NATIONAL COOPERATIVE
HIGHWAY RESEARCH PROGRAM REPORT **297**

EVALUATION OF BRIDGE DECK PROTECTIVE STRATEGIES

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

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TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. SEPTEMBER 1987

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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NCHRP REPORT 297

Project 12-32 FY'86

ISSN 0077-5614

ISBN 0-309-04566-5

L. C. Catalog Card No. 87-50965

Price $12.00

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Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

Printed in the United States of America

FOREWORD

By Staff
Transportation Research Board

This report contains an evaluation of existing information on the performance of various protective strategies used on concrete bridge decks at the time of their construction. Corroding reinforcing steel has been a major cause of bridge deck deterioration. Over the years, several methods and techniques have been employed during construction to prevent or inhibit the egress of chloride ions from deicing salts. Chloride ions are a major contributor to the corrosion process. Bridge construction, design, and maintenance engineers will be interested in the findings and recommendations of the research. Researchers will find the report to be an excellent source of data on the effectiveness of the various strategies.

During the 1960's and early 1970's, corrosion of steel reinforcement embedded in concrete contaminated by chloride deicing chemicals was determined to be a major cause of concrete bridge deck deterioration. As a result, various bridge deck protective strategies were developed, such as epoxy-coated steel reinforcement, latex-modified concrete overlays, high density concrete overlays, interlayer membranes, and thicker concrete cover over steel reinforcement. Laboratory studies and early experience indicate that these strategies were effective in improving the performance of bridge decks. However, because of the large national investment in bridges and their importance in the efficient operation of highways, it was appropriate to examine the performance of these bridge deck protective strategies to see if original expectations were being attained and to determine whether unforeseen problems had occurred.

Under NCHRP Project 12-32, "Evaluation of Bridge Deck Protective Strategies," the Washington State Transportation Center, University of Washington, reviewed existing information on the effectiveness of the different common strategies for protecting bridge decks against the harmful effects of chloride ions from deicing chemicals. Performance information on the various strategies was acquired through literature reviews, a survey of state transportation departments, and field visits to selected states. Factors involved in the design, construction, and maintenance of bridge decks that affect serviceability and life were identified. Recommendations for effective protective strategies were developed to provide extended bridge deck service lives.

The original scope of this NCHRP project included two phases. The first phase, which is reported herein, was designed to evaluate existing information. The second phase was to further investigate previously noted problems and conduct a field study. At the conclusion of Phase I, the NCHRP project panel judged the findings to be of great value and elected not to proceed with Phase II. This was based on the belief that, with the resources available, little new information would be developed to significantly modify the results of Phase I.

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ACKNOWLEDGMENTS

The research reported herein was conducted under NCHRP Project 12-32 by the Washington State Transportation Center (TRAC) at the University of Washington. Dr. Neil Hawkins, Professor and Chairman, Civil Engineering, and Mr. Khossrow Babaei, Senior Research Engineer, were the principal investigators for the study.

Appendix D of this report was written by Mr. James Guarre, Project Director, and Mr. Manfred Zinzerling, Project Engineer, both from ABAM Consulting Engineers. Dr. Ronald Terrel, Professor Emeritus of Civil Engineering, University of Washington, and Mr. Ed Henley, Bridge Technology Development Engineer, Washington State Department of Transportation, served as review panel for this project.

Special acknowledgment is made to the representatives of various state transportation departments who supplied information and enabled the research agency to conduct on-site visits of protected bridge decks. In particular, sincere appreciation is expressed to Mr. Willis Whitney, Bridge Condition Engineer, and Mr. Keith Anderson, Research Specialist, Washington State Department of Transportation; Mr. William Chamberlin, Research Engineer, and Mr. Paul St. John, Concrete Specialist, New York State Department of Transportation; Mr. Andrew Halverson, Research Project Engineer, and Mr. Mark Hagen, Research Assistant, Minnesota Department of Transportation; and Mr. Jerald Malasheskie, Chief of Evaluation and New Product Section, and Mr. Dale Mellott, Research Project Engineer, Pennsylvania Department of Transportation. Grateful acknowledgment is also extended for the excellent cooperation provided by the 46 state transportation departments who responded to the survey of this study.

Evaluation of Bridge Deck Protective Strategies

SUMMARY

Premature deterioration of concrete bridge decks is a major concern for this nation's highway agencies. Deterioration results primarily from winter salt applications causing corrosion of embedded reinforcing steel, and from bridge deck surfaces being exposed to direct impact from traffic and severe changes in temperature and moisture. To prevent premature deterioration, various bridge deck protection strategies have been used on newly constructed decks nationwide for 10 years or more. Concern with the prevention of reinforcing bar corrosion has generally directed development of those strategies. The more common protective strategies are (1) 3 in. or more of cover over the top reinforcing steel; (2) low-slump dense concrete overlays; (3) latex-modified concrete overlays; (4) interlayer membrane/asphaltic concrete systems; and (5) epoxy-coated reinforcing steel. The performance of those protective strategies was examined through a literature review, a survey of transportation departments, and visual inspection of selected protected decks for evidence of deterioration. Findings on the life cycle, cost effectiveness, and reliability of each strategy are presented in this study.

Concrete protective systems (i.e., increased cover depth, low-slump dense concrete overlay, or latex-modified concrete overlay) were found resistant, but not impermeable, to salt infiltration. Corrosion of embedded bars in decks using such systems is to be expected sometime within 50 years, depending on the rate of salt application. Variations in concrete protective strategies were devised that should provide 50 years or more of corrosion-free service life.

Interlayer membrane/asphaltic concrete systems were found effective in preventing salt intrusion into the underlying deck. However, after 15 years of service, membranes have deteriorated due to aging and repeated stressing caused by traffic. The systems have to then be removed and replaced.

Epoxy coating the reinforcing steel prevents its corrosion. However, breaks in that coating, or damage during construction or service provide potential sites for accelerated corrosion and fatigue fractures under repeated flexing. Epoxy coating of the deck's bottom reinforcing mat, as well as its top mat, is prudent. That procedure mitigates corrosion at breaks or damaged areas of the top mat. The long-term durability of epoxy coating in chloride contaminated concrete is unknown. The presence of pinholes in the coating and the durability of the coating's adhesion to the bar are causes for concern. Laboratory tests have shown that the bond between coated bars and concrete is adequate, although coating causes slight increases in development lengths. However, for the field the concern is possible loss of the coating's adhesion because of concrete cracking and repeated service stressing.

Regardless of the deterioration caused by corrosion, bridge decks may require maintenance in the form of resurfacing or overlaying before the end of their 50-year

service period. This maintenance may be due to traffic action, a severe environment causing surface distress, or stripping of the overlay (especially interlayer membrane systems) from the deck.

The various causes of cracking in bridge decks, and the contribution of cracking to corrosion-induced deterioration, were evaluated. Cracking in decks with increased cover to the top steel, and in decks overlaid with low-slump dense and latex modified concretes, can be sufficiently extensive that internal salt contamination and considerable reinforcing bar corrosion occur. A large fraction of the cracks in bridge decks are transverse cracks, occurring along the transverse steel and extending down to it. The durability of the epoxy coating for those bars is of concern and at this time unknown. The coating can be damaged by crack movements caused by structural flexing. Sealing those cracks when they form is desirable preventive maintenance. Methods of minimizing cracking and the effectiveness of crack sealing are examined in this report.

Flexibility of the superstructure contributes to bridge deck cracking. Structures with significant traffic-induced deflection reversals, or high traffic-induced vibration amplitudes and frequencies, are more likely than other structures to crack or have their existing cracks lengthen and deepen. More flexible structures result not only in larger deflections but also in vibrations with larger amplitudes at essentially the same frequency.

The cost effectiveness of each of these five protection strategies, and of double protection strategies resulting from their combinations, was evaluated based on lifetime costs that included possible resurfacing needs. The least expensive strategy was increased depth of cover to the top bar, and the next least expensive was epoxy coating of the top bars. Bridge decks doubly protected by epoxy coating of both top and bottom steel mats cost only 4 percent more than decks with only the top mat coated.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

BACKGROUND

Premature deterioration of concrete bridge decks is of major concern to the nation's highway agencies. Deterioration makes the ride quality of decks significantly worse. Necessary repairs or reconstruction causes serious inconveniences to the travelling public and problems for the highway agency. Deck deterioration, depending on its magnitude and location, can also reduce a bridge's load carrying capacity.

Bridge deck deterioration is caused primarily by corrosion of embedded reinforcing steel as a result of winter salt applications and moisture. The expansion accompanying corrosion causes internal rupturing of the concrete and results finally in spalls on the concrete's surface. Corrosion also causes losses in steel area and loss of bond between the steel and concrete. Bridge deck deterioration has become a major concern because design practices of the 1960s and early 1970s seldom considered protection of the deck against salt infiltration. Further, the use of salt to achieve bare road conditions has increased as the nation's highway network has increased.

Bridge decks are exposed to large changes in temperature and moisture, and are subjected to direct impact and repeated loading by traffic. Those conditions, combined with salt applications, create a very severe environment for the concrete. That environment, in addition to corrosion of the bars, can cause premature deterioration of the concrete surface and therefore also influences the serviceability of the structure.

Over the past two decades, many protective strategies have been developed to combat deterioration of concrete bridge decks. The concept of preventing bar corrosion has generally determined the protective strategies for new deck construction. Such strategies have included epoxy coating and galvanizing bars; overlaying decks with low permeability concretes containing latex particles or waxbeads, or reducing water/cement ratios and increasing densities; overlaying or impregnating decks with polymers; increasing the bar cover thickness; waterproofing with interlayer membranes having asphaltic concrete overlays; using sealers with or without overlays; and using cathodic protection. Subsequent evaluations of the constructibility, cost-effectiveness, and performance of early experimental installations of those systems has resulted in several being adopted nationwide.

Presently, commonly used systems are epoxy-coated bars in the deck, low-slump dense concrete overlays, latex-modifed concrete overlays, interlayer waterproofing membranes with asphaltic concrete overlays, and increased bar cover thickness.

PROBLEM STATEMENT

This research is aimed at providing definitive information about the likely service life of protected bridge decks. Generally decks for which protection was part of the original construction are performing better than unprotected decks. In the latter case some decks have required rehabilitation after only a few years of service. Highway agencies, however, question the likely service life of protected decks. They want decks with at least 50 years of corrosion-free service life.

One concern about concrete protective strategies is their resistance to chloride penetration and, in particular, the effects of cracking on permeability. Cracking is especially severe with some latex-modified and low-slump dense concrete overlays. Further, traffic and the environment cause cracks to extend and multiply with time. Even in the absence of cracking, chlorides penetrate with time to the bar under latex-modified and low-slump dense concrete overlays. Chloride contamination is inevitable, and the amount of time until corrosion begins depends only on the intensity of the salt applications. In addition, there are concerns about the overlays stripping from the deck. Some overlays have shown evidence of debonding and stripping under the action of traffic.

For decks protected with asphaltic concrete overlays paved over waterproofing membranes, the main concern is debonding and stripping. Such action has been frequently reported after 15 or fewer years of service.

The use of epoxy coating for deck bars eliminates some of the problems inherent in overlay strategies. However, the coating is susceptible to damage during fabrication and handling, and perhaps even during service. That susceptibility creates concerns about possible corrosion at unrepaired or undetected damage locations. Highway agencies question the long-term effectiveness of epoxy-coating in a chlorine-contaminated concrete environment and the long-term adhesion of the coating to the bar, especially in areas where a touch-up coating is applied over damage.

RESEARCH APPROACH

This research was divided into two phases. The objective of the first phase, the current study, was to evaluate the performance of the most commonly used protective strategies for any evidence that a 50-year bridge deck service life might not be attainable. The objective of the second phase, a future study, was to develop tentative guidelines for the design and construction of reinforced concrete bridge decks with a service life of 50 years or more.

For this current phase, information was first gathered on the use of salt on bridge decks, on protective strategies for new deck construction, and on the overall performance of those strategies. That action was accomplished by sending an appropriate questionnaire to the transportation departments of all 50 states and the District of Columbia.

Next, detailed performance information was collected for the five most commonly used protective strategies: (1) increased depth of cover over the top reinforcing steel (3 in. or more), (2) low-slump dense concrete overlays, (3) latex-modified concrete overlays, (4) interlayer waterproofing membranes, and (5) epoxy-coated reinforcement. Quantitative information on protected bridges with 5 to 15 years of service was developed through a comprehensive literature review. In addition, as a follow-up to the questionnaire, four state transportation departments were interviewed, several bridges in their jurisdictions were examined qualitatively, and the particular problems with the use and performance of the five selected protective strategies were reviewed. Those meetings laid the groundwork for possible participation of those transportation agencies in the second phase of the research.

The performance information collected was then analyzed for any evidence that a 50-year bridge deck service life might not be attainable and that use of a particular protective strategy or combination of strategies would be more cost effective than another. The research defined performance as a function of (1) the durability of the bridge decks and their protective components, independent of evidence of bar corrosion; (2) the bar corrosion prevention effectiveness of the given strategy; and (3) the degree of concrete cracking in the decks. Based on that performance information, issues needing additional examination were identified, and recommendations were made for tentative guidelines for the design and construction of bridge decks with service lives of 50 years or more.

Finally, a plan was developed for the second phase of the research. That plan proposed in-depth examinations of the significance of various factors affecting bridge deck corrosion-free life and serviceability, elaboration of the tentative guidelines for the design and construction of bridge decks, and field studies of a limited number of representative, protective strategies to verify, modify, and/or improve those tentative guidelines.

CHAPTER TWO

FINDINGS

This chapter summarizes the important findings from this study on (1) durability of bridge decks and their protective components, (2) corrosion prevention effectiveness of the five protective strategies studied, (3) cracking in bridge decks, and (4) cost-effectiveness of the five strategies and their combinations. In this discussion the effective service period of a deck is the number of years until deterioration affects 5 percent of the deck area. Appendixes A through G contain the detailed information from which these findings were derived.

DURABILITY OF BRIDGE DECKS AND THEIR PROTECTIVE COMPONENTS

Bridge decks and their corrosion protection components are exposed to deicing salts, freeze-thaw action, and the direct impact of traffic. Concrete scaling, surface wear, and debonding or stripping of the protective overlay can result and can, in turn, influence the serviceability and service life of the deck. This section summarizes findings on durability as evidenced by scaling, surface wear, and debonding or stripping of overlays. More detailed information is presented in Appendixes A, F, and G.

Bridge Decks Without Overlays

Decks in this category utilize conventional structural concrete only and are protected by epoxy coating on their bars or by the provision of at least 3 in. of concrete cover over uncoated bars. The survey, Appendix F, indicated that only one transportation department was dissatisfied with the scaling resistance of air-entrained concrete. However, scaling in air-entrained concrete has been documented in installations and environments nationwide (*1, 2, 3, 4*). Also, in the site visits (Appen. G) bridge deck scaling was observed in localized deck areas representative of certain concrete batches or certain environments (i.e., gutter areas). Regardless of the number of freeze-thaw cycles, scaling is likely in cold environments with high rates of snowfall. In those environments scaling is evident in the roadside surface of traffic barriers (Appen. G).

Traffic polishes the concrete's fine aggregate and causes a loss of skid resistance. Traffic also wears away the mortar (*1, 2, 3*) and eliminates transverse traction grooves built into the surface. In the extreme, wear results in surface rutting. Rut depths of about ½ in. can be expected in 15 to 20 years in urban areas where studded tires are permitted (Appen. G).

Bridge Decks with Concrete Overlays

The survey, Appendix F, showed that two transportation departments were dissatisfied with the scaling resistance of low-slump dense concrete and that two departments were dissatisfied with the scaling resistance of latex-modified concrete. However, the literature review showed that for latex-modified concrete scaling has been considerably less than for low-slump dense concrete. In low-slump dense concrete, scaling of up to 4 percent of the deck area has been found after 6 years of serve (*5*). In the same environment the scaling of latex-modified concrete has been limited to 1 percent after 9 years of service.

Evidence on the skid resistance of these special concretes conflicts. Higher skid numbers have been obtained on latex-modified concrete than on low-slump dense concrete (*6*). An average skid number of 48, corresponding to 2 to 6 years of service, was obtained for latex-modified concrete, while an average number of 39, corresponding to 2 to 3 years of service, was obtained for low-slump dense concrete. Other work, however, has shown the same skid resistance for both concretes (*7*). Wear for both concretes under traffic has been similar to that for conventional concrete. Traction grooves, initially $\frac{3}{16}$-in. deep, have totally disappeared in about 7 years under a 10,000 average daily traffic volume and the use of studded tires (Appen. G). One transportation department is presently dissatisfied with the wear of low-slump dense concrete and two are dissatisfied with the wear of latex-modified concrete.

Most concrete overlays have demonstrated satisfactory integrity with the underlying decks. Debonded areas, if any, have been less than 1 percent of the decks' areas after up to 7 years of service (*5, 8*). However, in some cases large areas of the overlay have debonded with little correlation to the age of the overlay. For example, in one case a low-slump dense concrete overlay debonded over 35 percent of the deck after only about 5 years of service (*8*). Debonding has been attributed to inadequate texturing of the substrate and methods of applying bonding agents that result in no bond or insufficient bond when construction is complete. The survey of Appendix F indicated that in two-course new construction decks, one department is dissatisfied with the bonding of low-slump dense concretes and four departments are dissatisfied with the bonding of latex-modified concrete. For overlays applied on existing decks, six departments are dissatisfied with the performance of low-slump dense concrete and eight are dissatisfied with latex-modified concrete. More departments are dissatisfied with these overlays for existing decks than new decks because nationwide these overlays are more common for existing decks than for new decks.

Bridge Decks with Asphaltic Concrete/Membrane Systems

Debonding and stripping of asphaltic concrete overlays has been a major problem for some transportation departments. Depending on the severity of the traffic and environment, some systems have required removal and replacement in 10 years or less (*5*, Appen. G). When water accumulates above the membrane, it weakens the bottom portion of the asphaltic concrete (*5, 9*). Eight transportation departments reported stripping of asphaltic concrete from two-course new construction decks (Appen. F).

For asphaltic concrete subjected to average daily traffic volumes exceeding 10,000, skid numbers below 35 may be expected in about 15 years (Appen. A). Excessive wear of the surface in wheeltracks, especially in the presence of studded tires, combined with consolidation or lateral movement of the pavement, causes rutting in the asphaltic concrete. Nine transportation departments are dissatisfied with the wear of asphaltic concrete overlays (Appen. F).

CORROSION PREVENTION CHARACTERISTICS OF PROTECTIVE STRATEGIES

Major factors determining the performance of concrete bridge decks are bar corrosion and subsequent concrete cracking and spalling. This section presents findings on the ability of different protective strategies to prevent corrosion. Detailed analysis and information are presented in Appendix B.

Bridge Decks with 3.5-in. Bar Covers

The effective service period (i.e., the number of years until deterioration affects 5 percent of the deck area) may be 50 years or more when salt exposure is less than 5 tons per lane-mile per year. For higher salt applications, the water/cement ratio of the concrete determines the service life. For salt exposures greater than 10 tons per lane-mile per year, specified maximum water/cement ratios must be 0.42 or less if the effective service life is to be 50 years. When salt applications reach 30 to 45 tons per lane-mile per year, the effective service period even for a specified water/cement ratio of 0.42 may be 10 to 15 years only. This study showed that construction procedures impose a tolerance of 0.03 on the water/cement ratio. Thus, the limiting ratio for permeability considerations should be the specified value plus 0.03 (Appen. B).

Bridge Decks with Concrete Overlays

The survey showed that three transportation departments are dissatisfied with the chloride proofing abilities of low-slump dense concrete overlays and six are dissatisfied with the chloride proofing abilities of latex-modified concrete overlays. The dissatisfaction is a result of the permeability of the overlays. For low-slump dense concrete overlays the specified water/cement ratio is 0.32. However the chloride permeability of field installations is similar to that of a concrete with a water/cement ratio of 0.40 to 0.45. The permeability is higher because the stiffer, low-slump mix causes construction difficulties. The chloride permeability of field installations of latex-modified concrete is about the same as the permeability of latex-modified concrete made in the laboratory. A 1.5-in. thick, latex-modified concrete provides the same chloride permeability as a 2-in. thick, low-slump dense concrete representing a water/cement ratio of 0.40.

The effective service period of a protective system consisting of 1.5 in. of latex-modified concrete, or 2 in. of low-slump dense concrete representing a water/cement ratio of 0.40, placed on a deck with a specified water/cement ratio of 0.45 and a bar cover depth of 1.5 in., is about 50 years only if annual salt applications do not exceed 12 tons per lane-mile.

Bridge Decks with Asphaltic Concrete/Membrane Systems

The field performance of preformed and thermoplastic, applied-in-place membranes (*5, 9, 10, 11*) indicates that such membranes are effective in preventing chloride intrusion. A statistical analysis of the 10-year performance of membranes (*11*) has shown that the rate of increase in chloride contamination at a depth of 1 to 2 in. into the concrete, the typical location for the deck's reinforcing bars, is less than 0.004 lb/cu yd per year. For that increase it takes more than 50 years before the concrete surrounding the bar is contaminated to the corrosion threshold level of 1.50 lb/cu yd. However, membrane deterioration can be expected after only 15 years of service because of the repeated stresses caused by traffic. Membranes also suffer from age embrittlement. Three states (Appen. F) were not satisfied with the chloride proofing abilities of interlayer membranes.

Bridge Decks with Epoxy-Coated Bars

Field measurements of the electrical resistance in bridge decks between epoxy-coated top mats and uncoated bottom mats have shown that electrical contact may exist between the two mats (*12*). As a consequence, a coated top mat in chloride-contaminated concrete can corrode at local coating breaks or pinholes in the original coating. Such breaks can occur during transportation of the bars, construction, or service. The maximum area allowable by FHWA regulations (*13*) for unrepaired coating breaks is 0.25 percent when only the top mat is coated and is 2 percent when both the top and bottom mats are coated. Actual measurements on structures before placing the concrete (*12*) have shown coating break areas considerably less than 2 percent.

Researchers have estimated that, at the same chloride exposure, corrosion may consume the same amount of top mat epoxy-coated steel in 12 years as it would consume top mat uncoated steel in one year (*12*). In other words, in salt environments, the life expectancy of bridge decks is increased considerably by epoxy-coating the top mat steel. The amount of metal consumed in the top mat is reduced 46 times by epoxy-coating both the top and bottom steel mats (*12*).

Pull-out bond tests have indicated that the bond strength for certain epoxy coatings, with film thicknesses 10 mils or less, is about 6 percent less than the bond strength of uncoated bars (*14*). That reduction is within the range of variability normal in bond tests. Flexural bond tests have shown that for bar development lengths greater than 12 in., the bond strength for an uncoated bar is 15 percent greater than the bond strength for an epoxy-coated bar (*15*). Flexural fatigue bond tests in the working stress range have shown that slip behaviors for uncoated and coated bars are essentially similar (*15*).

CRACKING IN CONCRETE BRIDGE DECKS

Concrete cracking affects both the durability and the corrosion prevention characteristics of bridge deck protective strategies by allowing moisture and salt to penetrate into the concrete. This section presents findings on the nature and magnitude of cracking in concrete bridge decks and the effectiveness of different procedures for crack sealing. Appendixes C and D contain more detailed information.

Transverse cracking is the most frequent cracking in bare decks (*1, 16, 17, 18, 19,* Appen. G). As much as 100 ft of transverse cracking per 1,000 sq ft of deck area has been observed in decks with bar covers deeper than 3 in. (*17*). The type of cracking found in two-course new construction bridge decks with low-slump dense or latex-modified concrete is generally a combination of random and transverse cracking. The amount of cracking in concrete overlays can exceed 100 lin-ft per 1,000 sq ft of deck area (*5,8*).

Many factors can cause cracking during a deck's construction and service life. The more important factors are (1) shrinking of plastic concrete, caused by evaporation of surface water; (2) flexure in plastic concrete, caused by flexure of the formwork over the supports of continuous structures; (3) settlement of finished plastic concrete around the uppermost reinforcing steel; (4) shrinkage of hardened concrete because of loss of moisture; (5) long-term flexure of continuous spans under service loads; and (6) traffic-induced, repeated deflections and vibrations.

Cracks in concrete decks are usually sealed by scrubbing the mortar portion of a concrete mix onto the cracks or by feeding a low-viscosity, low-modulus polymer into the cracks using gravity, and/or injection methods. Tests have demonstrated that scrubbing the mortar portion of concrete onto cracks does not seal cracks well against chloride penetration. However, use of a polymer penetrating sealer is effective (*20*).

Table 1. Present values of 50-year lifetime costs for bridge decks constructed with various protective strategies.

	Bridge Deck Protection Alternative	Cost per sq. ft. of Deck Area (typical 1986 cost, $)
Single Protection	Cover thickness of 3.5 inches	13.93
	Epoxy-coated top mat	14.35
	Latex-modified or low-slump concrete overlay	16.35
	Interlayer membrane	15.98
Double Protection	Epoxy-coated top and bottom mats	14.95
	Epoxy-coated top mat & latex or low-slump concrete overlay	16.95
	Epoxy-coated top mat & interlayer membrane	16.58

COST-EFFECTIVENESS OF PROTECTIVE STRATEGIES

The cost-effectiveness of the five protective strategies and that of double protection strategies resulting from their combination were evaluated based on lifetime costs. It was assumed that each strategy would prevent corrosion for 50 years, but that major maintenance in the form of deck resurfacing would be required. Appendix E provides detailed information.

