECTIVE SYSTEMS ON NEW DECKS: None	Cover ≥ 3 in.	Surface Sealant: Linseed Oil	Surface Sealant: Polymer Impreg.	Surface Sealant: Other	Coated Bar: Epoxy	Coated Bar: Galvanized	Concrete Overlay: Low-Slump	Concrete Overlay: Latex-Modified	Internally Sealed	Waterproof Membrane	DECK REPAIR METHODS: Deck Replacement	Patching (1): Concrete	Patching (1): Epoxy	Patching (1): Other	Injection Repairs	Surface Sealants	Concrete Overlay: Normal Slump	Concrete Overlay: Low-Slump	Concrete Overlay: Latex-Modified	Membrane	Cathodic Protection	TEST METHODS: Chloride Content	Half-Cell Potential	Delaminations	Depth of Cover
Alaska	X	2-1/2	0.44											S							S								
Arizona	X	2-1/2	0.52											E						S						X	X	X	
Arkansas	X	2	0.40			S		E							S					E						X		X	
California	X	2	0.45				E						E	S										S	E	X	X	X	
Colorado	X	2^(2)	0.44	S			E					E		S			E	E		E			E	S		X	X	X	X
Connecticut	X	2	0.44											S										S		X	X	X	X
Delaware	X	2^(3)	0.40		S				S														S			X	X	X	
Florida		2	0.44	S					S																				
Georgia	X	2	0.46			S																		S		X	X	X	X
Hawaii		1-1/2	0.53	S																									
Idaho	X	2-1/2	0.44		E		E		E	E	S	S	E	E		E	E	E						E	E	X	X	X	
Illinois	X	2	0.44						S		E			S						E				S	E			X	
Indiana	X	2-1/2	0.49						E	E		S		S									S	S				X	
Iowa	X	2-1/2	0.44						S	E	S	S		E								S	S			X	X	X	
Kansas	X	3	0.49		E	S		E			S			E		E	E	E	S	E		S		E		X	X	X	X
Kentucky	X	3	0.44			E			S	E	S	S	E	E			E					S	S	E		X	X	X	X
Louisiana	X	2	0.48			S								E		E	E									X	X	X	X
Maine	X	2^(3)	0.42		S										S						S		E	S			X		
Maryland	X	2-1/2	0.44	S					E		E	E			S							E	E			X	X	X	X
Michigan	X	3	0.49			S			S	E	S	S							E			S	S	S		X	X	X	
Minnesota	X	3	0.42		S				S		S	S			S							S	S	E		X	X	X	
Mississippi	X	1-1/2	0.49	S																									
Missouri	X	3	0.44		S				E		E	E		E									E	S	E	X	X	X	X
Montana	X	2-3/8	0.44	S							E							S										X	
Nebraska	X	2-1/4	0.49						S	E	S	S	E	S	S	S			E			S	S		E	X	X	X	X
Nevada	X	2	0.44						E		E	E		S										S				X	
New Hampshire	X	2-1/4	0.44											S										S		X	X	X	X
New Jersey	X^(5)	2	0.47						E	E	E		E										E	E		X	X	X	X
New Mexico	X	2	0.47						S		E	E		E								S				X	X	X	X
New York	X	2	0.44						S	E				E								S	S	S		X	X	X	
North Carolina	X	2-1/2	0.42						S			E		E				E		E		S				X	X	X	X
North Dakota	X	3	0.42		S				S													S				X	X	X	X
Ohio	X	2	0.50	S	E				E	E			E	S								E	E	S	E	X	X	X	
Oklahoma	X	2	0.49								S	S	E	S								S	S		E	X	X	X	X
Oregon	X^(5)	2-1/2	0.43						S					S								S	S	S		X	X	X	X
Pennsylvania	X	2-1/2	0.47						S	S			E	E								S	S			X	X	X	
Rhode Island	X	2	0.44						E			E		S						S				S	E	X	X	X	
South Carolina	X	2	0.49			S								S															
South Dakota	X	2-1/2	0.44						S		E	S		E					S			S	S	S		X	X	X	X
Tennessee	X	2	0.50								S	S		S								S	S	S			X	X	
Texas	X	2	0.49			S	E				E			S		S	S	E								X	X	X	X
Utah	X	2^(3)	0.44								E	E		S		S	E	E						S		X	X	X	
Vermont	X	2	0.44											S										S		X	X	X	
Virginia	X	2-1/2	0.46			S			S		S	S	E			S	S	S		S	S		S	S		X	X	X	X
Washington	X	2	0.44										E	S					S				S	S				X	
West Virginia	X	2^(4)	0.44		E		E	E	S			S		E						S			S			X	X	X	X
Wisconsin	X	2-1/2	0.40	S					E					E				S			S	E	E	E	E	X	X	X	
Wyoming	X	2	0.44	S						E													S	S		X	X	X	

Legend: E - Experimental Installation
S - Standard Procedure

Footnotes:
(1) Patching prior to other procedures, e.g., waterproofing, is not included.
(2) 2-1/2 in. on bare decks
(3) 3 in. on bare decks
(4) 3-1/2 in. on bare decks
(5) Deicing salts not applied to bridge decks.

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To share in the tasks of furthering science and engineering and of advising the federal government, the National Academy of Engineering was established on December 5, 1964, under the authority of the act of incorporation of the National Academy of Sciences. Its advisory activities are closely coordinated with those of the National Academy of Sciences, but it is independent and autonomous in its organization and election of members.

TRANSPORTATION RESEARCH BOARD
National Research Council
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