Given in Table 1 are present values of 50-year lifetime costs for bridge decks constructed with various protective strategies. For single protection decks, provision of a concrete cover over the reinforcing bars equal to 3.5 in. is the least expensive strategy followed, in turn, by the use of an epoxy-coated top mat, an interlayer membrane, and a low permeability concrete overlay (low-slump dense or latex-modified concrete). For double protection decks, provision of epoxy-coated top and bottom mats is the least expensive strategy followed, in turn, by protection with an epoxy-coated top mat used in conjunction with an interlayer membrane, and an epoxy-coated top mat used in conjunction with a low permeability concrete overlay. Interestingly, the cost of epoxy coating both mats is lower than the cost of protecting the deck with a low permeability concrete overlay or an asphaltic concrete and interlayer membrane only.

CHAPTER THREE

INTERPRETATION, APPRAISAL, AND APPLICATION

DURABILITY OF BRIDGE DECKS AND THEIR PROTECTIVE COMPONENTS

In this section procedures are suggested for providing more durable bridge decks through minimization of concrete freeze-thaw scaling, surface skid and wear, and overlay debonding and stripping. More detailed information is presented in Appendixes A, F, and G.

Bridge Decks Without Overlays

Presently at least 41 state transportation departments use epoxy-coated bars for conventional structural concrete decks built without overlays. Air entrainment of the concrete is the primary means for controlling freeze-thaw scaling. A previous NCHRP survey (*21*) showed that 1981 specifications, which were only slightly different from those of 1975, required an average air content of 5.5 percent, with a lower limit of 4 percent and an upper limit of 7 percent. Further, the specified air content limits had not changed significantly between 1970 and 1975 (Fig. A-3). The water/cement ratio for the concrete is another factor determining capillary void structure and freeze-thaw resistance. Presently 80 percent of the transportation departments specify a water/cement ratio of 0.45 or less for bridge deck concrete (Appen. F).

An air content of at least 4 percent is required for good concrete durability (Fig. A-4). Newlon and Walker (*4*) have suggested that acceptable freeze-thaw resistance can be expected from bridge deck concretes if they meet certain requirements on air void spacing, water/cement ratio, and water absorption. The air void spacing factor of the hardened concrete should be less than 0.008 in. Fresh concrete with air contents less than 4.5 percent cannot ensure such air void spacings (*4*). The calculated value for the water/cement ratio during construction should be limited to 0.45 and water reducers used in the mix. Reducers ensure indirectly a low water/cement ratio, regardless of the calculated value. Finally the concrete's water absorption should be limited to 4.5 percent. The absorption reflects the combined effects of the degree of consolidation achieved during construction and the water/cement ratio (*4*).

Elements that adversely influence the freeze-thaw durability of concrete include (1) a surface that has been sprinkled or overworked during construction (such actions alter the surface's air void system and decrease its strength); (2) an inadequate surface drainage system that causes ponding and ultimately saturation of the air-entrained concrete, especially for decks where deicing salts are used; (3) delays between concrete placement and curing (*4*); (4) an insufficient period of air drying before first application of deicing salts (*21*); and (5) treatment of the concrete surface with sealant, the effectiveness of which has not been demonstrated and which may accelerate deterioration (*4*).

For sufficient skid resistance a concrete surface needs a fine texture that provides adequate contact with a tire. The higher the proportion of fine aggregate in the mix the better the skid resistance (*22*). Some fine aggregate polish under traffic action to a degree that makes the surface slippery. One laboratory test that evaluates the polish susceptibility of aggregate is the acid insoluble residue test (ASTM D3042).

To provide coarse texture and prevent hydroplaning on wet surfaces the concrete deck should be tined, broomed, or turf-dragged when plastic. Generally tining provides $\frac{3}{16}$-in.-deep transverse grooves in the concrete. Alternatively, coarse texture can be built into the hardened concrete by saw cutting. When the grooves wear away under traffic, the coarse aggregate in the mix provides the pavement's skid resistance. Crushed aggregate provides better texture than uncrushed aggregate. Wear resistance can be increased by decreasing a concrete's water/cement ratio, increasing its cement factor, and employing proper finishing and curing procedures (*22*). Extensive wear results in rutting in wheeltracks. Extensive rutting may be repaired by partial patching using polymer materials that can develop sufficient bond in ultrathin layers.

Bridge Decks with Concrete Overlays

Low-slump dense concretes employ air contents of 6.5 ± 1.0 percent (*21*) and water/cement ratios of about 0.32. However, up to ten times the normal dosage of air entraining agents is sometimes needed to produce the desired air content (*21*). Thus, the effectiveness of the air entraining agent for low-slump dense concrete should be examined prior to its field use (*21*).

Latex-modified concretes do not entrain air, but do develop air voids with a typical upper limit of 6 percent. The concrete has a water/cement ratio of about 0.32. The air void spacing factor is generally greater than 0.008 in., but that factor is not significant for freeze-thaw durability (*23, 24*) because emulsion prevents water penetration and the void structure becomes unimportant (*23*).

As is the case for conventional concrete, the fine aggregate in the latex-modified mix provides the fine texture and the skid resistant qualities of the concrete overlay. The skid resistance of latex-modified concrete is better than that of conventional concrete, because latex-modified concrete has a higher proportion of fine aggregate. The coarse texture of the overlay's surface is usually provided by traction grooves in the same manner as for conventional concrete. Low-slump dense concrete, because of its low water/cement ratio and high cement factor, has a higher potential to resist wear under traffic than the other types of concrete. However, the field performance of low-slump dense concrete does not necessarily support this. Overworking or sprinkling the surface during the construction is likely because of the stiff nature of the mix, and that results in concrete with a lower surface strength.

Concrete overlays must have sufficient initial bond to prevent debonding during their service life. Repeated live loading, thermal cycling, and wetting and drying decrease the shear strength at the bonded interface. Overlays need an initial shear bond strength of at least 500 psi. One laboratory method for determining shear strengths from core samples obtained in the field is Iowa's Test Method Number 406. The appropriate procedure for bonding concrete overlays for two-course new construction is generally to sand blast substrate that has been broomed to a rough texture during the first stage of construction, and to then apply a bonding agent on a dry substrate for low-slump dense concrete and on a wetted substrate for latex-modified concrete. If the deck is rained on after sand blasting but before overlaying, blasting should be repeated when the deck is dry. It is also important that the bonding agent does not dry out when the overlay is applied. If the second stage construction is delayed and the deck has been open to traffic even for a short time, scarifying to a depth of $\frac{1}{4}$ in. prior to sand blasting may be required. However, some agencies require scarification regardless of when the overlay is applied. To improve a bond some agencies scrub a mortar portion of the overlay mix onto the deck. Other agencies require a grout specially made for that purpose. Polymer materials have also been used as bonding agents.

Bridge Decks with Asphaltic Concrete/Membrane Systems

The primary cause of asphaltic concrete overlays stripping off decks is the accumulation of water above the membrane in the bottom portion of the asphaltic concrete. This phenomenon, combined with freezing and thawing and repeated hydraulic pressure from traffic, weakens the bottom layer of the asphaltic concrete and the bond between the asphalt and the membrane. It can also strip the asphalt binder off the aggregate in that layer, resulting in no integrity among the mixture elements or adhesion between them and the membrane. Another factor causing debonding and stripping is the formation of blisters under preformed sheet membranes.

The properties of asphaltic concrete can be important for preventing stripping caused by water accumulation. The use of conventional paving mixtures on bridge decks may not be feasible. High density and low air void content are desirable, but these properties must be balanced with the potential for loss of stability under traffic and high temperature. Special designs may be needed that include high quality aggregate, stiffer binders, and anti-stripping agents. The construction phase should employ effective compaction to satisfy design density. The asphaltic concrete must be well bonded to the membrane, but the amount of the bonding element, which may be the membrane itself, must be optimized to prevent horizontal slippage. Weak seams should be avoided in asphaltic concrete construction joints by using proper rolling patterns and maintaining hot longitudinal joints between paving lanes. Adequate surface drainage should be provided to prevent surface ponding. Drainage from the surface of the membrane may be possible by installing vertical pipes through the deck. Blistering under preformed membranes can be prevented by providing sufficient dead weight in the form of 2 to 3 in. of asphaltic concrete overlay (*25*). Blistering may also be prevented by applying a perforated sheet of bituminous felt or a 1-in.-thick asphalt base course on the deck as a venting layer before applying the membrane (*25*).

The skid resistant qualities of the asphaltic concrete depend on the nature of the aggregate particles in the mix, which provides the fine texture, as well as the aggregate's gradation, which provides the coarse texture (*22*). A polish- and wear-resistant, crushed aggregate and sufficient coarse aggregate for an adequate void structure ensure good skid resistance and allow surface water to drain, reducing the potential for hydroplaning. The wearing course needs a low void content, but that also results in a surface with too little skid resistance. Further, a dense mixture is more likely to deform and rut under traffic. Thus, the design of asphaltic concrete for bridge decks requires a compromise among several factors.

For the repair of worn asphaltic concrete surfaces, surface treatments such as chip seals and slurry seals have been tried with limited success. Conventional asphalt binders for these techniques are being replaced by polymer modified asphalts. In addition, modified slurry seals are being used to fill wheeltrack ruts both with and without milling of the old pavement prior to repair.

Guidelines for the Design and Construction of Durable Bridge Decks

The following checklist summarizes the issues that should be considered in the design and construction of bridge decks. Neglect of these issues adversely affects a deck's durability and may require its premature resurfacing

1. *To minimize concrete freeze-thaw scaling, consider:* air voids and air entrainment, water/cement ratio, water reducers, water absorption of final products, surface drainage, curing procedure and season, and sealing.
2. *To minimize concrete wear and increase skid resistance, consider:* water/cement ratio, cement factor, proportion of fine aggregate, polish and wear resistance of the aggregate, nature of the coarse aggregate (crushed, uncrushed), and depth of traction grooves.
3. *To minimize debonding and stripping of concrete overlays, consider:* methods of substrate preparation (scarifying, sandblasting, wetting), nature of the bonding agents, and minimum required initial bond.
4. *To minimize asphaltic concrete wear and increase skid resistance, consider:* polish and wear resistance of the aggregate, nature of the coarse aggregate (crushed, uncrushed), and proportion of coarse aggregate.
5. *To minimize debonding and stripping of asphaltic concrete, consider:* air voids and density of the asphaltic concrete, binder anti-stripping agents, venting layers (preformed sheet membrane), thickness of the asphalt concrete (preformed sheet membrane), asphaltic concrete/membrane bond, field compaction intensity and pattern, drainage from above the membrane, surface drainage, and waterproof construction joints in asphalt concrete.

CORROSION PREVENTION CHARACTERISTICS OF PROTECTIVE STRATEGIES

This section describes the effects on corrosion-free service periods of variations in the design properties of the different protective strategies.

Bridge Decks with 3.5 in. of Bar Cover

Shown in Table 2 are the likely number of years between construction and maximum acceptable corrosion (5 percent of deck area) for decks designed with cover depths of 3.5 in. This table is developed in Appendix B and is based on FHWA laboratory work and the evaluation of bridge deck performance reported here. For the preparation of Table 2, it was assumed that 5 percent of the deck area would have covers less than the design target depth (3.5 in.) minus 0.65 in. The amount of time between corrosion and delamination can be assumed to be 3 years (*27*).

For Table 2, construction procedures were assumed to cause variations of 0.03 in the water/cement ratio (*28*). To achieve a 50-year or more effective service period, designs must consider the severity of salt application, the water/cement ratio, and the cover thickness. As discussed in Appendix B similar tables can be constructed for different cover thicknesses.

Bridge Decks with Concrete Overlays

Table 3 gives the likely number of years between construction and maximum acceptable corrosion (5 percent of deck area) for decks designed with 2-in., low-slump dense or 1.5-in., latex-

Table 2. Estimated years to maximum acceptable corrosion of reinforcing steel in bridge decks with design target bar depth of 3.5 in.

a. Based on 620 lbs. of salt per application per lane-mile

W/C Maximum Design	W/C[1] Adjusted for Construction	Number of Annual Salt Applications							
		5	10	15	20	30	50	100	150
0.50	0.53	99	49	33	25	16	10	5	3
0.49	0.52	108	54	36	27	18	11	5	4
0.48	0.51	117	59	39	29	20	12	6	4
0.47	0.50	128	64	43	32	21	13	6	4
0.46	0.49	151	75	50	38	25	15	8	5
0.45	0.48	177	89	59	44	30	18	9	6
0.44	0.47	208	104	69	52	35	21	10	7
0.43	0.46	247	123	82	62	41	25	12	8
0.42	0.45	292	146	97	73	49	29	15	10

b. Based on 350 lbs. of salt per application per lane-mile

W/C Maximum Design	W/C[1] Adjusted for Construction	Number of Annual Salt Applications							
		5	10	15	20	30	50	100	150
0.50	0.53	175	87	58	44	29	17	9	6
0.49	0.52	191	95	64	48	32	19	10	6
0.48	0.51	208	104	69	52	35	21	10	7
0.47	0.50	227	114	76	57	38	23	11	8
0.46	0.49	267	134	89	67	45	27	13	9
0.45	0.48	314	157	105	79	52	31	16	10
0.44	0.47	368	184	123	92	61	37	18	12
0.43	0.46	438	219	146	110	73	44	22	15
0.42	0.45	517	259	172	129	86	52	26	17

[1] Considering a tolerance of 0.03.

(shaded box) Years to corrosion ≥ 47 years.

Table 3. Estimated years to maximum acceptable corrosion of reinforcing steel in bridge decks overlaid with low-slump dense and latex-modified concrete.

a. Based on 620 lbs. of salt per application per lane-mile

Case	Number of Annual Salt Applications					
	30	40	50	70	100	150
I: Overlay: 2" LSDC or 1.5" LMC; Bare Deck: 1.5" Target Cover & W/C = 0.45	72	54	43	31	22	14
II: Overlay: 2.5" LSDC or 1.75" LMC; Bare Deck: 2.0" Target Cover & W/C = 0.45	170	128	102	73	51	34

b. Based on 350 lbs. of salt per application per lane-mile

Case	Number of Annual Salt Applications					
	30	40	50	70	100	150
I: Overlay: 2" LSDC or 1.5" LMC; Bare Deck: 1.5" Target Cover & W/C = 0.45	128	96	77	55	38	26
II: Overlay: 2.5" LSDC or 1.75" LMC; Bare Deck: 2.0" Target Cover & W/C = 0.45	301	226	181	129	90	60

(shaded box) Years to corrosion ≥ 47 years.
LSDC Low-Slump Dense Concrete
LMC Latex-Modified Concrete

modified concrete overlays. Those overlays are assumed to be placed on bare decks designed with a 1.5-in. cover thickness and a water/cement ratio of 0.45. The table also gives the effective service life for 2.5-in., low-slump dense or 1.75-in., latex-modified concrete placed on decks designed with a 2-in. cover thickness and a 0.45 water/cement ratio. The amount of time until delaminations appear can be determined by adding 3 years to the amount of time shown for corrosion (*27*). Table 3 assumes that the chloride permeability of field installations of low-slump dense concrete is the same as the permeability of conventional concrete with a 0.40 water/cement ratio (*29*). Some studies, however, indicate that that figure can be as high as 0.45 because of insufficient consolidation and curing.

To calculate the effective service period, it was assumed that the system comprised conventional concrete only with a water/cement ratio similar to that for the first stage construction but with a cover thickness larger than that of the first stage construction, so that the same chloride permeability could be expected. Table 3 was then prepared employing the same procedure as that used for bare decks. With that procedure, and as shown in Appendix B, similar tables can be prepared reflecting the effects of different variables such as overlay thickness, expected overlay field permeability, bar cover thickness, and the water/cement ratio for the first stage construction.

The effective service period of a concrete overlay is also a function of its durability and is generally less than 50 years. At the end of that period the overlay must be removed and/or resurfaced with an overlay of the same type. Consideration should be given to the feasibility of removing the concrete overlay or the likelihood of accumulating dead loads, if the initial overlay is not totally removed.

Bridge Decks with Asphaltic Concrete/Membrane Systems

Although the chloride-proofing abilities of some membrane systems seem to satisfy the 50-year service life criteria (*11*), the actual life is governed by deterioration of the asphalt wearing course. The life of that course is generally 10 to 15 years, depending on weathering and exposure to traffic. Factors such as overlay stripping, wearing and rutting, or lack of skid resistance can require removal of the overlay and/or resurfacing. When the asphaltic concrete is at least 2 in. thick and has deteriorated prematurely only on the surface, the upper portion of the asphaltic concrete can be removed and replaced while keeping the membrane intact. However, in such cases, consideration should be given to the membrane condition because membranes also age and deteriorate under traffic.

When the membrane is removed along with the asphaltic concrete, conventional removal procedures, such as cold milling and scarifying, may result in irregularities in and disturbances to the concrete surface due to unexpected variations in the thickness of the asphaltic concrete. New membranes cannot be applied to these irregularities. This problem may be solved by first applying a ½-to 1-in. thick leveling asphaltic concrete course on the disturbed concrete deck followed by the membrane and the wearing course.

Aside from design features, the permeability of the membranes depends greatly on construction practices and quality. Damage to a membrane under the asphaltic concrete paver and construction traffic has frequently been reported. Design features, such as protection boards, can prevent this damage as well as possible puncturing of the membrane under traffic by aggregate particles from the asphaltic concrete. The most common protection board is a ⅛-in. thick, asphalt-impregnated sheet (*25*). Sealing the edges of membranes at curb locations is also important because of relatively higher chloride intrusion in those areas.

Bridge Decks with Epoxy-Coated Bars

In some older bridge decks designed with only 1.5 in. of cover over uncoated steel, corrosion has induced deterioration in as little as 5 years. Epoxy coating the top steel, combined with limits of 0.45 on the water/cement ratio and 2 ½ in. of cover over that steel, promises to provide 50 years of corrosion-free life even in severe chloride environments. That promise can be further assured by epoxy-coating both the top and bottom steel.

FHWA's recommendations for maximizing the service life of an epoxy-coated bar system are to leave no more than 0.25 percent of the damaged areas in the epoxy film unrepaired when the top reinforcing mat only is coated and no more than 2 percent unrepaired when both the top and bottom mats are coated (*13*). Because in some bars the amount of damage existing before placing the concrete can exceed 0.25 percent of the surface area (*12*), and because breaks in the coating may become potential sites for bar pitting and subsequent bar fatigue failure, epoxy coating of the bottom mat seems prudent. Further, that coating adds only 4 percent to construction costs (Appen. E).

The long-term durability of epoxy coating in a chloride-contaminated concrete remains a concern. Two major elements that may contribute to deterioration of the coating and corrosion of a bar are the presence of holidays (pinholes not detectable by the unaided eye) in the coating and weak adhesion of the coating to the bar. In this regard, the pinhole and adhesion requirements specified by AASHTO M284 or ASTM 3963 are especially important. These requirements limit the average number of pinholes in the coated bar to 2 per lin-ft and necessitate evaluating the adhesion of the coating by bending coated bars around a mandrel. Of concern also is the adhesion of touch-up epoxy coating placed over damaged areas. There are indications that epoxy does not bond well to epoxy, thus increasing the possibility of corrosion at damaged areas.

Based on the results of flexural and fatigue tests of bond between epoxy-coated bars and concrete, a basic development length modification factor of 1.15 should be used for epoxy-coated bars in order to provide comparable performance with uncoated bars (*15*). The pull-out bond strength for coated bars should be determined using concrete prisms and equal to the smaller of the stresses corresponding to a free end slip of 0.002 in. or a loaded end slip of 0.010 in. The mean bond strength for coated bars should be at least 80 percent of the mean bond strength for uncoated bars (*14, 15*).

Guidelines for the Design and Construction of Bridge Decks with Bars Resistant to Corrosion

The following checklist summarizes the issues that should be considered in the design and construction of bridge decks in order to provide at least 50 years of corrosion-free performance.

1. *For bare decks and decks overlaid with low-slump dense or latex-modified concrete, consider:* extent of salt application, thickness of cover for the bare deck or first stage construction, water/cement ratio for the bare deck or first stage construction, thickness of the concrete overlay, consolidation and curing of the concrete, degree to which the concrete overlay in the field is permeable (previous experience), deterioration in the concrete, and feasibility of removing and/or resurfacing the concrete.
2. *For decks with asphaltic concrete/membrane systems, consider:* traffic intensity, thickness of the asphaltic concrete, additional protection for the membrane, deterioration of the asphaltic concrete and its membrane, age of the membrane, and methods of removing the asphaltic concrete and its membrane.
3. *For decks with epoxy-coated bars, consider:* extent of damage and breaks in the coating, extent of pinholes in the coating, adhesion of the epoxy coating to the bar, and bond between the coated bar and the concrete (structural requirement).

CRACKING IN CONCRETE BRIDGE DECKS

This section discusses cracking in concrete decks and suggests methods for minimizing it. Cracking in both bare and concrete overlaid decks can be extensive enough to accelerate chloride contamination. Further, flexing at cracks has the potential for wearing away the coating on bars at lugs. Transverse cracks comprise a significant proportion of the total cracks in bare decks. They generally occur along the uppermost bars and extend down to those bars. The findings of this study suggest that the damage possible to the coating for those conditions will be tolerable only when both the top and bottom mats are epoxy coated. Thus, when cracks appear they should be sealed. They cause problems related to both the durability and corrosion prevention characteristics of decks. Additional information on the interrelation between cracking and bar corrosion is contained in Appendix C.

Plastic Shrinkage Cracks

This type of cracking generally has a random pattern and appears in low-slump dense and latex-modified concrete more often than in conventional concrete. Often the former mixes do not have sufficient bleed water for evaporation purposes. For conventional concrete the evaporation rate during construction should not exceed 0.2 $lb/ft^2/hr$, and for latex-modified and low-slump dense concrete a maximum rate of 0.15 $lb/ft^2/hr$ may be more appropriate. ACI 305R-77 provides a chart for determining the surface evaporation rate based on environmental conditions. If construction is continued under critical evaporation conditions, measures should be taken, such as keeping the aggregate cool, using fog nozzles to maintain a sheen of moisture on the surface after finishing, and curing promptly after placement (*30*). For latex-modified and low-slump dense concretes, the amount of time between placing the concrete and curing should be kept below 30 min.

Flexural Cracks in Plastic Concrete

In unshored construction, transverse cracks may form in a deck's plastic concrete over and near the supports of the continuous spans due to the deck's dead weight causing negative moments in the girder system. This type of cracking is unlikely if the negative curvature of the supporting girder system is less than 4×10^{-4} in.$^{-1}$ for 7 ½-in.-thick decks. If the curvature is otherwise, the concrete should be placed first in the center of the spans.

Settlement Cracks in Plastic Concrete

This type of cracking usually forms over and parallel to the uppermost bars after the concrete is finished. These cracks are usually transverse, because the uppermost bar in most decks is transverse. As shown in Table 4 (*31*), different combinations of bar cover, bar size, and concrete slump can be used to minimize this type of cracking.

Table 4. Probability of subsidence cracking predicted by regression analysis.

	Probability of Cracking (%)*								
Slump (in.)	2			3			4		
Bar Size / Cover (in.)	#4	#5	#6	#4	#5	#6	#4	#5	#6
3/4	80.4	87.8	92.5	91.9	98.7	100.0	100.0	100.0	100.0
1	60.0	71.0	78.1	73.0	83.4	89.9	85.2	94.7	100.0
1-1/2	18.6	34.5	45.6	31.1	47.7	58.9	44.2	61.1	72.0
2	0.0	1.8	14.1	4.9	12.7	26.3	5.1	24.7	39.0

* Computed probability values of less than 0 percent or greater than 100 percent are reported as 0 percent or 100 percent, respectively.

Drying Shrinkage Cracks

These cracks are usually random and transverse. They form in the concrete after it has been cured and as it loses moisture to the environment. Any procedure that reduces the total water content for the mix, such as a reduced slump, an increased coarse aggregate size, an increased proportion of aggregate, or the placement of the concrete at lower temperatures, reduces drying shrinkage and the resulting cracking (*30*).

A minimum amount of reinforcement is needed, as specified by AASHTO (*32*), to counteract shrinkage, control the width of the cracks, and distribute them uniformly when they form. If shrinkage reinforcement is embedded too deeply in the concrete (3 in. or more), the lengths and widths of the shrinkage cracks may become excessive.

Flexural Cracking under Service Conditions

Concrete bridge decks crack transversely under dead and/or live loading, over and near the interior supports of continuous spans, due to the longitudinal flexibility of the superstructure. Concrete decks may also crack longitudinally due to differential girder deflections caused by transverse flexibility. Longitudinal cracking, however, does not occur frequently in bridge decks.

Typically, the extent of the potential transverse crack region in steel girder bridges is larger than that in prestressed concrete girder bridges because steel girder bridges are more flexible. However, with increasing strengths for the concrete used in prestressed girders, they too can become more flexible and the extent of their potential crack regions can increase (*33,*)Appen. D). Calculations show that transverse flexural cracking due to repeated live loading may extend through approximately 40 percent of the span on each side of an interior support for prestressed girder systems and through approximately 60 percent of the span for steel girder systems.

The widths at the supports for transverse cracks are similar for steel and prestressed girder systems. This can be attributed to the use of a stress limit for the reinforcing steel of 20 ksi (*32,* Appen. D). The effect of concrete overlays on possible widening of transverse cracks at the surface is minimal. Calculations show a likely maximum crack width of 0.010 in. for bare decks and 0.012 in. for decks with a 2-in. concrete overlay.

Cracking Caused by Traffic-Induced Vibrations

The dynamic effects of traffic-induced vibrations (and repeated deflection reversals) can cause cracking or can cause existing cracks in bridge decks to lengthen or deepen. The amplitude and frequency of the vibrations determine the risk of cracking due to dynamic effects (*34*). Excessive live load deflections of the superstructure may also result in increased dynamic deflections (amplitude) without a significant change in their frequency (*35,* Appen. C). Thus, the net effect may be an increased risk of cracking from traffic-induced vibrations. Repeated vibrations over time can weaken concrete and cause cracks, especially at vertical planes of weakness such as those formed in plastic concrete and located directly above and parallel to the uppermost transverse reinforcing steel. Cracking due to vibrations over a long period of time will logically occur in older bridges, bridges with higher traffic volumes, higher traffic speeds, and longer span lengths (Appen. C).

For a constant span length, the curvature developed by the superstructure for live loading is proportional to the deflection. Calculations show that for typical steel girder bridges (composite and noncomposite) the longitudinal curvature at midspan is approximately twice the same curvature for typical prestressed concrete girder bridges (Appen. D). Concrete overlays decrease the longitudinal midspan curvature about 20 percent only (Appen. D).

Guidelines for Minimizing Cracking in Concrete Bridge Decks

The following are issues that should be considered in the design and construction of bridge decks in order to minimize cracking:

- The flexibility of the superstructure.
- The amplitude and frequency of the superstructure's vibrations under live loading.
- The bar cover, uppermost bar size, and concrete slump.
- The concrete mix water.
- The curvature of an unshored girder system over the interior supports of continuous spans when placing the deck.
- The rate of surface evaporation and the availability of bleed water when placing the concrete.
- The amount of time between placing and curing the concrete.

COST EFFECTIVENESS OF PROTECTIVE STRATEGIES

Table 5 compares lifetime costs for the different bridge deck construction alternatives (Appen. E). Those costs are presented in a matrix so that the percentage of the additional cost of one alternative can be determined relative to another. Differences in deck cost for singly protected decks vary from 2 to 17 percent. For doubly protected bridge decks, differences in deck cost vary from 2 to 13 percent. When the double protection alternatives in Table 5 are compared with the single protection alternatives, the most expensive double protection alternative (epoxy coating the top mat in conjunction with a low-slump dense or latex-modified concrete overlay) costs 22 percent more than the least expensive single protection alternative (bar cover of 3.5 in.). The least expensive double protection alternative (epoxy coating both mats) costs about 9 percent less than the most expensive single protection alternative (low-slump dense or latex-modified concrete overlay).

Highway agencies use double protection because they lack long-term field experience with protective strategies. Although some agencies use double protection on every new deck, others have developed criteria for determining if a deck warrants double

Table 5. Comparison of lifetime costs of bridge deck construction alternatives.[1]

	Alternative No.[2]	Single Protection				Double Protection		
		II	III	IV	V	VI	VII	VIII
Alternative No.[2]	Protective Strategy	Cover thickness = 3.5"	Epoxy-coated top mat	Special conc. overlay	Interlayer membrane	Epoxy-coated top & bottom mats	Epoxy-coated top mat & special conc. overlay	Epoxy-coated top mat & interlayer membrane
Single Protection II	Cover thickness = 3.5"	100 / 100	103 / 100	117 / 100	115 / 100	107 / 100	122 / 100	119 / 100
Single Protection III	Epoxy-coated top mat	100 / 103	100 / 100	114 / 100	111 / 100	104 / 100	118 / 100	116 / 100
Single Protection IV	Special conc. overlay	100 / 117	100 / 114	100 / 100	100 / 102	100 / 109	104 / 100	101 / 100
Single Protection V	Interlayer membrane	100 / 115	100 / 111	102 / 100	100 / 100	100 / 107	106 / 100	104 / 100
Double Protection VI	Epoxy-coated top & bottom mat	100 / 107	100 / 104	109 / 100	107 / 100	100 / 100	113 / 100	111 / 100
Double Protection VII	Epoxy-coated top mat & special conc. overlay	100 / 122	100 / 118	100 / 104	100 / 106	100 / 113	100 / 100	100 / 102
Double Protection VIII	Epoxy-coated top mat & interlayer membrane	100 / 119	100 / 116	100 / 101	100 / 104	100 / 111	102 / 100	100 / 100

1 The table determines percent additional cost of one alternative relative to the other.
2 See Figure E-1 for detailed description of the alternative bridge deck construction.

protection. Typical criteria are the type of structure and the impact of possible deck repair on traffic. Epoxy coating the top mat steel is not only preferred by most state transportation departments for "single protection" of bridge decks, it is also a necessary element of double protection. Epoxy coating the bottom mat bar only adds about 4 percent to overall construction costs (Table 5). Therefore, since several highway agencies have not been completely satisfied with the performance of overlays on bridge decks (Appen. F), it may be prudent to epoxy coat both the top and bottom mats in every bridge deck built in a salt environment, rather than apply "overlay protective strategies." Such overlays may, however, be desirable for certain types of construction. Because of bond concerns prestressing steel is usually not epoxy coated. Therefore, a chloride proofing overlay is realistic for structures, such as box girders, that use prestressing in their decks. Overlays may also be useful as leveling courses on decks built of precast concrete girders, such as bulb-T girders.

CHAPTER FOUR

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

Phase I of this study, reported here, identified the effects on a bridge deck's serviceability and life of many factors involved in its design, construction, and maintenance. Accordingly, tentative recommendations were developed for cost-effective protective strategies to provide 50 years or more of service life.

The conclusions drawn from this study were the following:

1. The past performance of bridge decks indicates that they will require major maintenance, in the form of resurfacing or overlaying, before their 50-year service life is completed. Even when there is no deterioration caused by corrosion, maintenance will be needed as a result of distress caused by traffic action and weathering. Typical forms of distress are extensive wear in wheel lines, lack of skid resistance, scaling of concrete surfaces, and stripping of overlays. For interlayer membrane/asphaltic concrete systems, distress caused by traffic actions requires correction every 10 to 15 years.

2. The "corrosion-free" service life of bridge decks employing concrete protective strategies (such as increased depth of cover to the top reinforcing bar, a low-slump dense concrete overlay, or a latex-modified concrete overlay) is dependent on the frequency and severity of salt applications. This research developed procedures to predict approximately each strategy's "corrosion-free" life for a given salt environment and the characteristics required for each strategy to provide 50 years or more of service life free of corrosion-induced deterioration. By contrast, the "corrosion free" service life of decks with interlayer membrane/asphaltic concrete protective systems is not governed by the extent of salt applications. Such membranes, when properly constructed, can prevent salt infiltration indefinitely. Their service life is dependent on the rate at which the membrane deteriorates because of aging and traffic loading effects.

3. Current knowledge on the performance of epoxy-coated bars is drawn mainly from the results of laboratory investigations. That knowledge suggests that decks constructed with epoxy-coated top mats can have 50 years of "corrosion-free" service life even in extreme salt environments and severe exposure conditions. However, the "corrosion-free" life is sensitive to the amount of damage that occurs to the coating prior to, or after, placement of the bar in the deck. Such damage becomes a potential site for accelerated corrosion and the resultant pitting can cause premature fatigue failure of the bar. Epoxy coating of the bottom steel mat, in addition to the top mat, can mitigate the effects of damage. The long-term durability of epoxy coating in a chloride-contaminated concrete is also a concern. Deterioration of the adhesion between the coating and the bar is possible because of the presence of holidays in the coating and flexing of the bar adjacent to cracks.

4. Cracking affects the corrosion prevention characteristics of different protective strategies by allowing avenues for moisture and salt to move freely into the concrete. Unless cracks are sealed as they form, they can accelerate reinforcing bar corrosion. A large proportion of the cracks in bridge decks are transverse cracks that occur along the transverse reinforcing steel and extend down to it. Such cracking raises concerns about the long-term durability of the epoxy coating protecting those bars. Coating may be worn away by crack movements caused by the deck's flexing. For the transverse crack widths likely in bridge decks, the amounts of worn coating may be tolerable only when both top and bottom mats are epoxy coated.

5. Bridge deck cracking is the result of many factors dependent on both design and construction procedures for bridges. This study investigated those factors and suggested methods for minimizing cracking and appropriate procedures for sealing cracks.

6. The cost-effectiveness of different single protection strategies and realistic double protection strategies was evaluated based on 50-year lifetime costs. For singly protected decks the least expensive strategy was the provision of a concrete cover over the uppermost bar of at least 3.5 in. The other less expensive strategies, in order of increasing costs, were epoxy coating of the top steel mat, provision of an interlayer membrane/asphaltic concrete system, and provision of a low permeability concrete overlay (low slump dense or latex-modified concrete). Double

protection with epoxy coating of both the top and bottom steel mats was less expensive than an interlayer membrane/asphaltic concrete protection system.

RECOMMENDATIONS

Although tentative recommendations were developed for a second research phase that would include detailed studies of the factors identified as significant for corrosion protection of bridge decks, together with detailed field studies to validate and enhance the findings of the Phase I investigation, the NCHRP decided not to pursue further work in this area (see Foreword for additional information).

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APPENDIX A

DURABILITY OF BRIDGE DECKS AND THEIR PROTECTIVE COMPONENTS

Bridge decks are subject to freeze-thaw action and exposed to deicing salts and direct impact from traffic. The combination of these factors creates a very severe environment for concrete. Freeze-thaw action can cause scaling or deterioration of surface mortar, especially in the presence of deicing salts. Freeze-thaw action can also attack the aggregate within the concrete, causing distress in the form of popouts and surface pitting as a result of fracturing the aggregate near the surface. Concrete overlays may debond because repeated shear stresses caused by live loading and thermal cycling decrease the shear strengths in the interface. Wear and surface polishing of decks exposed to traffic may occur, influencing the skid resistance of the surface. Surface rutting in the presence of high traffic volumes may occur in the wheeltracks, especially when studded tires are permitted. Water then collects in ruts, accelerating deterioration.

This appendix reviews bridge deck durability information collected from published literature, analyzes that information and suggests ways to reduce problems relating to the durability of bare decks (i.e., decks with increased depths of cover and epoxy-coated bars) as well as those overlaid with special concretes (i.e., low-slump dense and latex-modified concrete) and asphaltic concrete/membrane systems. This appendix considers the durability of decks and their protective components to be an element pertinent to the determination of deck serviceability. The findings of this appendix are supplemented by information collected through a nationwide survey of transportation departments and in-depth interviews of selected departments, as presented in Appendixes F and G, respectively.

BRIDGE DECKS WITH OVERLAYS

This category includes bridge decks with epoxy-coated bars and those with increased bar covers. In both cases durability, as discussed in this report, concerns the exposed concrete. Since no overlays are applied, debonding and stripping are not problems. Thus, durability is mainly a function of the exposed concrete's resistance to freeze-thaw action and wear under traffic.

Performance History

The Portland Cement Association, in cooperation with the U.S. Bureau of Public Roads, began a study of bridge deck durability in 1961. They made condition surveys of over 1,000 randomly selected decks in eight states: California, Illinois, Michigan, Minnesota, New Jersey, Ohio, Texas, and Virginia. The decks were built between 1940 and 1962 and utilized both nonair-entrained and air-entrained concrete. The data demonstrated the improved resistance to scaling of air-entrained concrete (*1*). As illustrated in Figure A-1, air-entrained concrete showed fewer incidences of scaling and less extensive scaling than nonair-entrained concrete.

Among the eight states in the study, Virginia, which had adopted air entrainment comparatively late, reported significantly more scaling (*2*). Texas, which had not used air-entrained concrete at that time, also reported high frequencies of scaling. On the other hand, Minnesota had used air entrainment and, despite its severe climate and its use of salt, it reported the lowest incidence of scaling of any of the eight states except California (*2*).

This study also included detailed investigations of 68 decks in four states (Kansas, Michigan, California, and Missouri) to determine the cause of scaling in air-entrained concretes. Scaling was attributed mainly to a nonuniform air void distribution and, especially, to the presence of thin, irregular zones at the surface, probably caused by improper placing and finishing procedures (*1*). The researchers also noticed that deck drainage was an important factor influencing durability. Scaling was mainly confined to the gutter areas.

Cady and Theisen (*3*) recorded construction procedures and subsequently monitored the durability of seven bare decks built in the mid-1960s in Pennsylvania. The requirements for concrete slump was 2 ± 1 in. and the requirements for air content was 6.5 ± 1.5 percent. The 3-year monitoring period started when the structures were only a few years old. The decks' surface mortar deteriorated, and the level of deterioration generally increased with time. However, no distinction between general scaling and wear in wheeltracks was reported. In some bridges the deterioration extended over the entire deck. The deterioration was more extensive in those decks in which slump test values exceeded those specified and in those which had experienced sprinkling or overworking of the surface during construction. In five of the decks, more than 35 percent of the slump tests had values higher than the specified value.

In a subsequent study in Pennsylvania (*4*), 249 four-year-old decks, all built in 1966 and all air-entrained, were inspected. Ninety-five percent of the decks exhibited surface mortar deterioration, which included wear in wheeltracks, polishing, and freeze-thaw scaling. However, of the affected areas, 97 percent of the distress was attributed to wear in the wheeltracks and only 3 percent to freeze-thaw scaling. Surface mortar deterioration was related primarily to construction practices (especially finishing), but also to the use of antiskid materials and the average daily traffic volume.

The Concrete Reinforcing Steel Institute recently conducted a survey of 10- to 15-year-old, continuously reinforced concrete pavements in Oregon (CRSI-ARBP Transportation News, Winter 1986). Studded tires had caused polishing and rutting of the pavements in the wheel paths. Traffic on those pavements was approximately 33,000 ADT with 17 percent trucks.

In 1970 Newlon et al. (*2*) investigated concrete bridge decks in Virginia to determine changes in their performance since the inspection of the same decks in the 1961 Portland Cement Association/Bureau of Public Roads survey. Sixty-six bridges comprising 206 bare spans were examined in the 1970 survey.

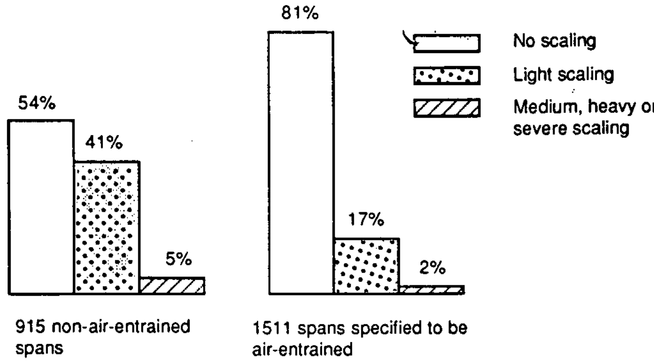

Figure A-1, Influence of air entrainment on occurrence of scaling (1).

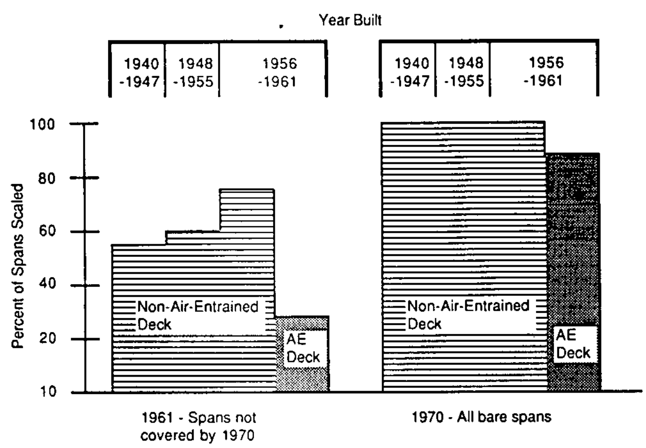

Figure A-2. Influence of air entrainment and age on occurrence of scaling in Virginia bridge decks. (Adapted from Ref. 2)

Findings on the combined influence of age and air entrainment on scaling are summarized in Figure A-2. In 1970 only 10 percent of the air-entrained decks were free of scaling, compared to 72 percent in 1961. One reason for this was reported to be the difficulty in distinguishing between light scaling and abrasion. The other reason was believed to be the tendency to work during construction at the lower limit of the air content, specified as the 3 to 6 percent range. Spans that were free of scaling in 1970 were all air entrained. The Virginia specifications were revised in 1965 to require an air content of 6.5 ± 1.5 percent (*2*).

Newlon and Walker conducted an in-depth investigation of the performance of 17 Virginia bridge decks, 14 years after their construction, in order to relate their existing condition to construction procedures (*5*). The decks were constructed in 1963-1964 and inspected in 1977. The estimated cumulative freeze-thaw cycles ranged from 180 to 585 cycles, and the estimated total number of deicer applications ranged from 28 to 702. The condition of the decks ranged from no scaling to medium scaling on 60 percent of the deck area, with heavy scaling on 25 percent of the deck area. A few concrete decks that met certain requirements had resisted scaling for 14 years and up to 560 freeze-thaw cycles and gave no indication of a change in that behavior in the foreseeable future. Those requirements were an air void spacing factor of <0.008 in., a water/cement ratio of <0.45, a water absorption for the concrete of <4.5 percent, and use of a water reducer. The investigation also showed the effectiveness of linseed oil treatment in preventing scaling and the significant detrimental effect for scaling of a silicone surface treatment. Another factor contributing to scaling was the deck geometrics. Where there was better surface drainage the scaling was lower. There was also evidence that poor performance was related to excessive delay in curing.

Discussion

Scaling. Bridge decks are exposed to deicing chemicals, saturated freezing conditions, and high numbers of freeze-thaw cycles. Presently, air entrainment is the primary means of controlling freeze-thaw scaling damage. As can be seen in Figure A-3, the importance of air entrainment has been recognized increasingly over the years and its specified limits have increased correspondingly. Until 1960, most specifications required a 3 to 6 percent air content in the concrete (*6*). Concrete sensitivity to freeze-thaw damage when its air content is in the transition zone between 1 ½ and 3 ½ percent is shown in Figure A-4 (*5*). The figure, which is based on work by Cordon and Merrill (1963), suggests that an air content of at least 4 percent is required to assure good durability in concrete, particularly for a high freeze-thaw environment. A nationwide NCHRP survey (*6*) showed that 1981 specifications, which were only slightly different from those of 1975, required on the average air contents for which the lower limit was 4 percent, the upper limit was 7 percent, and the midpoint was 5.5 percent. Two states (Hawaii and Georgia) required air content lower limits of less than 3 percent, and one state (West Virginia) required a 10 percent upper limit. Variations depended on the severity of the freeze-thaw action in each area.

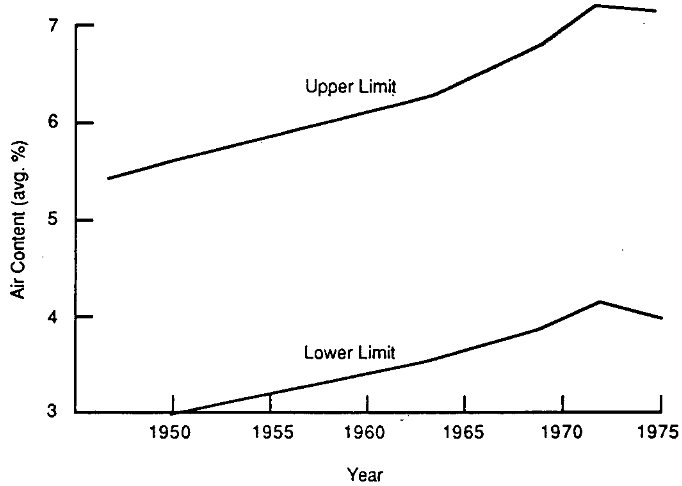

Figure A-3. Average air contents specified from 1947 through 1975. (Adapted from Ref. 6)

Examinations of the field conditions of bridge deck concretes by Newlon and Walker (*5*) indicated a strong relationship between air contents determined in the fresh concrete and the air void spacing factor. They did not find acceptable spacing factors (less than 0.008 in.) for fresh concretes with air contents less than 4.5 percent. They suggested, as described previously, that acceptable freeze-thaw resistance should be expected from bridge decks constructed according to requirements, including an air void spacing factor in hardened concrete of less than 0.008 in. However, note that air contents in fresh concrete greater than 4.5 percent do not necessarily provide such air void spacing factors. Further, the water/cement ratio is another important element and its value during construction should be limited to 0.45 (*5*). A reduction in the water/cement ratio, coupled with

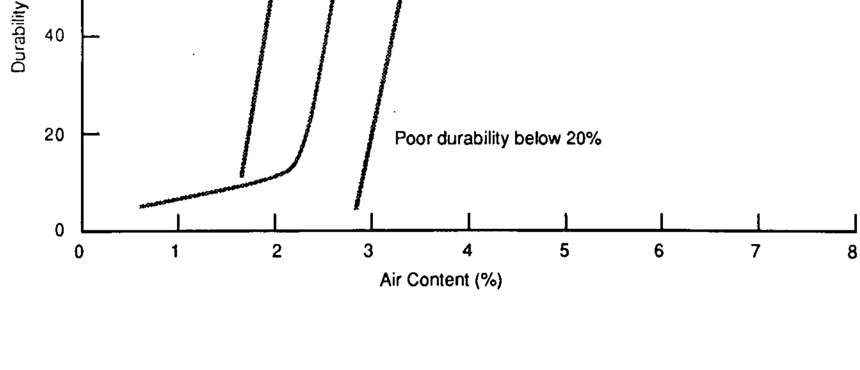

Figure A-4. Durability of concrete containing various aggregates, cement contents, water/cement ratios, and air contents. (After Condon and Merrill, 1963; Ref. 5) proper consolidation, results in less capillary voids in the hardened concrete, providing less permeability and a higher freeze-thaw resistance. A 1979 NCHRP survey showed that 60 percent of the 48 states participating in the survey required water/cement ratios smaller than 0.45 in bridge decks (*7*). The use of water reducers in the mix also lowers the water/cement ratio, regardless of its calculated value. Thus, reducers indirectly assure durable concrete. Finally, the moisture absorption should be limited to 4.5 percent. That absorption reflects the combined effects of the degree of consolidation achieved during construction and the water/cement ratio (*5*).

Other elements that adversely influence the freeze-thaw durability of bridge decks include sprinkling or overworking of the surface during construction, which can alter the air void system at the surface and decrease the concrete's strength; an inadequate surface drainage system that causes water ponding and ultimately saturation of the air-entrained concrete, especially in the presence of deicing salts; delays between concrete placement and curing (*5*); an insufficient air drying period before the first application of deicing salts (*6*); and treatment of the concrete surface with sealants, the effectiveness of which has not been demonstrated (*5,8*).

Air entrainment is of little value in increasing the freeze-thaw durability of concrete if frost susceptible aggregates are used. Reducing the size of the aggregate may minimize such damage (*9*), although Kansas has reported that such reductions do not work for all aggregates. The best procedure is to preclude the use of frost susceptible aggregates. Several test methods are available to identify frost susceptible aggregates (*10*).

Wear. A concrete surface requires textures to provide skid resistance. Generally, two textures are needed, one fine and the other coarse. The function of the fine texture is to supply enough contact area with the tire that adequate skid resistance is developed. The fine aggregate in the mix provides the fine surface texture. The higher the fine aggregate proportion in the concrete the better the skid resistance. The skid resistance of pavements can be measured quantitatively using ASTM E274. Some fine aggregates polish under high traffic action to such a degree that the surface becomes slippery. One laboratory test that evaluates the polish susceptibility of an aggregate is the acid insoluble residue test (ASTM D3042).

A surface needs coarse texture to provide channels for the surface water to escape and to prevent hydroplaning on wet pavements. Coarse texture also adds to the skid quality of the surface. In concrete the coarse texture is provided by tining or brooming the plastic surface. Tining generally provides $\frac{3}{16}$-in.-deep transverse traction grooves in the concrete. Coarse texture may also be built into a hardened surface by saw cutting grooves. However, traction grooves in concrete can wear rapidly and completely disappear under high traffic volumes. When that occurs, the coarse aggregate in the mix affects the skid resistance. Crushed aggregate produces better texture than uncrushed aggregate. The wear resistance of concrete can be increased by decreasing its water/cement ratio, increasing its cement factor, and employing proper finishing and curing procedures. Detailed information on the skid qualities of pavements is available in the AASHTO publication, "Guidelines for Skid Resistant Pavement Design."

Extensive surface wear results in wheeltrack rutting. Rut depths of $\frac{1}{2}$ in. can be expected in the concrete of urban freeways after 15 to 20 years of studded tire wear. Rutting in turn can cause further surface distress by permitting water to accumulate. Extensive rutting in concrete may require complete overlaying or partial patching of the ruts. For partial patching, a polymer type material that can develop sufficient bond in ultrathin layers is required.

BRIDGE DECKS WITH CONCRETE OVERLAYS

Included in this category are low-slump dense concrete and latex-modified concrete overlays. The durability of overlayed bridge decks depends on the exposed concrete's resistance to freeze-thaw scaling and wear under traffic, and on the likelihood of debonding and stripping of the overlay from the deck. Cracking in overlays is not an imminent distress problem; however, it can contribute to further internal deterioration of the overlay and to its stripping from the deck. This action may occur when surface moisture accumulates in the cracks and sufficient freeze-thaw cycles develop (*11*). The performance of concrete overlays with respect to cracking, the causes of cracking, methods for preventing cracking, and methods for repairing cracks are presented in Appendix C.

Performance History (Low-Slump Dense Concrete)

Scaling and Wear

An evaluation of concrete bridge deck surfacing in Iowa, conducted in 1978 (*12*), showed no evidence of surface distress in 15 low-slump dense concrete overlays. The age of the overlays ranged from 5 to 13 years. An evaluation program of bridge deck protective systems conducted in Minnesota (*13*) in 1982 investigated the conditions in 31 low-slump dense concrete overlays with ages ranging from 4 to 6 years. An analysis of the Minnesota data indicates that 39 percent of the overlays did not show any sign of scaling, 45 percent showed scaling over less than 1 percent of the deck area, and 16 percent showed scaling over one to 4 percent of the deck area. The scaling was more evident in older decks. For many overlays scaling appeared in curb areas after the first winter. That scaling was reported to be caused by overfinishing of the concrete. In 1978 Minnesota (*14*) reported the skid resistance of 19 low-slump, dense concrete overlays after 2 to 3 years of service. That data showed the average skid number (measured at 40 mph) as 39, with a maximum of 45 and a minimum of 32.

Debonding and Stripping

The 1982 Minnesota study (*13*) included seven 2-course new construction decks. None of the overlays showed debonding after 4 to 6 years of service. Some decks were exposed to average daily traffic volumes as high as 46,000. The overlays had thicknesses of 1.5 or 2 in. In 1983 a bridge deck performance study in Missouri (*15*) investigated debonding of five 2-course new construction decks in their driving lanes. Those overlays had thicknesses of 2 and 2.25 in. The decks were from 3 to 5 years old at the time of the investigation. One overlay showed no signs of debonding, and three of the overlays sounded hollow over 0.2 percent, 0.3 percent, and 3.1 percent of their surface areas and were assumed to have debonded. Interestingly, 12.9 percent of the driving lane of the overlay with 3.1 percent debonding had been debonded before the lane was opened to traffic; it had been subsequently repaired and patched after construction. Thirty-five percent of the fifth overlay's driving lane had debonded, as verified by drilling, but that overlay did not show any debonding initially. The latter two overlays were constructed on a non-textured substrate with water blast treatment prior to overlaying. Since that time Missouri has required a very rough texture and use of a wire comb or scarifier prior to overlaying.

Performance History (Latex-modified Concrete)

Scaling and Wear

Iowa's evaluation of concrete bridge deck surfacing (*12*) included three latex-modified concrete overlays that were 5 years old. The deck condition determination (conducted in 1978) reported no evidence of surface distress in the overlays. In 1982 Minnesota's bridge deck protective system evaluation program (*13*) reported conditions for eight latex-modified concrete overlays with ages ranging from 6 to 9 years. Only three overlays (37 percent) showed signs of scaling over approximately 1 percent of the surface area. Two of the scaled sites were 8 and 9 years old and were mortar type overlays. In 1978 Minnesota (*14*) reported skid qualities of seven latex-modified concrete and mortar overlays after 2 to 6 years of service. The Minnesota data gave the average skid number (measured at 40 mph) as 48, with a maximum of 56 and a minimum of 38. Bishara (*16*), in an investigation of 132 bridge decks overlaid with latex-modified concrete in Ohio (47 bridges), Michigan (57 bridges), Kentucky (17 bridges), and West Virginia (11 bridges), reported that the overlays provided adequate freeze-thaw resistance, and that virtually no scaling was observed. When surveyed, these overlays were between 1 and 13 years old. Bishara (*16*) also investigated the anti-skid qualities of 44 of the 132 overlays. The ages of these overlays varied between 2 and 5 years. The limited available data indicated that the skid number (at 40 mph) decreased from an average of 45 to an average of 41 after 5 years of service. Sprinkel (*17*) studied the freeze-thaw performance of 12 latex-modified concrete installations in Virginia and reported in 1984 that scaling due to freezing and thawing had not been a problem. Those installations were between 1 and 10 years old.

Debonding and Stripping

Of six 2-course new construction decks tested for debonding in Minnesota (*13*) (ranging from 6 to 7 years in age), the surface of one was debonded 0.4 percent and the surface of another was debonded 0.3 percent. (Since the chloride content of the concrete at the level of the steel was less than the threshold value, the authors assumed the defective areas were debonded and not delaminated.) One of the debonded decks was a ¾-in. thick, mortar-type overlay. Among five 2-course new construction decks tested in Missouri (*15*) for debonding (ranging in age from 4 to 7 years), one showed no sign of debonding and the surface areas of the others were debonded 0.1, 0.1, 0.2 and 1.1 percent. An analysis of the date indicated that for the Missouri decks age was not a factor in the debonding. Bishara's investigation of bridge decks overlaid with latex-modified concrete (*16*) included six 2-course new construction decks in Ohio (1 to 5 years old), 12 in Michigan (1 to 6 years old), one in Kentucky (2 years old), and four in West Virginia (2 years old), for a total of 23 decks. An analysis of data indicated that slight debonding was found only in two Ohio decks. The debonding covered 0.13 percent of the surface area for both decks and the debonded overlays were 3 and 4 years old.

Discussion

Scaling. Although some scaling of low-slump dense concrete overlays has been reported, especially in curb areas, 13-year-old performance of this type of overlay with no sign of freeze-thaw scaling has also been documented. Factors that contribute to the scaling of conventional air-entrained concrete, as discussed earlier, can also contribute to the scaling of low-slump, dense concrete. Air-entrained, low-slump dense concrete, because of its lower water/cement ratio, has an excellent ability to resist freeze-thaw and deicer scaling. Low-slump dense concrete incorporates air contents in the range of 6.5 ± 1.0 percent and develops satisfactory air void systems (*6*). However, failure to achieve the specified air content in low-slump dense concrete is possible. Whiting and Stark (*6*). However, failure to achieve the specified air content in low-slump dense concrete is possible. Whiting and Stark (*6*) have reported that up to ten times the normal dose of air entraining agent is sometimes needed to produce desired air contents in low-slump dense concrete mixes. They suggested that the effectiveness of air entraining agents be examined in low-slump dense concrete applications prior to field use because achieving the specified air content in these overlays, even with high dosages of particular air-entraining agents, may not be possible. An important element contributing to the durability of low-slump dense concrete is its consolidation. Inadequate consolidation in field applications due to the stiffness of the mix may result in entrapped air voids that can adversely affect freeze-thaw durability.

Latex-modified concrete overlays have provided adequate freeze-thaw resistance and performed satisfactorily for at least 13 years with insignificant scaling. Latex-modified concrete does not incorporate air entrainment and its air void spacing factor is generally larger than 0.008 in., considered the maximum desirable size for freeze-thaw durability (*17,18*). However, field performances show that the latter requirement is not necessary for latex-modified concrete. One explanation is that the latex emulsion prevents the penetration of water so that an adequate void structure is not needed (*17*).

Wear. Field evaluations in Michigan (*19*) indicated that low-slump dense and latex-modified concrete overlays have almost the same skid resistance. However, field tests in Minnesota (*14*) showed higher skid numbers for latex-modified concrete overlays, regardless of their service period. That finding may be due to the nature of the latter mix, which can be finished and textured more easily than a low-slump dense concrete and to the higher proportion of fine aggregate in the latex-modified concrete. Limited data are available on the changes in wear and anti-skid qualities of low-slump dense and latex-modified concrete overlays over time and with traffic use. In spite of their low water/cement ratios, the performances of these concretes have not differed markedly from the performance of conventional concrete. As in conventional concretes, the use of polish- and wear-resistant, fine aggregate improves the skid qualities. Also, for low-slump dense concrete overlays, due to the stiff nature of the mix, cutting traction grooves in the hardened concrete may be more effective than timing the plastic mix.

Debonding and Stripping. Low-slump dense concrete overlays have demonstrated satisfactory integrity with the underlying decks after many years of service life and under heavy traffic exposure. However, there have been a few cases of considerable debonding in which construction procedures and inefficient substrate texture have been identified as the main cause of the problem. The procedure for bonding on two-course new construction is generally to broom the substrate to a rough texture during the first stage of construction, sand blast it (*20*), and then apply a bonding grout to the surface dry substrate. If the second stage construction is delayed and the deck has been open to traffic, scarifying to a depth of ¼-in. prior to sand blasting may be required (*7*). However, some agencies may require scarification regardless of when the overlay is placed. If after sand blasting and prior to overlaying the deck becomes wet because of rain, blasting should be repeated when the deck is surface dry. It is important that the bonding agent does not dry out when the overlay is applied. Some agencies may scrub a mortar portion of the overlay mix on the deck to serve as a bonding agent. Use of a grout especially made for this purpose, however, may be more effective. When proper construction measures are taken, the bond of low-slump dense concrete overlays should be durable, although it can be affected by repeated loading, thermal cycling, and wetting and drying. Shear stresses of up to 64 psi can develop between a 7-in. thick uncracked slab and a 2-in.-thick overlay under an AASHTO H-20 truck loading (*7*). The overlay and substrate have almost identical coefficients of thermal expansion and moduli of elasticity, which provide thermal compatibility. However, temperature and moisture differentials exist throughout the depth of the deck, causing shear stresses. Laboratory tests (*19*) have found satisfactory bond between the overlay and substrate after 105 freeze-thaw cycles. The average shear bond strength at the conclusion of those tests was 478 psi.

While most latex-modified concrete overlays have shown total integrity with the underlying deck, at least after 7 years of service, some overlays have debonded. However, the amount of debonding has been small and generally below 0.5 percent of the surface area. Further, no overlay stripping has been reported with the two-course new constructions. In investigations in Virginia (*17*) the bond strengths of latex-modified concrete overlays were determined by coring the installations and conducting in the laboratory shear bond tests at the interface. A two-course new construction demonstrated over 500 psi shear strength after 9 years of service. Lee et al. (*21*) in their laboratory investigation of the flexural fatigue strength of latex-modified and low-slump dense-layered concrete beams observed no bond failures between layers. The minor debonding problems in latex-modified concrete overlays are probably related to inadequate construction procedures, which result in an initial low bond strength at the interface. During a deck's service life, the shear stresses caused by repeated loading, and temperature and moisture differentials throughout the depth of the deck, may reduce the bond strength at the interface and ultimately cause debonding. Therefore, provisions to provide satisfactory initial bond strength are important. Requirements for bond for latex-modified concrete are the same as those for low-slump dense concrete except that the deck must be kept wet for at least one hour prior to overlaying. The bonding agent can be either a latex slurry or the mortar portion of the overlay mix scrubbed onto the wet deck. However, the former procedure is a better construction practice because workers frequently put excess mortar material back into the overlay instead of wasting it. The wetness of the deck will prevent latex from rapid drying (*7*).

BRIDGE DECKS WITH ASPHALTIC CONCRETE/ MEMBRANE SYSTEMS

This protective system consists of a waterproofing membrane applied to the concrete deck, followed by the application of an asphalt concrete overlay for the purpose of protecting the membrane and providing a wearing surface. Several different materials have been used as waterproofing interlayer membranes. Generally such materials can be divided into two groups: factory laminated (preformed) sheet membranes and applied-in-place liquid membranes (*7*). Durability concerns for the system are first debonding and stripping of the asphalt concrete and, second, wear of the overlay. Cracking of the asphalt overlay is also a factor affecting the deck's durability. Cracks may develop into debonding and stripping in their later stages.

Performance History

Debonding and Stripping

Minnesota reported in 1982 (*13*) that debonding had occurred in seven new construction decks protected by both preformed sheet and applied-in-place interlayer membranes. The extent of the debonding varied from 9 to 43 percent of the decks' surface areas. The decks were from 5 to 7 years old, and their average daily traffic ranged from 1,700 to 6,000 vehicles. Thus, the total traffic prior to testing was between 3 and 15 million vehicles. Some installations in Minnesota were removed after only 5 years because stripping and debonding of the overlays, especially in the presence of high traffic volumes, required frequent maintenance. Penetration of water into the asphaltic concrete and its accumulation above the membrane contributed to the debonding (*13*).

In 1982-1983 interlayer membrane performance was studied in Kansas (*22*) for eight old, salt-contaminated decks that had been waterproofed between 1967 and 1974. The types of interlayer membrane included both preformed sheet and applied-in-place liquid membranes. Although the asphaltic concrete overlays exhibited debonding over 2 to 22 percent of the decks' areas, they are still performing well after 9 to 16 years of use and had required little maintenance. The only exception was the failure of part of a 9-year-old membrane that was placed on a steep downhill slope with a traffic light at the bottom end of the deck. The total traffic over the Kansas test bridges ranged from 3 to 160 million vehicles.

A Washington investigation of interlayer membranes documented that the asphalt overlay condition of a waterproofed new construction built in 1969 was good, with no debonding, patching, or cracking after 14 years of service (*23*). The bridge was located in eastern Washington and was exposed to severe winters and an average daily traffic of 6,707 vehicles, or approximately a total of 34 million vehicles up to 1983 when the tests were made. The membrane was of an applied-in-place, liquid material. However, debonding and stripping of the asphalt overlay was documented in another eastern Washington bridge 6 years after waterproofing and overlaying with asphaltic concrete (*24*). The defective area was 1.4 percent of the deck area and the average daily traffic was 3,792 vehicles, or approximately 8 million vehicles up to 1985 when the tests were made. This membrane was also of an applied-in-place, liquid material. Where the debonding and stripping had occurred, a coring operation revealed that the asphalt binder in the bottom layer of the asphaltic concrete was completely stripped off the aggregate, resulting in loss of integrity between the mixture elements or of adhesion between them and the membrane.

Cracking and Wear

Asphalt overlay cracking was found in Minnesota in seven waterproofed new construction decks, 5 to 7 years old (*13*). The cracks were transverse, random, and longitudinal. An analysis of the Minnesota data shows that 29 percent (two decks) of the asphalt overlay had 10 to 100 ft of cracking per thousand square feet of surface area, 42 percent (three decks) had 100 to 200 ft of cracking, and 29 percent (two decks) had greater than 200 ft of cracking per thousand square feet of surface area. The cracking of the waterproofed new constructions was about the same as the cracking in decks waterproofed during their service periods, regardless of the presence of delaminations in the concrete in the latter group of bridge decks.

Kansas reported cracking in five bridge decks waterproofed during their service periods (*22*). The waterproofing systems were 11 to 12 years old, and the cracking ranged from 67 to 150 ft per thousand square feet of deck area. Although corrosion tests below the membrane were not made in that study, the satisfactory performance reported for those bridges suggests that the cracking was a characteristic of the asphalt overlay rather than concrete deterioration. Kansas also reported (*22*) skid resistance number for the asphaltic concretes from the mid-to-upper 20s after exposure to 76 to 160 million vehicle passes. For lower traffic volumes the skid numbers ranged from the upper 30s to the low 50s.

Discussion

Debonding and Stripping. Even where the interlayer waterproofing membrane is well bonded to the concrete bridge deck, several factors may contribute to debonding of the asphaltic concrete wearing course.

One major factor is the accumulation of water above the waterproofing membrane in the bottom portion of the asphaltic concrete. When this water is subjected to repeated freezing and thawing, as well as repeated hydraulic pressure under traffic, the two layers tend to separate. Not only is the bond weakened, but the asphaltic concrete itself is often damaged by loss in strength or modulus, and these severe actions also contribute to stripping of the asphalt from the aggregate, accelerating deterioration. In summary, the asphaltic concrete wearing course is subjected to a severe environment that can reduce the service life compared to that for conventional asphalt pavement on grade.

The engineering properties of asphaltic concrete play an important role in providing a successful wearing course on bridge decks. Although the design of asphalt paving mixtures is a compromise of several factors, the design of paving mixtures presents some special challenges. Key factors that need to be considered include, at a minimum, the following:

- High density and low air void contents are desirable, but these properties must be balanced with the potential loss of stability under traffic and high temperature. It may not be feasible to use conventional paving mixtures on bridge decks. Special designs may be needed including high quality aggregate, stiffer binders that may include modified asphalt, and use of anti-strip additives.
- The construction phase should employ effective compaction to satisfy the design density, considering that the use of certain types of compactors may not be allowed on bridges. Construction techniques may need to be modified to achieve the required quality when overlaying asphaltic concrete. Asphaltic concrete must be well bonded to the membrane system. With careful forethought and planning the waterproofing membrane can also serve as the tack coat to assure a good bond. Quantities must be optimized; too little may result in premature debonding and too much may cause horizontal slippage under traffic, tearing, and cracking the surface.
- Particular care in the construction of joints in the asphaltic concrete layer will pay off in reduced raveling and water penetration. Proper rolling patterns and formation of hot longitudinal joints between paving lanes are required for good results.
- Surface drainage is very important. Sufficient cross slope, in combination with drains through the deck at the curb line, will reduce the potential for standing water and thus the opportunity for water to enter the pavement structure. If there is a concern about water entering the pavement and accumulating above the membrane, methods for drainage from the surface of the membrane, such as the installation of vertical pipes through the deck, may be required.

Blistering under the membrane, as well as the compatibility of the waterproofing membrane with both the concrete bridge deck and the overlying asphaltic concrete, is critical to the success of the system. Preformed membranes need sufficient amounts of dead weight from asphaltic concrete to offset any blistering. Preformed membranes may also need two different binders or a double layer so that adequate waterproofing or mastic is available to seal the surface of the concrete. Elastomeric properties are needed to resist movement and volume changes in the concrete, to bridge cracks, and to ensure that the membrane remains intact and prevents water penetration. The top surface of the membrane must serve as a bonding layer for asphaltic concrete and also resist shearing stresses caused by thermal and traffic action.

Cracking and Wear. Different elements contribute to the cracking of asphalt paved over an interlayer membrane. Transverse cracking may occur in negative moment areas due to live loading and the presence of bond between the system and the bridge deck slab. Overlay slippage under accelerating and decelerating traffic can cause crescent-shaped cracks. An interlayer membrane may reduce or delay reflective cracking in the asphaltic concrete when cracking exists in the underlying concrete slab. Longitudinal cracks may appear along construction joints as a result of weak seams. Transverse and longitudinal cracks in the asphaltic concrete can also form shortly after the asphaltic concrete is paved when the protective fabric covering the membrane shrinks. Oklahoma (*25*) found this type of cracking at joints and laps in a preformed sheet membrane covered by a polypropylene fabric. These cracks were filled with hot asphaltic concrete prior to compaction. This problem might be alleviated with lower asphaltic concrete mix temperatures (*25*). The deflation of blisters under certain types of membranes can cause dish-shaped depressions in the asphaltic concrete surface, and subsequent "Y" shaped cracking will occur in the depressions (*25,26*).*NCHRP Synthesis of Highway Practice 57* (*7*) suggested different methods to prevent blistering. These methods include applying 2 to 3 in. of wearing course to provide sufficient dead weight to offset the blistering; incorporating a perforated sheet of bituminous felt as a venting layer to allow blistering pressure to disperse; or applying a 1-in.-thick asphalt base course to act as a venting layer on the deck under the membrane. Other causes of cracking in asphaltic concrete overlays are aging and shrinkage of the asphalt. The latter usually initiates pattern cracking with excessive load repetitions.

Performance data show that in many cases cracking in asphaltic concretes paved over the interlayer membranes has exceeded 100 ft of cracking per thousand square feet of deck area. Early repair of cracks is important, because they can propagate rapidly, allowing water seepage. Cracks in asphaltic concrete can be cleaned out and sealed with emulsion slurry or liquid asphalt mixed with sand (*27*), but higher quality elastomeric crack fillers are probably better for bridge decks.

Conventional, well-designed asphaltic concrete generally has good skid resistance. However, depending on the level of traffic and the quality of the aggregate, skid resistance may decrease with time. Performance data indicate that under high traffic volumes (over 10,000 ADT), low skid numbers (below 35) can be expected after about 15 years. The use of a nonpolishing, crushed aggregate, sufficient coarse aggregate, and an adequate void structure, improves skid resistance, allows surface water to drain, and reduces the potential for hydroplaning.

Designing and maintaining a good surface on a bridge deck is a challenge and is a compromise among several factors. As indicated earlier, the wearing course needs to have a small amount of voids, but this generally results in a surface that is too tight and smooth to provide good skid resistance. Furthermore, a dense mixture has a greater tendency to deform and rut under traffic. This type of rutting, as well as wear in the wheel paths, can create ponding, thereby increasing the potential for cars to hydroplane on water or skid on ice. This ponding also contributes to accelerated damage.

The maintenance of an acceptable surface and, if necessary, its repair are important. Surface treatments such as chip seals and slurry seals have been tried with limited success, particularly under high speed and high volume traffic. Conventional asphalt binders for these treatments are being replaced by polymer modified asphalt in Europe. In addition, modified slurry seals are being used to fill in wheelpath ruts, both with and without the old pavement having been milled out before repair.

APPENDIX B

CORROSION PREVENTION CHARACTERISTICS OF PROTECTIVE STRATEGIES

The prime factors affecting the performance of concrete bridge decks are reinforcing bar corrosion and subsequent corrosion induced concreted deterioration as a result of the application of deicing salts. The need to prevent bar corrosion has directed the development of protective strategies for new bridge decks. The goal is to provide at least 50 years of deterioration-free bridge deck service.

This appendix reviews the ability of different protective strategies (i.e., increased depth of cover, low-slump dense and latex-modified concrete, interlayer membrane, and epoxy coating) to prevent bar corrosion in bridge decks. The appendix reviews the chloride-proofing characteristics of those protective strategies and predicts how long it takes chloride to build up to the threshold level for corrosion (corrosion does not necessarily initiate at the same time as chlorides reach their threshold value). This review neglects the effects of cracks in the concrete on the chloride-proofing abilities of those protective strategies. The effects of cracking on bar corrosion and concrete deterioration are discussed in Appendix C of this study. For bridge decks containing epoxy-coated bars, corrosion prevention characteristics are indicated by bar corrosion or corrosion-induced concrete deterioration when the chlorides at the level of the bar have reached threshold values.

BRIDGE DECKS WITH A DEPTH OF COVER >3 INCHES

This protective strategy employs conventional concrete with a thicker than normal concrete cover over the bar. The chloride-proofing abilities of this strategy can be determined from the performance history of conventional bare decks.

Performance History

Bishara (*16*) reported chloride contents after three, five, and eight winter exposures for a Michigan bridge deck built in 1969. The results are shown in Figure B-1. At 2 in. from the surface the chloride content was 1.3 lb/cu yd after three winter exposures, 2.5 lb/cu yd after five winter exposures, and 5.6 lb/cu yd after eight winter exposures. At 2.5 in. from the surface, the chloride content was approximately 4 lb/cu yd after eight years of winter exposure. The chloride content was the average of three to four samples taken each time and obtained in an area approximately 4 to 5.5 ft from the curb-line. The average salt application rate for the bridge was 51 tons of salt per two-lane mile per year. A single salt application can vary from 100 to 800 lb per lane-mile (*28*). Assuming an average of 650 lb per lane-mile, the number of salt applications on the Michigan bridge was approximately 78 times per year. The maximum allowable water/cement ratio for bridge decks in Michigan was reported in a 1977 NCHRP survey as 0.49 (*7*).

Information presented in one Washington bridge deck study (*24*) indicated that concentrations of chlorides were generally higher in driving lanes than in passing lanes. The five Washington bare bridge decks evaluated were built in 1965 and were tested for chlorides 14 years later, in 1979. Chloride samples were obtained at 1.5 in. to 2 in. below the surface. The average chloride content of 24 samples from driving lanes was 4.95 lb/cu yd with a maximum of 10.19 lb/cu yd and a minimum of 1.31 lb/cu yd; the average chloride content of 24 samples from the passing lanes was 2.72 lb/cu yd, with a maximum of 9.05 lb/cu yd, and a minimum of 0.20 lb/cu yd. The bridges' maximum water/cement ratio requirement was 0.45, and they were located in eastern Washington where winters are severe.

In 1977, Newlon and Walker (*5*) examined the chloride content of ten bare decks in Virginia after 14 years of service. Figure B-2, adapted from their work, depicts chloride penetrations into two decks, both having high exposure to salt. Bridge A was salted approximately 660 times during its service period, and Bridge B was salted about 650 times, or an average of approximately 47 salt applications on each bridge per year. Chloride penetration at 3 in. below the surface was negligible for Bridge A and was approximately 0.5 to 1.25 lb/cu yd for Bridge B, depending on the sample location. According to the Virginia study (*5*), the concrete of Bridge A had a water absorption of 3.96 percent (average of two samples) of its weight compared to an absorption of 5.41 percent (average of two samples) for Bridge B. The study determined that the higher concrete absorption was the main reason for the higher chloride content in Bridge B. Interestingly, the calculated water/cement ratios of the ten bridges studied varied from 0.40 to 0.49 but did not necessarily agree with the values of absorption. The maximum water/cement ratio specified for Virginia bridge decks was 0.47 from 1963 through 1983 (*5*). The study also found that the difference between the chloride contents of the samples for each bridge was the result of the samples' environments. The samples

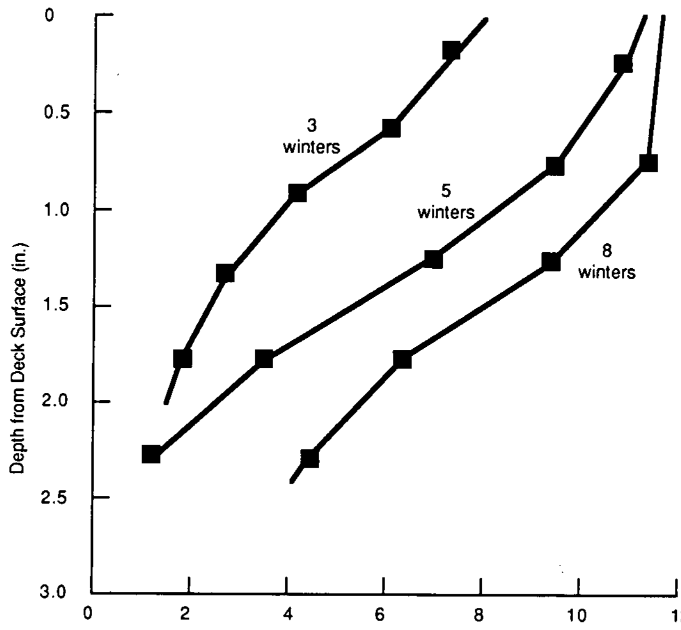

Figure B-1. Chloride content vs. depth for different winter exposures for a Michigan bridge deck built in 1969. (Adapted from Ref. 16)

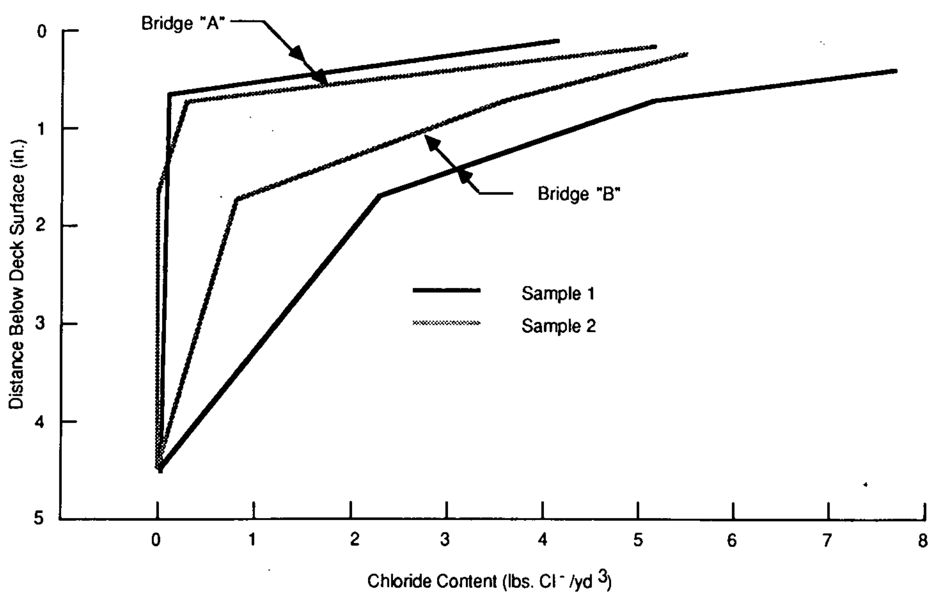

Figure B-2. Chloride content corrected for base level in two 14-year old concrete bridge decks in Virginia. (Adapted from Ref. 5 and assuming a concrete unit weight of 145 lb./cf)

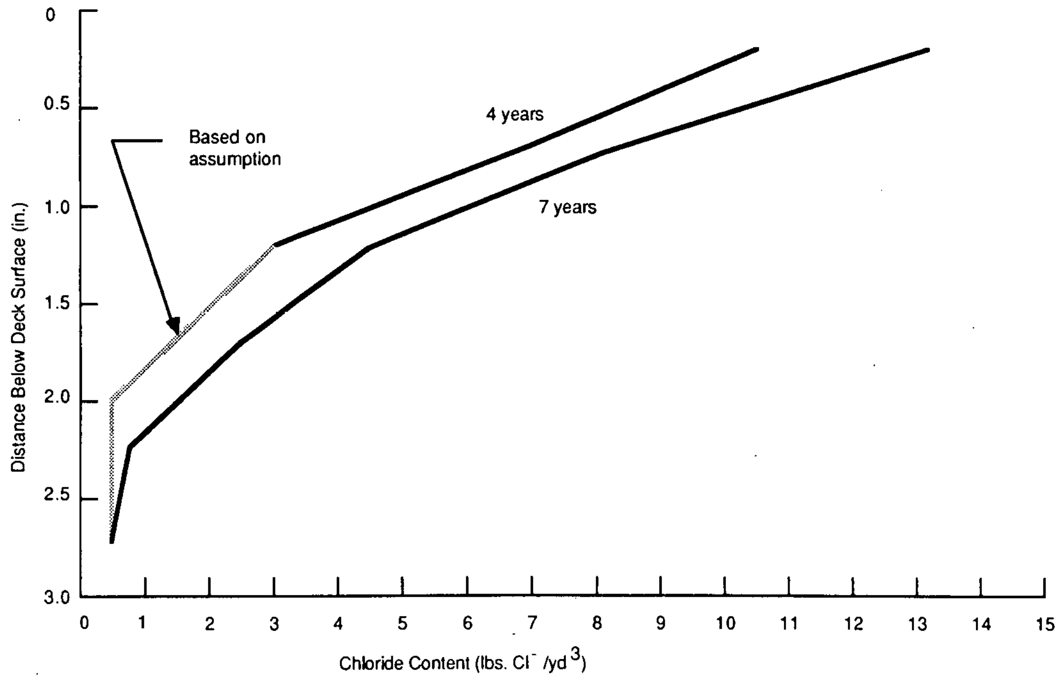

Figure B-3. Chloride content in a Minnesota bridge deck after 4 to 7 years of service. (Adapted from Refs. 13 and 14 and assuming a concrete unit weight of 145 lb/cf)

with higher chloride contents were closer to the curb-line or the bridge's drainage system.

Illustrated in Figure B-3 are average chloride contents in a bare Minnesota bridge deck after 4 and 7 years of service. The figure is adapted from information provided by Minnesota in 1978 and 1981 (*13,14*). The chloride content at 2 in. below the surface was approximately 1.5 lb/cu yd after 7 years of service. The maximum water/cement ratio allowed for bridge decks in Minnesota was 0.42, as reported in a 1977 survey of highway agencies (*7*).

A 1985 bridge deck study (*29*) in Pennsylvania reported the chloride contents of four bridge decks. The decks were 8 to 9 years old when they were tested. Figure B-4 shows how chloride content decreased with depth. Each curve in the figure represents the average of three samples taken in the driving lanes. At a depth of 1.5 in. below the surface and after 9 years of service, the chloride content was between 1.5 and 2.0 lb/cu yd. At 2 in. below the surface and after 9 years, the chloride content was small and only slightly more than the original concrete chloride content. In 1977, Pennsylvania reported a maximum water/cement ratio requirement of 0.47 (*7*).

Irwin and Chamberlin reported (*30*) the chloride accumulation rates for New York bridge decks constructed in 1975 and 1976 with cover depth requirements of 3.25 ± 0.25 in. The results are given in Figure B-5. After 4 to 5 years of service, the average accumulation of chlorides at 2 in. below the surface was approximately 1 lb/cu yd. The maximum water/cement ratio requirements for bridge decks in New York, 0.44, was reported in a survey conducted in 1977 (*7*). Chamberlin reported estimates of salt application rates for New York bridge decks from 1977 to 1979 (*31*). These estimates indicate that 50 percent of the bridges received approximately 8 to 13 tons of salt per lane-mile per year, or an average of about 10 tons. At the extreme, the bridges studied received a maximum of 43.2 and a minimum of 2.3 tons of salt per lane-mile per year. Assuming 650 lb of salt per lane-mile per application, the latter figures translate into approximately 30 (average), 133 (maximum), and 7 (minimum) salt applications per year.

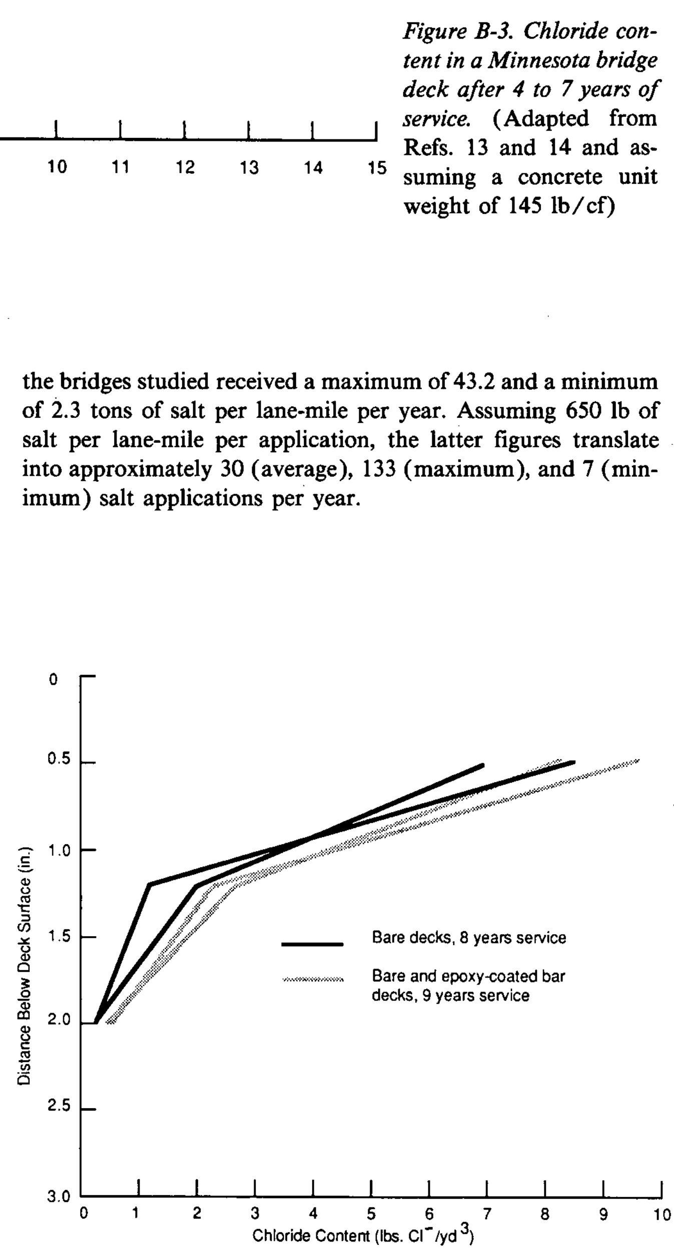

Figure B-4. Chloride content in four Pennsylvania bridge decks after 8 and 9 years of service. (Adapted from Ref. 29)

Discussion and Analysis

In order to determine how long a bridge deck will function free of chloride-induced corrosion and resulting deterioration, a criterion is needed for the condition at which distress initiates. The following criterion is suggested: "95 percent or more of the deck area should remain free of corrosion-induced distress for the effective service period of the deck."

Corrosion distress begins as delaminations in areas where the cover over the bars is the least, since the less the bar cover the easier for chlorides to reach the bar and cause corrosion. Thus the cover over the bar and the distribution of that cover are important factors governing the corrosion prevention characteristics of bare decks containing uncoated reinforcing steel.

Cover Depth Distribution

In an investigation of bridge deck bar cover depth in New York (*32*), Leslie reported that decks with a cover requirement of 3.25 ± 0.25 in. had an average cover of 3.3 in. This value is almost equal to the design target value. The data from that investigation indicate that 95 percent of the measurements were larger than approximately 2.65 in. or the design target value minus 0.60 in. O'Rourke and Ritchie (*33*) studied variations in cover depth in Michigan bridge decks. They reported that overall, average measured covers and design target values concurred. The distribution of cover depth was consistent, with approximately 95 percent of the measurements within a range of ± 0.75 in. around the average value, regardless of the actual design cover specifications. Statistically, this translates into 95 percent of the measurements being larger than the average value minus approximately 0.63 in. FHWA studies by Daveer (*34*) showed that the average cover depth and design target values were almost equal and that to obtain a minimum cover 90 percent of the time, design target values had to be 0.500 in. to 0.625 in. greater than the minimum desired value.

The foregoing discussion implies that 5 percent of the area will have cover depths smaller than the design target value minus approximately 0.65 in. Thus, in bare decks, if the design target is 3.5 in., 5 percent of the constructed deck area is likely to have a cover less than 2.85 in. (3.5 in. minus 0.65 in.) and if the design target is 4.0 in., 5 percent will have a cover less than 3.35 in. (4.0 in. minus 0.65 in.).

Effects of Cover Depth and Water/Cement Ratio on Time to Corrosion

Clear (*34*) established a relation between cover depth and the number of salt applications needed to induce corrosion by ponding test slabs daily to a depth of $\frac{1}{16}$ in. with a 3 percent sodium chloride solution. The amount of chloride applied each time was equivalent to 620 lb of salt per lane-mile. Figure B-6, taken from Clear, gives the resultant relation for three different water/cement ratios. There is a correlation between large cover depths, small water/cement ratios and prolonged lack of corrosion. The authors generated mathematical equations of best fit for those curves by applying regression analysis. The resultant equations are included as Table B-1. Equations were also generated for each 0.01 increment in water/cement ratio by interpolating between values plotted in Figure B-6. By inserting a bridge

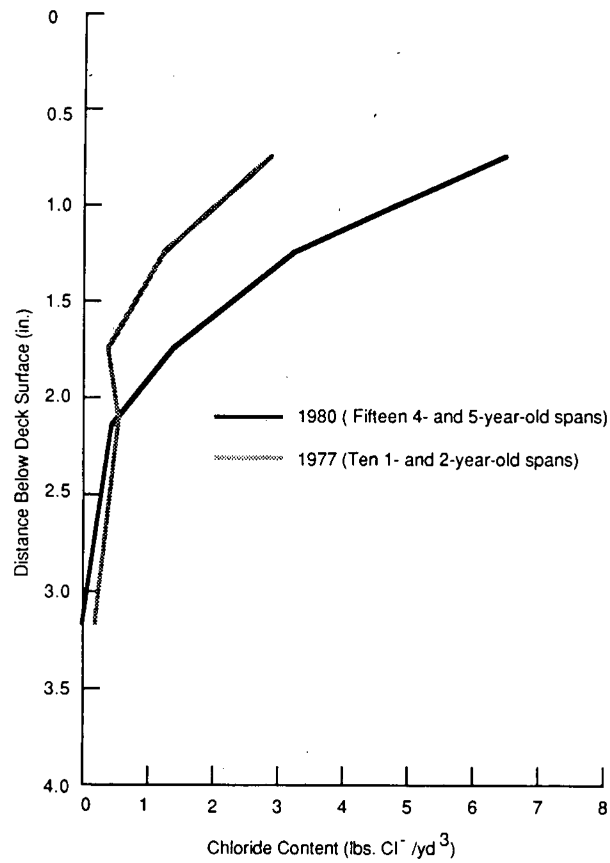

Figure B-5. Average chloride accumulation in selected New York bridge decks after 1–2 and 4–5 years of service. (Adapted from Ref. 30)

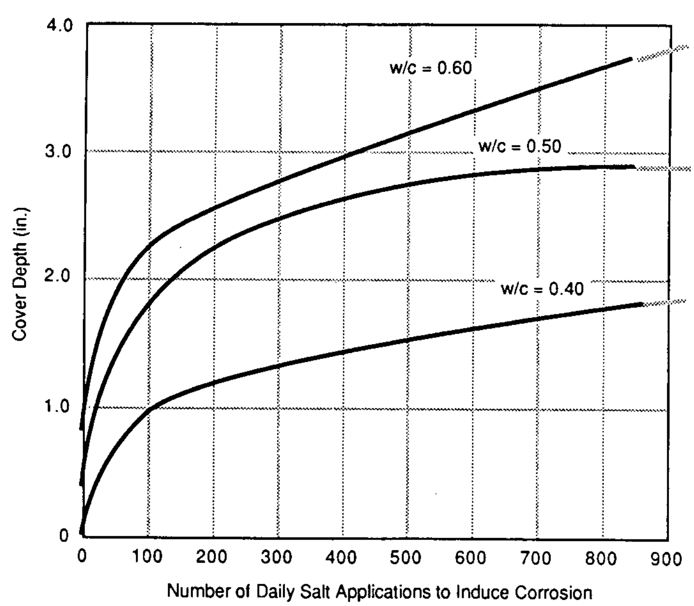

Figure B-6. Effects of water/cement ratio and cover depth on time to corrosion. (From Ref. 34)

Table B-1. Suggested mathematical relations between number of salt applications, corrosion of reinforcing steel in bridge deck, and cover thickness over steel (based on information provided in Ref. 34).

Water/ Cement Ratio	N = Number of salt applications to corrosion* D = Cover (in.)
0.60	$N = 6.83\ D^{3.566}$
0.53	$N = 10.80\ D^{3.648}$
0.52	$N = 11.60\ D^{3.665}$
0.51	$N = 12.47\ D^{3.676}$
0.50	$N = 13.40\ D^{3.689}$
0.49	$N = 16.29\ D^{3.661}$
0.48	$N = 20.73\ D^{3.585}$
0.47	$N = 25.49\ D^{3.541}$
0.46	$N = 31.73\ D^{3.495}$
0.45	$N = 39.47\ D^{3.447}$
0.44	$N = 51.10\ D^{3.357}$
0.43	$N = 64.99\ D^{3.292}$
0.42	$N = 83.00\ D^{3.229}$
0.40	$N = 141.85\ D^{3.037}$

* Ponding test slabs daily to a 1/16" depth with a 3% sodium chloride solution (equivalent application of 620 lbs. of salt per lane-mile).

deck's cover thickness into the equation corresponding to that deck's water/cement ratio, the number of salt applications that would induce corrosion in that deck's bars in the laboratory can be obtained. When that number is multiplied by the laboratory rate of salt application (i.e., 620 lb per lane-mile per application) the cumulative amount of salt that would induce corrosion in the bars is obtained. The effective service life of the deck can then be determined by knowing the average number of salt applications (or the average amount of salt) per year for the bridge and allowing for the normal amount of time it takes corrosion to cause deterioration in concrete. This approach assumes that for each design a certain amount of salt deposited or accumulated on the deck will initiate corrosion regardless of the frequency of its application during the service period. Three years is a reasonable amount of time for corrosion to cause delamination in bridge deck concrete when the concrete over the top mat is thicker than 2 in. (*35*). The service period determined by using Table B-1 may be adjusted linearly if the rate of salt application differs from 620 lb per lane-mile.

In determining the effective service period, the cover depth must correspond to the fifth percentile value. That depth equals the target design value minus 0.65 in. as explained earlier. The water/cement ratio should also be adjusted for the possibility that the specified water/cement ratio may not be reached or consolidation may not have been complete as a result of the numerous factors involved in construction. Newlon (*5*) reported that the calculated water/cement ratios for concretes, intended to be 0.47, varied within the range 0.40–0.51. A tolerance of +0.03 for the water/cement ratio seems reasonable (*36*) and, thus, the water/cement ratio associated with permeability should be taken as the specified value plus 0.03.

One important factor for employing the equations given in Table B-1 to determine the effective service period is the field equivalency for the number of test salt applications given by those equations. Clear (*34*) attempted to determine the field equivalency of the number of test salt applications. His limited experiments indicated that one test salting is equivalent to somewhat less than one field salting, when the same rate of salt application was used. However, he reached no definite conclusion. He hypothesized that the small difference was due to downward chloride ion migration into the concrete in its wet stage because of ions trying to achieve chemical equilibrium. This phenomenon may occur over time in wet, chloride contaminated concrete, regardless of further salt applications. Also, during laboratory ponding, the hydraulic pressure associated with tires that forces salt water into the concrete is absent. However, not every day in an average year provides moisture comparable to the daily wetness of the FHWA test slabs. If 50 days in an average year are assumed to provide the same amount of moisture in the concrete, 16 years of field conditions may provide the same condition for ion migration as 800 salt applications did on the test slabs. This time period can be even longer if one considers the effects of rainfall in flushing the chlorides from the deck surface.

Correlation Between Laboratory and Field Salt Applications

The authors studied correlations between the number of salt applications presented in Table B-1 and the number of field salt applications. If the corrosion threshold chloride content is assumed to be 1.5 lb/cu yd, a corresponding threshold cover thickness can be determined for a given chloride content profile. If that "threshold cover thickness" is used in the equations in Table B-1, the number of laboratory salt applications necessary to raise the chloride content for that depth to the threshold level is obtained and can be converted into a total amount of salt applied to the deck during its service period, using the laboratory rate of salt application of 620 lb per lane-mile. The figure obtained in this way is divided by the annual rate of salt application on the bridge to determine the service period. The latter figure is then compared to the actual service period.

Case I, Michigan Bridge (16)

Average chloride profiles are presented for a Michigan bridge in Figure B-1. "Threshold cover thicknesses" are 2.0 in. for three, and 2.2 in. for five, winter exposures. The actual construction water/cement ratio can be assumed to be the maximum specified water/cement ratio (i.e., 0.49) because the latter is relatively high. From Table B-1, for a water/cement ratio of 0.49 the number of salt applications is 206 for a 2-in. cover thickness and 292 for a 2.2-in. cover thickness. Those numbers correspond to 128 and 182 tons of salt application per two-lane mile, using 620 lb of salt per application per lane-mile. Given that the actual annual salt application was 51 tons per two-lane mile on the bridge, those chloride profiles predict 3 and 4 years of winter exposure, as compared to the 3 and 5 years reported.

If the actual construction water/cement ratio is assumed to be equal to the maximum design water/cement ratio of 0.49 plus a tolerance of 0.03 (i.e., water/cement ratio=0.52), the calculated service period using Table B-1 is 2 and 3 years, respectively, as compared to the 3 and 5 years reported.

Case II, Virginia Bridge (5)

In Figure B-2, sample 1 of Bridge B had the highest absorption among the 20 samples from the ten bridges studied. Its absorption was 6.10 percent, as compared to the lowest absorption of 3.43 percent for sample 2 of Bridge B. The calculated water/cement ratios for the ten bridges, intended to be 0.47, varied by as much as 0.40 to 0.49, but those values did not necessarily agree with the absorptions, indicating that the calculated values were not always representative of the actual water/cement ratios. The water/cement ratio representing the permeability of sample 1 of Bridge B may be assumed to be at least 0.49, consistent with its high absorption. The "threshold cover thickness" for sample 1 is 2.9 in. from Figure B-2, given that the original chloride content of the concrete was approximately equal to 0.16 lb/cu yd (0.004 percent by concrete weight (*5*)). For those values, Table B-1 gives the number of salt applications as 803, or 17 years of service (considering 47 salt applications per year and assuming 620 lb of salt per application per lane-mile for the bridge), as compared to the 14 years of actual service reported. Specified salt application rates on Virginia roads were reported in a 1974 NCHRP survey (*28*) to be 400 to 550 lb per lane-mile for temperatures below 10°F. The actual rates for bridges, however, can be higher. If a water/cement ratio of 0.50 (0.47 + 0.03) is used in Table B-1, the calculated service period is 14 years.

Case III, New York Bridges (30)

Average chloride content profiles are given in Figure B-5 for bridges tested in New York in 1977 and 1980. Leslie and Chamberlin estimated a mean construction water/cement ratio of 0.48 for decks built in New York (*37*). The "threshold cover thicknesses" for the chloride profiles in Figure B-5 are 1.1 in. for the 1- to 2-year-old spans and 1.7 in. for the 4- to 5-year-old spans. For a water/cement ratio of 0.48, the number of salt applications from Table B-1 is 29 for the 1- to 2-year-old spans and 139 for the 4- to 5-year-old spans. Those values correspond to 9 tons of salt (1- to 2-year-old spans) and 43 tons of salt (4- to 5-year-old spans) applied per lane-mile for 620 lb of salt per application per lane-mile. Because the average annual salt application on New York bridges was 10 tons per lane-mile, the calculated service period is 1 and 4 years for the two chloride profiles, as compared to the real service periods of 1 to 2 and 4 to 5 years, respectively.

Prediction of Service Life

The relatively good correlation demonstrated in the foregoing discussion suggests that the equations of Table B-1 can be used to predict the amount of the time between construction and the initiation of corrosion of the steel in concrete bridge decks, even though those equations are then extrapolated to ranges beyond those applied to the FHWA test slabs (*34*). Shown in Tables B-2 and B-3 are the corresponding predictions for the approximate number of years between construction and corrosion of the reinforcing steel for bridge decks having design target cover depths of 3.5 and 4.0 in. respectively. The cover depths used for preparation of these tables were the design target values minus 0.65 in., the fifth percentile cover depth. The tables are for the average figure of 620 lb of salt per application per lane-mile and for different numbers of annual salt applications and water/cement ratios. The water/cement ratios used for preparation of the tables were the maximum design values plus a tolerance of 0.03 to conservatively represent field conditions. The shaded areas in the tables indicate values where the amount of time until maximum acceptable corrosion is reached is longer than 47 years and the amount of time until maximum acceptable

Table B-2. Approximate years to maximum acceptable corrosion of reinforcing steel in bridge decks with design target bar depths of 3.5 in.[1]

W/C Maximum Design	W/C[2] Adjusted for Construction	Number of Annual Salt Applications							
		5	10	15	20	30	50	100	150
0.50	0.53	99	49	33	25	16	10	5	3
0.49	0.52	108	54	36	27	18	11	5	4
0.48	0.51	117	59	39	29	20	12	6	4
0.47	0.50	128	64	43	32	21	13	6	4
0.46	0.49	151	75	50	38	25	15	8	5
0.45	0.48	177	89	59	44	30	18	9	6
0.44	0.47	208	104	69	52	35	21	10	7
0.43	0.46	247	123	82	62	41	25	12	8
0.42	0.45	292	146	97	73	49	29	15	10

[1] The table is prepared for 620 lbs. of salt per application per lane-mile.

[2] Considering a tolerance of 0.03.

(shaded) Years to corrosion ≥ 47 years.

Table B-3. Approximate years to maximum acceptable corrosion of reinforcing steel in bridge decks with design target bar depths of 4.0 in.[1]

W/C Maximum Design	W/C[2] Adjusted for Construction	Number of Annual Salt Applications							
		5	10	15	20	30	50	100	150
0.50	0.53	178	89	59	44	30	18	9	6
0.49	0.52	195	97	65	49	32	19	10	6
0.48	0.51	212	106	71	53	35	21	11	7
0.47	0.50	232	116	77	58	39	23	12	8
0.46	0.49	272	136	91	68	45	27	14	9
0.45	0.48	316	158	105	79	53	32	16	11
0.44	0.47	369	184	123	92	61	37	18	12
0.43	0.46	434	217	145	109	72	43	22	14
0.42	0.45	509	255	170	127	85	51	25	17

[1] The table is prepared for 620 lbs. of salt per application per lane-mile.

[2] Considering a tolerance of 0.03.

(shaded) Years to corrosion ≥ 47 years.

concrete deterioration is reached is longer than 50 years. For salt application rates other than 620 lb per lane-mile, the number of years between construction and corrosion can be adjusted linearly in proportion to 620 divided by the alternate application rate.

Table B-2 suggests that for bridge decks constructed with design covers of 3.5 in., the effective service period (i.e., the number of years until deterioration affects 5 percent of the deck area) may be 50 years or more when salt exposures are less than 15 times per year (5 tons per lane-mile per year). For higher salt applications the water/cement ratio of the concrete is the determining factor. If the salt exposure is higher than 30 times per year (10 tons per lane-mile per year) the specified maximum water/cement ratio needs to be smaller than 0.42 to provide a 50-year or more effective service period. When salt applications reach 100 to 150 times per year (30 to 40 tons per lane-mile per year) the effective service period, even for the specified water/cement ratio of 0.42, may only be 10 to 15 years.

Table B-3 is the same as Table B-2 except that the design target bar depth is 4 in. In this case decks may have an effective service period of 50 years or more when salt exposures are less than 20 times per year (6 tons per lane-mile per year). Lower water/cement ratios will be required for higher salt applications. For salt exposures of more than 50 times per year (16 tons per lane-mile per year) the specified water/cement ratio needs to be smaller than 0.42 for a satisfactory condition. For extremely high salt applications such as 100 to 150 times per year (30 to 45 tons per lane-mile per year), the effective service period may only be 17 to 25 years, even for a 0.42 water/cement ratio.

BRIDGE DECKS WITH CONCRETE OVERLAYS

The chloride-proofing characteristics of concrete overlays are the same, regardless of whether the protective systems are on new or rehabilitated decks. Quantitative information regarding the chloride-proofing characteristics of concrete overlays has been obtained by determining the chloride content gradient through the overlays.

Performance History (Low-Slump Dense Concrete)

Iowa determined chloride content profiles in 15 low-slump dense concrete overlays in 1973 and again in 1978 (*12,38*). The chloride content profiles for each deck represented the average of test results at four locations. Shown in Figure B-7 are average chloride profiles for two Iowa bridge decks after 6 and 11 years of exposure to deicing salts. For both bridges chlorides at the surface were substantially higher after 11 years of exposure than after 6 years of exposure. However, at deeper locations the correlation between the increase in chloride content and the number of service years decreased, so that in both cases the correlation vanished at a depth of 1.75 in. from the surface. Interestingly, Bridge B had the highest chloride content of the 15 Iowa test bridges at a depth of 1.25 in. After 11 years of service, this chloride content was 1.5 lb/cu yd.

Figure B-8 gives the average chloride content profiles in 1- to 6-year-old low-slump dense concrete overlays installed in Minnesota in 1975 (*13*). Substantial chloride amounts had deposited in the surface concrete after only 2 years of service. At

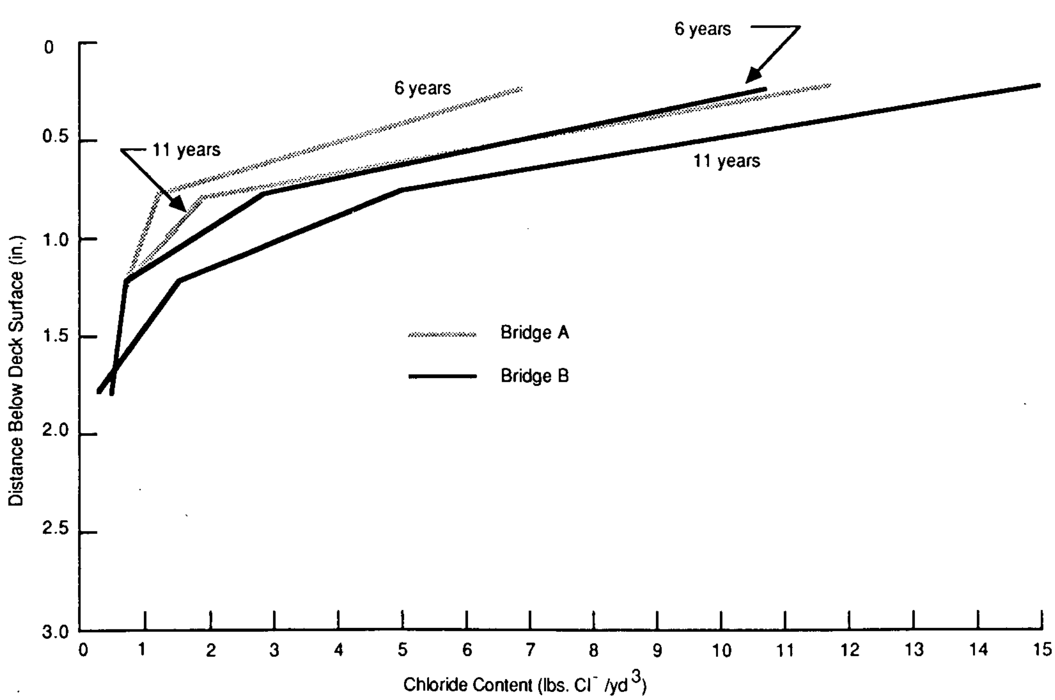

Figure B-7. Average chloride accumulation in two Iowa low-slump dense concrete sites. (Adapted from Refs. 12 and 38) a depth of 1.25 in. the chloride content increased gradually with years of service so that after 5 to 6 years of service the average chloride content at that depth was about 3 lb/cu yd.

Performance History (Latex-Modified Concrete)

Figure B-9 presents the results of monitoring the chloride content in a bridge deck in Michigan overlaid with a 1-in. latex-modified concrete at the time the bridge deck was placed in 1969 (*16*). Shown are the average chloride contents in the concrete for three, five and eight winter exposures. Although a large amount of chlorides existed in the surface of the deck after 8 years of exposure, the chlorides at a depth of 1.75 in. were below 1 lb/cu yd and only slightly above the chloride content corresponding to 3 years of exposure. On the other hand, a span of the bridge which had not been overlaid had a chloride content of about 6 lb/cu yd at the depth of 1.75 in. from the surface.

Curve	Years of Service	No. of Decks
A	1	16
B	2	12
C	3	9
D	4	12
E	5	12
F	6	11

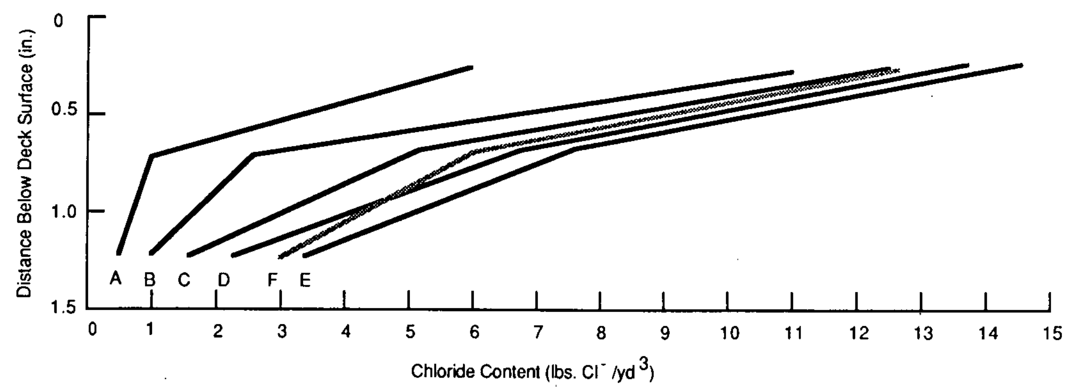

Figure B-8. Average chloride content in Minnesota low-slump dense concrete overlays with different years of service. (Adapted from Ref. 13 and assuming a concrete unit weight of 145 lb/cf)

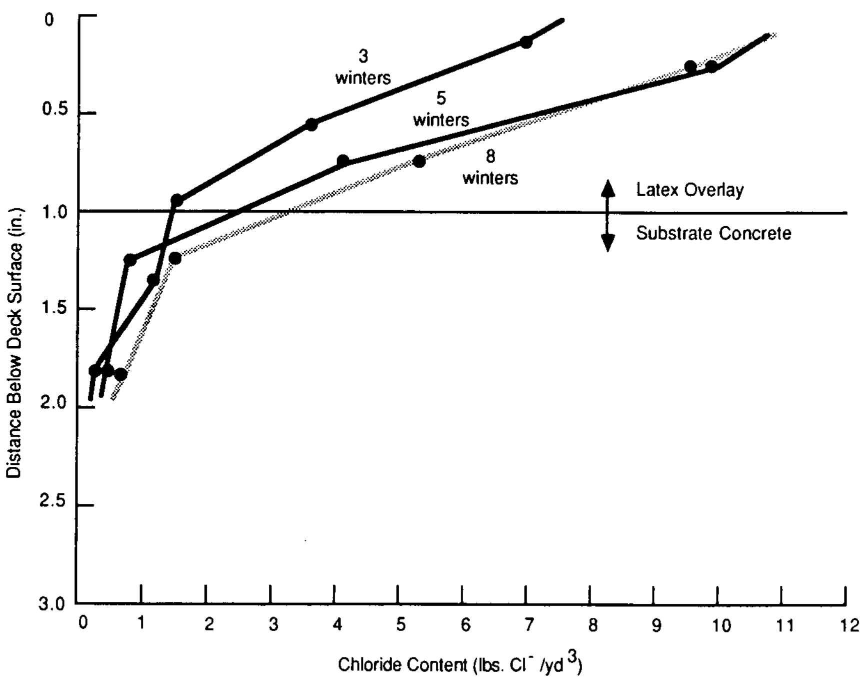

Figure B-9. Chloride content vs. depth for different winter exposures for a Michigan bridge deck placed and overlaid with latex-modified concrete in 1969. (Adapted from Ref. 16)

The average salt application on this bridge was reported to be 51 tons of salt per two-lane mile per year.

Figure B-10 gives the average chloride content in the concrete for 1- to 7-year-old Minnesota bridge decks overlaid with 1.25 in. of latex-modified concrete at the time the bridge decks were placed (*13*). As noted in the figure, the chloride content in the surface of the decks increased with the increase in years of service. After 7 years of exposure, the chlorides deposited in the surface concrete were about the same as those for the Michigan bridge after 8 years of exposure (see Fig. B-9). At a depth of 1.75 in. from the surface and after 7 years of service, the chloride content of a Minnesota bridge deck was approximately 1 lb/cu yd.

Discussion and Analysis

The performance of low-slump dense concrete overlays in Minnesota (Fig. B-8) indicates that considerable amounts of chloride can reach deep into the overlay after only 6 years of exposure. Therefore, the main issue is how long the chlorides will take to reach the corrosion threshold value at the level of the top bar mat. The specified maximum water/cement ratio for low-slump dense concrete overlays is generally 0.32 and the minimum thickness is usually 2 in. In contrast to the situation for bare decks, the specified minimum thickness of the overlay can be readily obtained since construction operations are relatively simple. The field permeability of these overlays, however, may represent the permeability of a concrete with a water/cement ratio of 0.40 or greater, rather than 0.32. The standard rapid chloride permeability tests conducted on cores extracted from Wisconsin field installations of low-slump dense concrete gave an electrical charge value of 1,770 coulombs, which is quantitatively closer to the charge passed through a concrete with a water/cement ratio of 0.40 (*39*) rather than 0.32. Rapid chloride permeability tests conducted on a Washington low-slump dense concrete overlay also showed unexpectedly large values of electrical charge. The average charge passed through four core samples was 2,401 coulombs, with a standard deviation of 359 coulombs. At the extreme, this translates into a water/cement ratio of 0.45. The cause of the higher permeability in low-slump concrete is probably the combination of unacknowledged water in the mix, the inadequate consolidation of the mix due to its stiff nature, and the nature and time of the curing stage. Curing time can be especially influential for overlays applied on existing bridges due to rapid construction policies. In the following section, two cases representing two combinations of bridge deck bar cover and overlay thickness are analyzed to estimate the amount of time between construction and corrosion of the bar when a low-slump dense concrete overlay is applied.

Case I

For this case the specified minimum thickness of the overlay is assumed to be 2 in. and the design target bar cover for the bare deck is assumed to be 1.5 in. The specified maximum water/cement ratios of the overlay and deck concrete are assumed to be 0.32 and 0.45.

The actual minimum overlay thickness will be at least equal to the specified minimum value of 2 in. The actual minimum

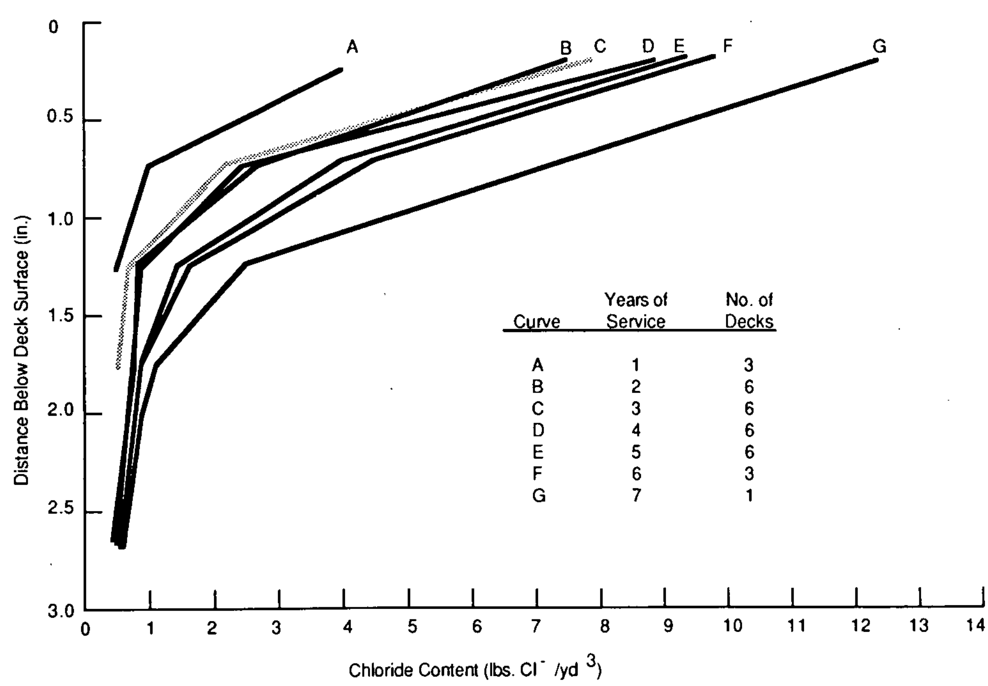

Figure B-10. Average chloride content vs. years of service for Minnesota bridge decks overlaid with $1\frac{1}{4}$-in. latex-modified concrete at the time of construction. (Adapted from Ref. 13 and assuming a concrete unit weight of 145 lb/cf)

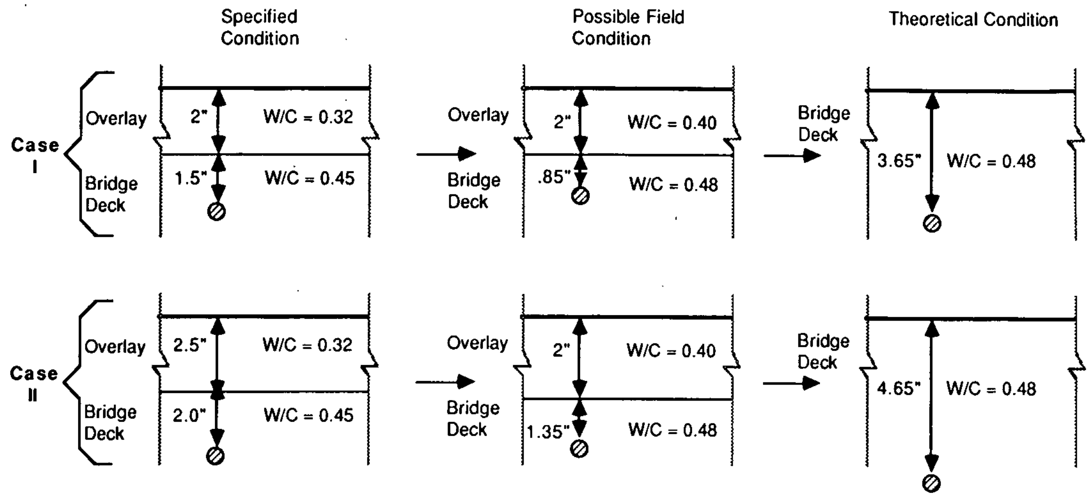

Figure B-11. Concrete water/cement ratio and thickness in bridge decks overlaid with low-slump dense concrete.

bar cover (fifth percentile cover) will be the design target value minus the tolerance of 0.65 in., as discussed previously, or 1.5 − 0.65 = 0.85 in. At best, the actual permeability of the overlay may equal the permeability of a concrete with a water/cement ratio of 0.40, as discussed earlier. The water/cement ratio of the bridge deck concrete can reach the specified maximum value plus 0.03, as discussed previously, or 0.45 + 0.03 = 0.48 in. The specified condition and the possible field condition for Case I are illustrated in Figure B-11. Figure B-11 also gives a theoretical condition for the bridge deck, as if the overlay had the same water/cement ratio as the bridge deck.

In order for the theoretical condition to represent field chloride-proofing characteristics, the thickness of the overlay needs to be adjusted. Clear found that with a 0.50 concrete water/cement ratio, about 1 in. more of cover was needed over the bar than with a 0.40 concrete water/cement ratio, if the amount of time to produce corrosion with exposure to salt was to be the same for both decks (see Fig. B-6) (*34*). Therefore a 2-in.-thick overlay with a 0.40 water/cement ratio is equivalent to a 3-in.-thick overlay with a 0.50 water/cement ratio, so far as the chloride-proofing characteristics are concerned. By interpolation, a 2-in.-thick overlay with a 0.40 water/cement ratio is approximately equal to a 2.8-in.-thick overlay with a 0.48 water/cement ratio. Therefore, the minimum thickness of the theoretical condition can be written as:

2in. (min. overlay) + 0.8 in. (adjusting overlay thickness for W/C = 0.48) + 0.85 in.
(min. bare deck) = 3.65 in.

When a cover of 3.65 in., a water/cement ratio of 0.48, and the appropriate equation of Table B-1 are used, the number of years between construction and the maximum acceptable corrosion of the bar for the Case I bridge deck design and different rates of salt application are obtained, as shown in Table B-4. The shaded squares in Table B-4 indicate values where the number of years between construction and the maximum acceptable corrosion is more than 47 and the maximum acceptable concrete deterioration is more than 50. Three years are assumed for development of delaminations because the total depth of cover exceeds 2 in.

Case II

As shown in Figure B-11, this case is the same as Case I, but with a specified minimum overlay thickness of 2.5 in. and a design target cover depth of 2 in. Table B-4 also presents for Case II the estimated number of years between construction and the maximum acceptable corrosion of the reinforcing steel for different rates of salt application. Those results were calculated using the same rationale as used for Case I.

Table B-4. Approximate years to maximum acceptable corrosion of reinforcing steel in bridge decks overlaid with low-slump dense and latex-modified concrete.[1]

Case	Number of Annual Salt Applications					
	30	40	50	70	100	150
I: Overlay: 2" LSDC or 1.5" LMC Bare Deck: 1.5" Target Cover & W/C = 0.45	72	54	43	31	22	14
II: Overlay: 2.5" LSDC or 1.75" LMC Bare Deck: 2.0" Target Cover & W/C = 0.45	170	128	102	73	51	34

[1] The table is prepared for 620 lbs. of salt per application per lane-mile.

Years to corrosion ≥ 47 years.

LSDC Low-Slump Dense Concrete

LMC Latex-Modified Concrete

Latex-Modified Versus Low-Slump Dense Concrete

In FHWA laboratory tests, research found, by determining the chloride content in the concrete, that a 1-in. cover of latex-modified concrete provided the same protection against chlorides as about 3 in. of concrete made with a 0.50 water/cement ratio (*34*). In another FHWA study rapid chloride permeability tests were conducted on laboratory-made latex-modified concrete, and permeability values were correlated with electrical charges as shown in Figure B-12 (*39*). The electricity charge that passed by 1 in. of laboratory-made latex-modified concrete was about the same as that for 3 in. of concrete with a 0.50 water/cement ratio. This result supports the earlier FHWA finding (*34*). Standard rapid chloride permeability tests conducted on 2-in-thick cores taken from a Washington bridge overlaid with latex-modified concrete gave the electrical charge of 469 coulombs for 2-in. overlays, 582 coulombs for 1.75-in. overlays, and 621 coulombs for 1.25-in. overlays. Since the charge for a 2-in., laboratory-made latex-modified concrete was not more than 1,000 coulombs, it is evident that the permeability of field-made latex-modified concrete is no more than the permeability of the same, laboratory-made concrete. Possibly that result is due to the latex particles making the mix more workable and, thus, eliminating the need for additional water to consolidate the concrete. For latex-modified concretes wet curing does not influence permeability as much as it does for low-slump dense concretes due to the partial curing effects present even in the field under dry conditions. In Figure B-12, the curves for W/C = 0.4 and latex can be said to represent possible field installations of low-slump dense and latex-modified concrete, respectively, and then according to these curves, 2.0 and 2.5 in. of low-slump dense concrete provide about the same chloride permeability as 1.5 and 1.75 in. of latex-modified concrete for field installations. Therefore, in Table B-4 1.5-in. and 1.75-in. thicknesses of latex-modified concrete are included in Cases I and II for the situation in which latex-modified concrete is overlaid on the deck instead of low-slump dense concrete.

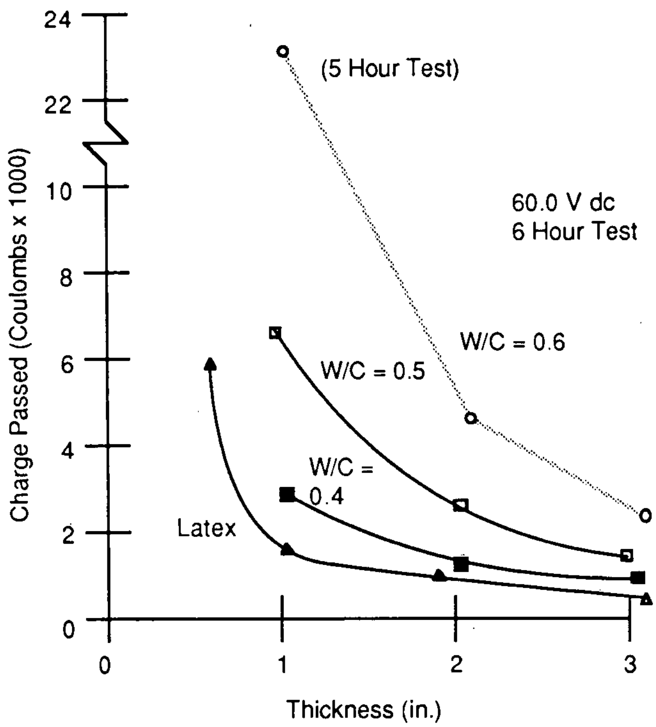

Figure B-12. Effect of core thickness on charge passed in rapid chloride permeability tests. (Adapted from Ref. 39)

Prediction of Service Life

Table B-4 suggests that for overlaid bridge decks the effective service period (i.e., the elapsed time until delaminations cover 5 percent of the deck area) may be at least 50 years when annual salt exposures do not exceed about 40 times (12 tons per lane-mile) for Case I and 70 times (22 tons per lane-mile) for Case II. For higher rates of salt application, adjustments in the thickness of the overlays, the depth of the bar cover, and the water/cement ratio of the deck's concrete are necessary to prolong the effective service period. Also, it has been reported (*5,40*) that use of an epoxy bonding compound instead of conventional grout prior to overlaying reduces chloride penetration. Remember, however, that for low-slump dense concrete, field permeabilities can be expected to be higher than the permeability used in analyzing Cases I and II. Thus, analyses may use a water/cement ratio of 0.45 rather than 0.40 to conservatively estimate the amount of time between construction and maximum acceptable corrosion.

Two-Stage Versus Monolithic Construction

Bridge decks protected by concrete overlays are constructed in two stages. Two-stage construction permits placement of more durable and more chloride-proof material on the surface of the bridge deck than one-stage or monolithic construction. The special concretes used in the second stage of two-stage construction cannot be used in monolithic construction either because of their high cost or their engineering properties. Overlays permit better final deck profiles, especially where profiles are affected by differential beam deflections created by concrete placement in the first stage of construction. A corrected profile permits better alignment and drainage (*40*). The unpredictability of beam deflections resulting from first stage construction dead-load effects and possible movement of the reinforcing mats during construction cause unexpected variations in the bar cover. When, however, an overlay is placed on a hardened deck the bar cover cannot be less than the minimum thickness of the overlay, and that thickness is readily controlled. Further, a bridge deck often requires overlaying after 20 to 25 years of service due to wear under traffic or other distress such as freeze-thaw scaling. Overlays have a negative feature. The deck surface may need to be scarified to a depth of ½ to ¼ in. before overlaying. In monolithic construction, the top mat steel may be damaged during that scarifying if its cover is only ½ to ¼ in., regardless of the specified cover thickness. However, this possibility should not be a problem, inasmuch as greater bar depths are currently specified for bare decks.

BRIDGE DECKS WITH INTERLAYER MEMBRANES

The chloride-proofing characteristics of interlayer membranes are best determined by collecting data on the chloride content of the substrate concrete. This includes waterproofed new decks as well as the rehabilitated portions of existing decks.

Performance History

Frascoia studied the field performance of various types of membrane systems applied on new bridge decks in Vermont (*41*). Performance was determined from average chloride samples obtained from substrates between 1971 and 1982. During the evaluation period the application of deicing salts ranged from 8.5 to 38.3 tons per two lane-mile per year, with an average of 29.5 tons. Table B-5, adapted from Frascoia's work (*41*), summarizes the chloride-proofing characteristics of the membrane systems studied and compares them with those of exposed bridge decks subject to the same environment. The membrane systems in Table B-5 are in rank order of decreasing performance, as determined by the percentage of concrete samples contaminated with chloride as well as the intensity of that contamination. The study considered samples to be contaminated when the chloride content of the concrete was 0.2 lb/cu yd more than the chloride content when the deck was constructed. As noted in the table, in decks waterproofed with tar emulsion, the worst ranked systems, 60 percent of the samples obtained from the top 1 in. of the concrete, were contaminated with an average chloride content of 0.65 lb/cu yd after 10 years of exposure. By comparison, 97 percent of the samples obtained from the top 1 in. of exposed decks had an average chloride content of 6.97 lb/cu yd after the same period of exposure. Standard preformed membranes performed best, with only 19 percent of the samples showing slight contamination with chlorides after 7 years of exposures. Thermoplastic membranes showed the second best performance, with only 17 percent of the samples showing slight contamination after 9 years.

Studies in Washington determined the chloride content in the rehabilitated concrete of four bridge decks 6 years after they were rehabilitated and protected with a thermoplastic membrane consisting of applied-in-place rubberized asphalt (*24*). Table B-6 gives the chloride content of the rehabilitated concrete for different concrete depths. Severe chloride leakage had occurred in only one of seven locations tested (i.e., 14 percent of the test sites). That leakage occurred where the asphaltic concrete overlay had cracked and deteriorated. For the other six locations, chloride contents were very small and below 0.36 lb/cu yd. The chloride content profiles suggest that those chlorides were the base concrete chloride contents and that no additional chlorides had penetrated into the concrete at those locations. The average pre-rehabilitation/protection chloride content of the four bridges tested was 3.92 lb/cu yd at a depth of 1.5 to 2 in. after an average exposure of 12 years.

Table B-5. Comparison of field performance of various types of membranes and exposed concrete decks in Vermont. (*Adapted from Ref. 41*)

Type of Membrane System	Average Winters Salted	Concrete Samples Contaminated (%)		Average Chloride in * Contaminated Samples (lb/c.y.)	
		0 - 1 in.	1 - 2 in.	0 - 1 in.	1 - 2 in.
Standard Preformed	7	19	7	0.50	0.34
Thermoplastic	9	17	8	0.84	0.52
Polyurethane	9	26	17	0.33	0.28
Epoxy	9	50	22	0.46	0.27
Tar Emulsion	10	60	35	0.65	0.49
Exposed Decks (bare)	10	97	82	6.97	3.55

* Assuming 250 ppm equal to 1 lb/c.y.

Table B-6. Post-rehabilitation/protection chloride content of patch concrete in Washington decks, 6 years after waterproofing with a rubberized asphalt interlayer membrane (*24*).

Bridge Number	$\bar{C}l$ Profile Depth (in.)	$\bar{C}l$ (lb/c.y.) corresponding to depth	
		First Sample	Second Sample
1	0 - 1/2 1/2 - 1 1 - 1-1/2	0.20 0.25 0.30	
2	0 - 1/2 1/2 - 1 1 - 1-1/2	0.25 0.26 0.20	0.19 0.24 0.35
3	0 - 1/2 1/2 - 1 1 - 1-1/2	0.32 0.21 0.20	 2.59
4	0 - 1/2 1/2 - 1 1 - 1-1/2 1-1/2 - 2	0.29 0.29 0.32 0.12	0.25 0.36 0.23

Discussion

Evaluations of the field performances of interlayer membranes in Vermont (*41*), Washington (*24*), Minnesota (*13*), and Kansas (*22*) indicate that certain types of membrane systems are effective in preventing chloride intrusion into the underlying concrete, although spot chloride leakage may occur, especially in the curb-line area where ponding is severe (*41,23*). Frascoia, using a statistical analysis of the 10-year performance of interlayer membranes in Vermont, estimated that the rate of increase in chloride contamination at a depth of 1 to 2 in. into the underlying concrete (generally the depth of cover for the uppermost bar, specified in conjunction with interlayer membranes) is less than 0.004 lb/cu yd (1 ppm) per year (*41*). Obviously, under this condition, concrete at a depth of 1 to 2 in. would take more than 50 years to be contaminated to the corrosion threshold level of 1.50 lb/cu yd.

Although the chloride-proofing abilities of some membrane installations seem to satisfy the 50-year criterion for the service life of a bridge deck, deterioration of the asphaltic concrete wearing course and aging of the membranes determine the actual performance. The effective service life of an asphaltic concrete wearing course is generally 10 to 15 years, depending on weathering and exposure to traffic. Factors such as overlay debonding and stripping, rutting in the wheeltracks, or lack of skid resistance may determine the service life. The condition of the membrane, however, does not necessarily depend on the condition of the wearing course. For example, if the wearing course deteriorates only through rutting, the membrane may still be satisfactory, depending on its age. Fifteen years of effective service may be expected of some membranes. Beyond this point membranes may deteriorate as a result of fatigue stresses caused by traffic. Membranes may also become brittle as they age.

Some asphaltic concrete systems can be repaired and replaced without disturbing the waterproofing membrane. For example, when the asphaltic concrete is at least 2 in. thick the upper portion can be removed by cold milling. With thinner pavement structures, or totally damaged asphaltic concrete/membrane systems, the entire pavement and membrane may need replacing. Following cold milling down to the concrete, the surface should be scrubbed by sand blasting or high pressure water to remove remnants of old membrane material and to obtain a clean surface.

With care in preparation, the surface may be treated similarly to a new PCC deck and the replacement membrane applied directly to the deck. In the case where the deck surface is too rough or irregular, a thin leveling course of asphaltic concrete may be required before the membrane is installed, so that the possibility of its puncturing is minimized. That course can also reduce the likelihood of blistering.

Aside from design features, the permeability of membranes depends greatly on construction practices and quality. Membranes are often damaged by the asphaltic concrete pavers and construction traffic. Some design features such as protection boards can prevent this type of damage, as well as possible puncture of the membrane by aggregate particles in the asphaltic concrete under traffic. Protection boards must have low absorption to prevent freeze-thaw damage and must be primed for satisfactory bond. Commonly, asphalt-impregnated protection boards ½-in. thick are used (*7*). It is important that the edges of membranes at curb locations be sealed to prevent possible leakage in that area of relatively higher chloride intrusion.

BRIDGE DECKS WITH EPOXY-COATED BARS

For decks constructed with epoxy-coated bars, the chloride content of the concrete surrounding the bar is not the appropriate criterion for determining corrosion-induced deterioration. The performance of this protective strategy is best judged by monitoring the physical condition of the concrete. However, knowledge about the level of chloride contamination is necessary in order to determine the effectiveness of the system when no corrosion-induced deterioration is present.

Performance History

Maryland monitored the performance of eight bridge decks, seven built with epoxy-coated bars and one built with conventional bars (*42*). The study included detecting deterioration in the concrete and measuring the level of chloride contamination. The bridges were 3 to 4 years old. None of the decks showed deterioration including the deck with the conventional bars. However, the chloride contamination of the decks was very small and below the corrosion threshold level. This was probably due to both the short service period and the use of high quality concrete to retard penetration of chlorides. The deck with the highest chloride content had only 2.49 and 1.29 lb/cu yd of chlorides at depths of 0 to 1.75 in. and 1.75 to 2.25 in., respectively, within the concrete. The latter was at the depth of the upper steel. Because of the low levels of chloride contamination, no conclusions can be drawn from this study regarding the effectiveness of the epoxy-coated bars.

Minnesota tested conditions for three bridge decks built with epoxy-coated bars (*13*). The decks were 4 to 7 years old and none showed deterioration in the concrete. The oldest deck was 7 years old and was the most contaminated. The average cover to its bars was 2.7 in., the smallest among the three decks. The chloride threshold depth for this deck after 7 years of exposure averaged 2.0 in., assuming a chloride threshold level of 1.5 lb/cu yd (Fig. B-3). If the fifth percentile corresponds to 0.65 in. of deviation from the design target cover of 3 in., less than 1 percent of the deck had cover depths smaller than the threshold depth of 2 in. Thus, probably less than 1 percent of the top mat in this deck was exposed to chlorides above the threshold chloride content and therefore no definite conclusions regarding the effectiveness of the protective strategy can be drawn from that study.

In 1984, Virginia evaluated the performance after 7 years of exposure of two bridge decks constructed with epoxy-coated bars (*43*). Bar level chloride samples obtained from random locations on each deck were between 0 and 0.58 lb/cu yd. Those levels are not sufficient to initiate corrosion. However, at one location on a transverse crack, the chloride content corresponding to a depth of 1 ¾ to 2 ¼ in. was 2.72 lb/cu yd, a value high enough to cause corrosion. Sounding the decks did not disclose any deterioration in the concrete, including the cracked location. The results of the Virginia study on the effectiveness of epoxy-coated bars were inconclusive because of the small chloride contamination for most of the deck areas.

Weyers evaluated the performances of 11 bridge decks constructed with epoxy-coated bars in Pennsylvania and compared them with the performances of the same number of bridge decks constructed with bare steel (*29*). Visual inspection of the 22 decks disclosed two decks with patches, one deck with spalls, and one deck with transverse cracking. All were built with bare steel. In-depth studies were conducted on two 9-year-old epoxy-coated steel decks and two 8-year-old bare steel decks. One of the bare steel decks was patched. Sounding of the four decks revealed deterioration (3 percent of the deck area) only for the patched bare steel deck. Average chloride contents for the four decks are shown in Figure B-4. Weyers found mean cover depths to be 2.29 and 2.41 in. for the two epoxy-coated steel decks and 2.18 and 2.19 in. for the two bare steel decks. The study estimated that 2.9 percent and 3.6 percent of the bars in the two epoxy-coated steel decks were shallower than the "threshold depth" (corresponding to a corrosion threshold chloride content of 1.2 lb/cu yd). For the two bare steel decks the figures were smaller, 0.6 percent and 0.9 percent, with the latter figure belonging to the deteriorated deck. Because the percentage of bars shallower than the "threshold depth" was more for the epoxy-coated decks than for the bare decks and the former did not show any sign of deterioration, these results suggest that epoxy coating may be an effective protection strategy.

Discussion

The literature is not conclusive regarding the effectiveness of epoxy coating to prevent the corrosion of bars subjected to chloride-contaminated concrete (*13,42,43*). In bridge decks exposed to salt, the bottom steel mat, located in chloride-free concrete, can act as the major cathode, and areas of the top mat in chloride-contaminated concrete can act as the anode. Since the top and bottom mats are electrically coupled by different metallic elements, such as tie wires, bar chairs, and expansion dams, corrosion current can flow. When the top mat is epoxy-coated, the dielectric nature of the epoxy prevents any electrical coupling between the two mats. However, field measurements of electrical resistance between the epoxy-coated top mat and the bare steel bottom mat in 17 bridge decks in Kentucky and Virginia showed that on eleven of the bridge decks some of the epoxy-coated bars were in electrical contact with the bottom mat steel, and that on two of the decks all the bars tested were in electrical contact (*44*). Therefore, the possibility of macrocorrosion of the coated top mat steel exists at local coating breaks caused by transportation or fabrication of the bar, by the concrete bearing against the bar's lug as the bar is stressed repeatedly adjacent to a crack by traffic and environmental loadings (*79*), or by the epoxy coating not totally isolating the steel from the chloride environment due to the existence of pinholes.

Attempts have been made to determine the state of corrosion of epoxy-coated bars in bridge decks. Because of the dielectric nature of the epoxy coating, lead wires are usually attached to certain bars in the top mat steel and half-cell potentials are detected along those bars only. Research conducted in FHWA laboratories has shown that although half-cell potentials in the range representing corrosion activity may be obtained on the epoxy-coated top mat surrounded by chloride-contaminated concrete, the amount of steel consumed, or rust production, is negligible (*44*). FHWA researchers Virmani and Clear (*44*) analyzed this condition and found that the reason for the low level of steel consumption was the high electrical resistance of the circuit between the two mats, caused by the presence of the epoxy coating. The high resistance of the circuit causes a small corrosion current to flow between the two mats and results in small steel consumption, even though the corrosion current density at local small coating breaks may be high. Virmani and Clear (*44*) estimated that for a given chloride exposure, the amount of top mat epoxy-coated steel consumed in 12 years would be the same as the amount of top mat bare steel consumed in one year. In other words, in environments exposed to salt, the life expectancy of bridge decks may be increased considerably by epoxy coating only the top mat steel. The FHWA tests included coating breaks larger than those usually caused during handling of the coated bars. No deep pitting at the bare areas was found in the tests. The life expectancy may be further increased when both mats are epoxy coated. In the latter case, because the major cathode (the bottom mat steel) does not receive oxygen, cathodic polarization occurs and results in a smaller difference between the cathode and anode half-cell potentials by shifting the cathode potentials toward the anode potentials. As a result, the corrosion current between the two mats, and consequently the metal consumption in the top mat, is further reduced. The FHWA researchers (*44*) showed that when both top and bottom steel mats were epoxy coated, and regardless of the presence of coating breaks, the amount of top metal consumed was reduced to $\frac{1}{46}$ of the amount consumed when neither the top nor bottom mat were coated.

Table B-7. Comparison of anticipated life of bridge decks built with damaged epoxy coating with those built with undamaged epoxy coating. (*Adapted from Ref. 47*)

	Type of Coating				
	Top Mat Only			Both Mats	
Degree of Damage (% of Area)	0%	0.24%	0.86%	0%	0.86%
Ratio of * Anticipated Life to Life of Undamaged Coating	1.00	1.00	0.29	1.00	0.76

* Based on years for corrosion to consume equal amount of steel

Epoxy coating of the top mat steel in bridge decks, combined with recent requirements on water/cement ratios and bar covers (generally 0.45 and 2.5 in., respectively) promises 50 years of corrosion-free life in chloride environments that caused corrosion-induced concrete deterioration in only 5 years in older bridge decks with design covers of 1.5 in. over bare steel. The 50-year service life can be further assured by epoxy coating both the top and bottom steel. A recent survey conducted by the Concrete Reinforcing Steel Institute showed that 17 states require both top and bottom mats to be epoxy-coated (*45*). This requirement, however, may apply only in certain environmental conditions. Some states also require epoxy-coated bars in other elements of bridge decks such as parapets and traffic barriers.

Damage and Breaks in Coating

In order to minimize corrosion of epoxy-coated bars damaged during their shipment or installation, FHWA recommends no more than 0.25 percent bare, unrepaired, damaged areas when only the top mat reinforcing steel in the deck is coated and no more than 2 percent when all reinforcing steel is coated (*46*). These figures, which generally apply to each 1-ft length of bar, are recommendations based on the FHWA's rate of corrosion tests, in which comparisons were made, as shown in Table B-7 (*47*), of the anticipated life of bridge decks built with epoxy-coated bars with different degrees of damage.

The maximum amount of damage detected on a job site immediately prior to installation of bars was 0.88 percent of the surface area for a 1-ft length for No. 11 bars and 1.08 percent for No. 5 bars (*44*). However, the overall average for the thirty No. 11 bars surveyed was 0.065 percent and for the six No. 5 bars 0.22 percent. Those bars had been coated and fabricated in North Carolina and shipped to Iowa via truck. In order to minimize damage to epoxy films, coated bars should be held together with nylon rope at the construction site, and bearing rollers and bending wheels should be covered with nylon during fabrication. The use of nonmetallic coated chairs and tie wires when only the top mat steel is epoxy coated is also necessary to minimize electrical coupling between the top and bottom mats (*46*).

Bond and Fatigue Considerations

A laboratory investigation of epoxy coatings conducted by the National Bureau of Standards under FHWA sponsorship indicated that certain epoxy coatings satisfied the requirements for bonding to concrete, and also satisfied the requirements for creep or long-term slippage of the bars in concrete under tensile stresses (*48*). Pullout tests showed that epoxy-coated bars had bond strength essentially equal to uncoated bars when the film thicknesses were approximately 10 mils or less. In general, the bond strength of the coated bars was about 6 percent less than that of uncoated bars, which is in the acceptable range. Evaluation of long-term slippage indicated that the epoxy coatings did not have a detrimental effect. Laboratory flexural fatigue tests on concrete beams have shown that epoxy-coated bars can sustain, without failure, two million cycles of a zero to maximum stress range that is 10 percent higher than the stress range for similar uncoated bars (*79,80,81*). In 1976, FHWA began specifying epoxy-coated reinforcing steel in bridge decks. Since that time many bridge decks have been built with this system. Ten or more years after states have employed the system, the bond of the coated bars with the concrete seems to be satisfactory and no fatigue failures have been reported.

Durability Concerns

The long-term durability of the epoxy coating in chloride-contaminated concrete environments may still be a concern. Laboratory specimens subjected to accelerated corrosion environments have only been tested for a few years, and the lives of field installations are generally limited to about 10 years. Two major elements that may contribute to deterioration of the coating and corrosion of bar are the presence of holidays in the coating (pinholes not detectable by the unaided eye) and weak adhesion of the coating to the bar. In view of this concern, the holiday and adhesion requirements specified by AASHTO M 284 or ASTM D 3963 are especially important. These requirements limit the average number of holidays to two per linear foot of the coated bar and necessitate evaluating the adhesion of the coating by bending coated bars around a mandrel. Of special importance also is proper bonding of any touch-up epoxy coating placed over damaged coating. There are indications that epoxy does not bond well to epoxy, thus increasing the possibility of a loss of bond between the concrete and the reinforcing steel. The durability of the coating may also be affected by bonding pressures on the coating near cracks. Traffic and environmental loadings may wear away coatings. This subject is discussed further and analyzed in Appendix C.

APPENDIX C

CONCRETE CRACKING IN BRIDGE DECKS

Cracking in concrete, although not considered a distress problem, may affect both the durability and corrosion prevention characteristics of bridge deck protective strategies. Cracking can cause internal concrete deterioration in the presence of freeze-thaw cycling through accumulation of water and salt in the cracks. Cracking can also cause reinforcing steel corrosion by allowing salt and moisture to move directly to the reinforcing steel when cracks reach the bar or its vicinity.

This appendix reviews the performance of bridge decks with regard to cracking and discusses the causes of cracking. It also suggests procedures for minimizing cracking and for repairing cracks. Finally, it discusses the contribution of cracking to bar corrosion. Some of the protective strategies considered in this appendix do not include overlays (i.e., increased depth of cover and epoxy-coated bar), while others do (i.e., low-slump dense and latex-modified concrete). Cracking in bridges overlaid with asphaltic concrete/interlayer membranes is not discussed because of the crack bridging characteristics of such membrane systems.

CRACKING IN BARE DECKS

Performance History

More than 1,000 randomly selected decks in eight states—California, Illinois, Michigan, Minnesota, New Jersey, Ohio, Texas, and Virginia—were surveyed beginning in 1961. The decks were built between 1940 and 1962 (*1*). About two-thirds of the spans surveyed had some form of cracking, with transverse cracking being the most frequent. Longitudinal, diagonal, and pattern cracking appeared with less frequency. The amount of transverse cracking increased with increases in the age of the spans and their lengths. Transverse cracking was also greater for continuous spans and was slightly greater for structural steel spans than reinforced concrete spans.

Newlon et al. (*2*) reported on cracking in 206 Virginia bridge deck spans surveyed in 1961 and 1970. Results are shown in Table C-1 (*2*). These spans were generally built between 1948 and 1961. Among the different types of cracking, transverse cracking was the most prevalent type and affected 23 percent of the spans in 1961 and 59 percent of the spans in 1970. However, most of the cracking, regardless of type, was light cracking classified as a few cracks per span and only a small percentage of the spans in 1970 had medium or heavy cracking of some kind. The absence of widespread cracking was attributed to the use of a $1^{11}/_{16}$-in. clear cover and a high proportion of simply supported spans. In the Virginia study the influence of traffic volumes and span lengths on cracking was also examined. Results are shown in Figures C-1 and C-2, respectively. Cracking was mainly associated with spans having high traffic volumes and long lengths. This trend was especially evident in the 1970 survey because the more heavily travelled and longer spans were also the younger spans (*2*). Of the 206 spans surveyed in 1970,

Table C-1. Distribution of cracks in 206 Virginia bridge deck spans. *(Adapted from Ref. 2)*

Type of Crack	Spans Cracked			
	1961 Condition		1970 Condition	
	Number	Percent-age	Number	Percent-age
All Cracks	63	30	155	75
Transverse	47	23	121	59
Longitudinal	7	3	30	15
Diagonal	4	2	9	4
Pattern	12	6	47	23
"D"	0	0	0	0
Random	9	4	105	51

Table C-2. Extent of transverse cracking in New York bridge decks. *(Adapted from Ref. 37)*

No. of Spans Surveyed	Age (years)	Design Cover Depth (in.)	% of Spans with magnitude of cracking: Smaller than 10 ft/1000 s.f. (1) (2)	Between 10 ft/1000 s.f. & 100 ft/1000 s.f. (3)	Larger than 100 ft/1000 s.f. (4)	% of Cracked Spans w/ Delams at Cracks
50[(a)]	1-2	2.0	94	6	0	0
50[(a)]	4-5	2.0	88	12	0	13
16	10-11	2.0	87	13	0	40

(1) Includes spans with no cracking
(a) Same spans surveyed twice

Definitions of magnitude of cracking

(2) Light – Less than 10 ft/ 1,000 s.f.
(3) Medium – 10 to 100 ft/ 1,000 s.f.
(4) Heavy – Greater than 100 ft/ 1,000 s.f.

only 20 spans had spalling. Of those 20, 14 had transverse cracking. However, no consistent relationship was found between spalling and transverse cracking.

Leslie and Chamberlin (*37*) studied transverse cracking in New York bridge decks of different ages and 2-in. bar cover depths. Table C-2 is based on the data obtained in their study (*37*). The table shows that most of the spans surveyed had either no cracking or less than 10 ft of cracking per 1,000 sq ft of span area (classified in this report as light cracking). Also, none of the spans had more than 100 ft of cracking per 1,000 sq ft

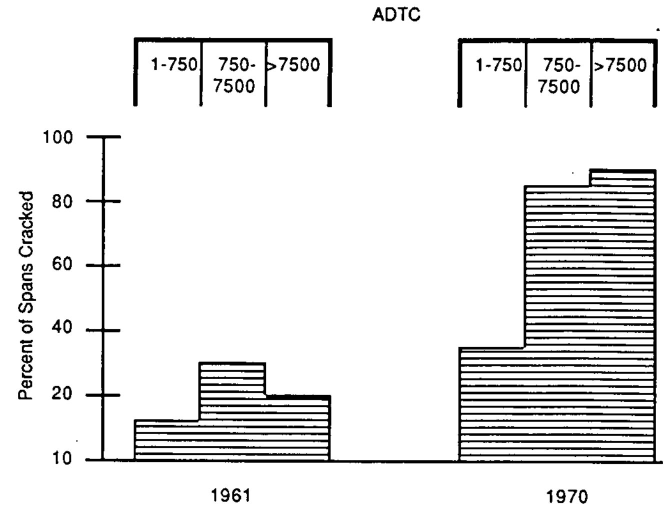

Figure C-1. Influence of traffic volume on occurrence of transverse cracking in 206 Virginia deck spans. (Adapted from Ref. 2)

of span area (classified in this report as heavy cracking). The percentage of spans with 10 to 100 ft of cracking per 1,000 sq ft of span area (classified in this reoprt as medium cracking) was twice as much in spans 4 to 5 years old as in spans 1 to 2 years old. However, medium cracking was not substantially greater for spans 10 to 11 years old and having the same bar depth. The coincidence of transverse cracking and delamination was more evident for spans that were 10 to 11 years old than those 4 to 5 years old. The New York study did not attribute the cracking to traffic, since medium cracking was detected in two of the three spans unopened to traffic and 4 to 5 years old.

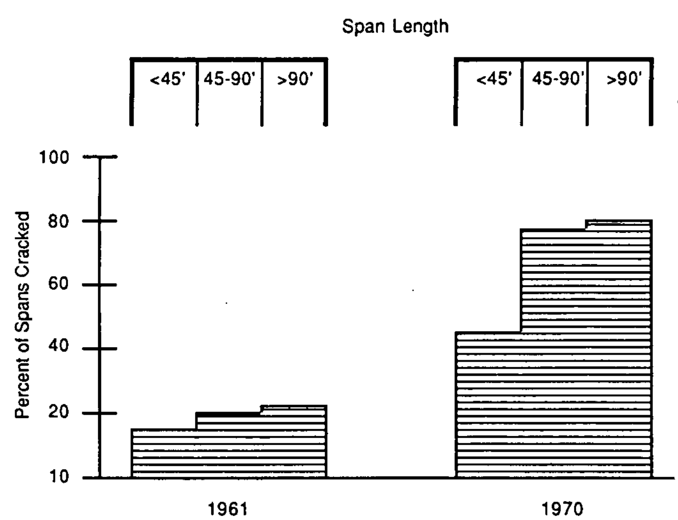

Figure C-2. Influence of span length on occurrence of transverse cracking in 206 Virginia bridge deck spans. (Adapted from Ref. 2)

New York researchers Irwin and Chamberlin (*30*) evaluated cracking in 15 bridge deck spans built in New York with 3¼-in. cover depth requirements. The spans were surveyed 1 to 2 and 4 to 5 years after their construction. Sixty percent of the spans did not show cracking of any type, but 40 percent showed medium or slightly more severe cracking (as classified in this report) in both surveys. In those surveys, most of the cracking was transverse and more extensive than in decks designed with 2-in. cover depths and of the same age (Table C-2).

Causes of Cracking

Concrete cracking in bridge decks is caused by many factors involving different stages of the decks' construction and service lives. In the following section these factors are analyzed and suggestions are made to prevent or minimize cracking.

Shrinkage of Plastic Concrete

A concrete surface, soon after its placement and in its plastic stage, may crack in a random, pattern, and/or transverse form because of shrinkage and restraints provided by the concrete mass. This type of cracking is usually shallow. Plastic shrinkage cracks appear when the evaporation rate exceeds the rate at which bleed water rises to the concrete surface. The rate of the evaporation at the concrete surface is high when (1) the concrete temperature is high, (2) the air temperature is high, (3) the air humidity is low, and (4) the wind velocity is high. ACI 305R-77 provides a chart for determining surface evaporation rates based on environmental conditions. If the rate of evaporation is expected to reach 0.2 $lb/ft^2/hr$ when the concrete is poured, either the construction should be delayed or precautions taken to minimize plastic shrinkage cracking by reducing the rate of surface evaporation. The more important precautions include placing the concrete when temperatures are low such as at night; erecting wind breakers; keeping the aggregate cool by shading them; using cold mixing water, possibly by incorporating ice; using fog (not spray) nozzles to maintain a sheen of moisture between the placement of concrete and the start of curing; and curing the concrete promptly after placement. Plastic shrinkage cracks that develop early may be repaired by a delayed floating operation (*49*).

Flexure in Plastic Concrete

Transverse cracks in plastic bridge deck concrete often form during construction over the supports of continuous unshored structures as a result of the concrete's dead weight causing negative moments. Hilsdorf and Lott (*50*) showed that between 2 and 4½ hours after mixing of the concrete, transverse cracks developed when a curvature of approximately 5×10^{-4} in.$^{-1}$ was applied to concrete placed in flexible formwork simulating deck slabs. The cracks were concentrated near the transverse reinforcing bars. The specimens in their work were 6 in. thick. According to the theory of bending

$$\rho = \frac{\epsilon}{Y}$$

where ρ = curvature, ϵ = extreme fiber strain, and Y = distance from neutral axis to extreme fiber.

Thus, thicker specimens require less curvature for the same strain that causes cracking in plastic concrete (*51*). Accordingly, for a 7½-in.-thick bridge deck, cracking in plastic concrete may develop under curvatures of 4×10^{-4} in.$^{-1}$. In order to reduce this type of cracking in continuous bridge decks, concrete should be placed first in the center of the spans. Once flexural cracking appears, revibration of the concrete surface before its penetration resistance exceeds 60 psi (ASTM C403-68), or approximately ½ to 1 hour prior to the initial set, may repair the cracking (*50*).

Settlement of Plastic Concrete

Fresh concrete settles after finishing and during the bleeding period. Horizontal reinforcement (or ducts) in the deck resist subsidence and cause cracking over and parallel to the reinforcement or duct. Cracks of this nature are usually transverse on decks supported by longitudinal girders and longitudinal on concrete slab bridges, according to the orientation of the uppermost bar. The possibility of subsidence cracking increases with a decrease in cover depth, an increase in concrete slump, and an increase in bar size. Dakhil and Cady (*52*) quantified the effects of pertinent factors on the likelihood of subsidence cracks, as shown in Table C-3. For cover depths of 2 in. or more and bar sizes No. 5 or less, the probability of subsidence cracking is substantially reduced by using concrete slumps less than 4 in. In order for at least 95 percent of the deck steel to have a cover of 2 in. or more, the design value for the cover should be 2.65 in. (see Appen. B for "Bridge Decks with Depth Cover ≥ 3 in."). Specification of the latter value for cover can mitigate subsidence cracking in decks with uppermost bars no larger than No. 5 (usually decks supported by longitudinal girders). As is the case for flexural cracks in the plastic concrete, revibration of the concrete surface may be effective for repairing subsidence cracks (*50*).

Table C-3. Probability of subsidence cracking predicted by regression equation. (*Ref. 52*)

	Probability of Cracking (%)*								
Slump (in.)	2			3			4		
Bar Size / Cover (in.)	#4	#5	#6	#4	#5	#6	#4	#5	#6
3/4	80.4	87.8	92.5	91.9	98.7	100.0	100.0	100.0	100.0
1	60.0	71.0	78.1	73.0	83.4	89.9	85.2	94.7	100.0
1-1/2	18.6	34.5	45.6	31.1	47.7	58.9	44.2	61.1	72.0
2	0.0	1.8	14.1	4.9	12.7	26.3	5.1	24.7	39.0

* Computed probability values of less than 0 percent or greater than 100 percent are reported as 0 percent or 100 percent, respectively.

Drying Shrinkage in Concrete

Bridge deck concrete on exposure to the atmosphere loses some of its original water to the environment and shrinks. The drying shrinkage of plain concrete ranges from about 400 to 800×10^{-6} at 50 percent relative humidity, while its tensile strain capacity can be as low as 200×10^{-6} (*11*). In a deck's early stages, loss of moisture is greater from the surface than from the interior of the concrete. Therefore, differential shrinkage effects develop as the interior restrains the surface and random, pattern, and/or transverse cracking can result. As drying progresses, the likelihood of transverse cracking increases because slabs are generally much longer than they are wide, and the longitudinal girders present in many bridges also restrain shrinkage. The greatest restraint occurs with steel girders. The least restraint is with cast-in-place concrete girders because those girders shrink at almost the same rate as the deck (*1*). The restraint provided by precast, prestressed concrete girders is intermediate between those two effects. The shrinkage of such girders is less than with cast-in-place girders because the precast girders are usually steam-cured and have then shrunk between fabrication and use (*49*). The topmost reinforcement in bridge decks restrains shrinkage and causes shrinkage cracks to be evenly distributed and of relatively small width.

Cyclic wetting and drying causes alternate swelling and shrinkage, which further contributes to the cracking of bridge decks. A decrease in temperature also makes concrete shrink. The coefficient of thermal expansion in unrestrained concrete is about 5×10^{-6} in./in. °F. For bridge decks, however, cooling generally occurs during the season in which concrete reaches its maximum moisture content. Therefore, for mature concrete, shrinkage due to cooling generally offsets that due to drying (*53*).

The total water content of concrete is the major factor affecting its drying shrinkage. Lowering the water content of the mix reduces shrinkage. A 40-lb reduction in the water content of 1 cu yd of concrete reduces the drying shrinkage by about 15 percent (*49*). Any procedure that reduces the water content of a concrete, such as using a smaller slump, a larger coarse aggregate, a higher proportion of aggregate in the mix, or a water reducing agent, or concreting at lower temperatures, reduces drying shrinkage and consequent cracking. The types of cement and aggregate also affect drying shrinkage. Concretes made with Type II cements generally shrink less than those made with Type I and much less than those made with Type III cements (*49*). Excessive deleterious materials, such as clay, in the fine aggregate can increase drying shrinkage. Washing deleterious materials out of the fine aggregate reduces cracking (*5*). The fine aggregate should have a sand equivalent value above 80 percent (ACI 224R-80). Also, less shrinkage can generally be obtained by using aggregates with a high modulus of elasticity and low absorption. The creep properties of concrete can be utilized to mitigate cracking due to drying shrinkage in a concrete's early stages. Preventing the surface from drying quickly by allowing the wet cure cover to remain in place for several days after the specified curing period until it is dry will take advantage of the concrete's creep properties (*49*).

To counteract shrinkage of bridge decks, control crack widths and distribute cracks uniformly, a minimum amount of reinforcement is necessary in both directions near the exposed surfaces of bridge decks. According to AASHTO Specifications for Highway Bridges Section 8.20, the total area of reinforcement provided should be at least 0.125 sq in. per ft, with spacing no more than three times the slab thickness, or 18 in. In bridge decks supported by longitudinal girders, the usual practice is to place the longitudinal shrinkage reinforcement under the primary transverse reinforcement so that the transverse moment capacity is a maximum. In such bridges failure due to transverse bending is unlikely. Logically, the width of transverse cracks will be better controlled when the longitudinal reinforcement is placed on top of the transverse bars and therefore closer to the deck's surface. The performance of decks with 3¼-in. design cover depth suggests that cracking is more extensive when shrinkage reinforcement is embedded deep in the concrete. Also, in determining the desirable amount of shrinkage reinforcement, the influence of factors such as slab dimensions, the location of joints, and the degree of restraint provided by the supporting system need to be considered.

Flexure Under Service Conditions

Continuous concrete bridge decks crack in negative moment regions due to flexural stresses caused by dead and live loadings. Cracking of this nature occurs mainly in areas over internal supports. The width of the flexural cracks at the concrete surface may be controlled through proper design. Flexural crack widths can be minimized by properly distributing the reinforcing steel in tension zones, decreasing the stress in that steel, and decreasing the cover depth. Gergely and Lutz (*54, 55*) found that the maximum crack width at the surface could be predicted with the following equation:

$$Z = F_s \sqrt[3]{d_c A}$$

in which Z = W/0.091 in kips per inch and W is the crack width at the surface in thousandth inches; F_s = tensile stress in steel at the load, for which crack width is determined in ksi (F_s may be approximated as 0.6 times the yield strength of the reinforcement under service); d_c = the thickness of the concrete cover measured from the extreme tension fiber to the center of the bar closest to that fiber; and A = the effective concrete tension area surrounding the tension reinforcement and having the same centroid, divided by the number of bars. According to AASHTO Specification 8.16.8.4, when the reinforcement consists of several bar sizes, the number of bars should be determined as the total area of reinforcement divided by the area of the largest bar used.

When decks act as the flanges of longitudinal girders, as in T-girder and box-girder bridges, flexural tension reinforcement should be distributed over an effective tension flange width equal to one-tenth of the span length or effective compression flange width, whichever is smaller (AASHTO 8.17.2.1). That distribution reduces the possibility of excessively wide cracks in the deck adjacent to the girder web and at the same time places the reinforcement in as much tension as possible. If the spacing of the girders exceeds the effective width, additional reinforcement, with an area at least equal to 0.4 percent of the excess slab area, needs to be provided in the excess portions of the slab (AASHTO 8.17.2.1). For composite steel-girder bridges, in negative moment regions, the area of longitudinal reinforcement should be at least 1 percent of the cross-sectional area of the deck. Two-thirds of this reinforcement should be placed within the effective flange width (AASHTO 10.38.4.3).

Table C-4. Tolerable crack widths for reinforced concrete (ACI 224 R-80).

Exposure Condition	Tolerable crack width (in.)
Dry air or protective membrane	0.016
Humidity, moist air, soil	0.012
Deicing chemicals	0.007
Seawater and seawater spray; wetting and drying	0.006
Water retaining structures *	0.004

* Excluding nonpressure pipes

Table C-5. AASHTO 8.9.2-recommended minimum thickness for constant depth members for highway bridges.*

Structure Type	Minimum ** Thickness (ft.)
Bridge slabs with main reinforcement parallel or perpendicular to traffic	$\frac{S+10}{30} \geq 0.542$
T-Girders	$\frac{S+9}{18}$
Box-Girders	$\frac{S+10}{20}$

* When variable depth members are used, table values may be adjusted to account for change in relative stiffness of positive and negative moment sections.

** Recommended values for continuous spans: simple spans should have about 10 % greater thickness.

S = span length [as defined in Article 8.8, in feet]

According to AASHTO Specification 8.16.8.4, the quantity Z shall not exceed 170 kips per in. for members in moderate exposure conditions and 130 kips per in. for members in severe exposure conditions. The latter values of Z correspond to crack widths 0.015 and 0.012 in., respectively. ACI 224R-80 presents Table C-4 as a guide for tolerable crack widths. However, for protection from corrosion the choice between a small crack width or large cover thickness is left to the judgment of the designer. When members are exposed to deicer chemicals, AASHTO 8.16.8.4 requires a protective measure in addition to the requirement on crack widths.

For unshored construction, engineers may try to minimize the occurrence of flexural transverse cracks in areas over supports by first placing concrete in the center portions of the adjacent spans and then over the supports, thus minimizing the effects of dead loads. However, an evaluation of cracking in continuous bridges in Virginia (*43*) found that the sequence in which the deck concrete was placed had no discernible effect on transverse cracking of the deck. Transverse cracking often occurred in positive moment regions in the central portion of continuous spans with a severity equal to that at the supports.

Repeated Deflection and Traffic-Induced Vibrations

Bridge decks can be subject to repeated deflection reversals as a result of traffic loadings. The total live load deflection at any point on a deck is the sum of the static and dynamic deflection. The static deflection is the portion of the total deflection that would occur if the speed of the moving vehicle was close to zero. Dynamic deflections, or vibrations, are the result of disturbances in the vehicle caused by its speed while passing over irregularities on the deck surface such as expansion joints (*51*) and the bridge deflecting in real time to the vehicle moving over it. The magnitude of the latter component depends primarily on the natural frequency of the bridge and the characteristics of the frequency of the bridge and the characteristics of the vehicle moving over it. A typical relation between deflection and vehicle position for a continuous span is shown in Figure C-3 (*56, 51*). Repeated deflection reversals occur at position B both when the vehicle is on the bridge and when it has left the bridge. In time, such a phenomenon can cause cracking or make the existing cracks, especially any transverse cracks, deeper and wider. The abrasion action caused by crack movement in conjunction with water infiltration can deteriorate and widen cracks (*57*). The magnitude of the curvature caused by service conditions also influences crack propagation. Curvature increases with increase in deflection, but decreases with an increase in span length. However, increases in span length also increase deflection. AASHTO's recommended depth limitations for superstructure members to control deflections are given in Table C-5 as a function of span length.

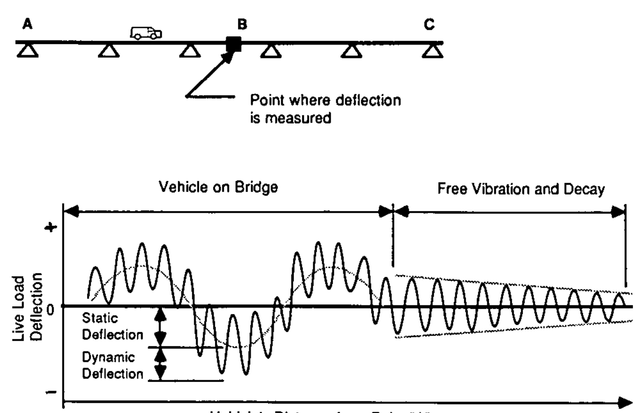

Figure C-3. Typical deflection-vehicle position relation for a 5-span continuous bridge. (Adapted from Refs. 56, 51)

The dynamic effects of traffic-induced vibrations may also cause cracks to lengthen or to deepen. The peak particle velocity of bridge decks has been suggested as an indicator of the risk of cracking from the dynamic effects of traffic-induced vibrations (*51*). The peak particle velocity can be determined from the equation:

$$V = 2\pi a f$$

in which V = peak particle velocity caused by vibration, a = amplitude of vibration, and f = frequency of vibration.

Although amplitudes increase as the flexibility of the structure increases, frequencies also increase as the stiffness of the structure increases. The stiffness increases as the span decreases or the flexural rigidity, *EI*, increases. Researchers have suggested that the peak particle velocity in concrete 7 days or more old should be limited to 4 in. per sec to control cracking from blasting operations (*58, 51*). The results of tests by the Transport and Road Research Laboratory in England (*59, 51*) on eight bridges showed that peak particle velocity ranged from 0.12 in. per sec to 0.87 in. per sec, which was well below the recommended limit for hardened concrete. The authors analyzed those test results. Interestingly, the factor that most influenced the peak particle velocity was the amplitude, or dynamic deflection, rather than the frequency. While the maximum dynamic deflections of the eight test bridges ranged from 0.004 in. to 0.039 in., the frequencies ranged from 2.3 Hz to 5.6 Hz. The maximum dynamic deflection generally increased as the maximum static deflection increased. Excessive flexibility of bridges may result in higher dynamic deflections while their frequency may not be reduced markedly. Thus, the net effect of increasing flexibility can be an increase in both dynamic deflections and peak particle velocities. Although the recommended threshold peak particle velocity does not seem to suggest that vibrations in bridge decks can cause cracking, repeated vibrations undoubtedly decrease the effective stiffness of the structure and permit incipient cracks, especially those at vertical planes of weakness, such as directly above and parallel to the top transverse reinforcing steel, to develop fully. That scenario helps explain the higher incidence of cracking in older bridges, in bridges carrying high traffic volumes, fast moving traffic, and in bridges with long span lengths.

CRACKING IN CONCRETE OVERLAYS

Performance History (Low-Slump Dense Concrete)

Cracking in low-slump dense concrete overlays was investigated in Minnesota (*13*) and Missouri (*15*). Data from the Minnesota investigation show that cracking in the seven 2-course new construction decks tested was negligible and was less than 10 ft per 1,000 sq ft of surface area. The Minnesota decks were 4 to 6 years old, and the types of cracking were mainly transverse and random. Data from the Missouri bridges, although they were somewhat younger, indicate relatively higher degrees of cracking in some overlays. In the fifty-eight 0- to 5-year-old, 2-course new construction bridge decks tested in Missouri, 40 percent of the overlays had less than 10 ft, 45 percent had 10 to 100 ft, and 15 percent had greater than 100 ft of cracks per 1,000 sq ft of surface area. Compared with the rehabilitated/overlaid decks, 2-course new construction decks showed a 73 percent reduction in surface cracking. Data from the Minnesota investigation (*13*) also indicated a lower rate of cracking for the 2-course new construction decks than for the rehabilitated/overlaid decks.

Performance History (Latex-Modified Concrete)

Cracking in latex-modified concrete overlays was reported in the Minnesota (*13*) and Missouri (*15*) investigations. In the six 5- to 7-year-old, 2-course new construction bridge decks tested in Minnesota, 17 percent of the overlays had less than 10 ft, 66 percent had 10 to 100 ft, and 17 percent had greater than 100 ft of cracks per thousand sq ft of surface area. These cracks were mainly transverse and random. In the twenty-two 3- to 7-year-old 2-course new construction bridge decks tested in Missouri, the percentages of decks corresponding to the crack intensity classification were 18, 73, and 9 percent, respectively.

A survey of conditions in 23 two-course new construction decks in Ohio, Michigan, Kentucky and West Virginia (*16*) showed that only 23 percent of the decks (five decks) had cracking. The cracking was random and transverse and covered from 5 to 50 percent of the decks' areas. That value corresponds to between 15 and 175 ft of cracking per 1,000 sq ft of surface area, using the multiplier of approximately 3.5 developed in this research by a regression analysis of the results presented in the Missouri investigation (*15*).

Causes of Cracking in Concrete Overlays

The main factors contributing to cracking in protective concrete overlays placed on bare decks are the following: shrinkage in the plastic concrete, drying shrinkage of the hardened concrete, flexure under service conditions, and repeated deflections and traffic induced vibrations.

Plastic shrinkage is the major contributing factor to cracking in overlays. Typically both low-slump dense and latex-modified concrete overlays have water/cement ratios of about 0.32, and therefore only limited bleed water rises to the concrete surface. Because surface evaporation rates are likely to be higher than bleed rates, random, pattern and/or transverse cracking results. The Washington State Department of Transportation specifies that the maximum allowable evaporation rate at the surface of plastic latex-modified concrete be from 0.15 to 0.20 $lb/ft^2/hr$ and that the concrete be covered with wet burlap as soon as possible after placement. Under certain conditions, the length of uncovered plastic concrete behind the screed may be limited to 10 to 20 ft, which translates into approximately 10 to 20 min of exposure. Precautions to minimize plastic shrinkage cracking, as discussed for bare decks, are necessary when critical evaporation rates develop.

Unlike plastic shrinkage, drying shrinkage in a low-slump dense concrete overlay may not be a major problem because the amount of mix water is very small. However, the final shrinkage of latex-modified concrete is relatively high and is of the order 800×10^{-6} (*16*). In two-course new construction bridge decks, the newly constructed underlying decks may shrink due to drying at about the same rate as the overlays. This may explain the relatively smaller incidence of cracking in low-slump dense concrete overlays than in latex-modified concrete overlays and in overlays placed on newly constructed decks than in those placed on existing decks. The possibility of long-term cracking in latex-modified concrete can be reduced by thoroughly soaking the substrate prior to placing the overlay so that the substrate can shrink at a higher rate as it dries. Prolonging the wet curing period and allowing the wet cure cover to remain several days without wetting after the specified wet curing period is over may also minimize the incidence of cracking.

In concrete overlays placed on continuous spans transverse cracking occurs mainly due to the negative moments caused by live loading. Thus the extent of flexural cracking in overlays is normally less than in bare decks. However, the enlargement of existing cracks in overlays, initiated by various causes, or the creation of new cracks can be expected with time because of vibrations and repeated flexing of the deck under travelling vehicles. In this regard, one advantage of overlaying decks with concrete is the resulting lower flexibility. Bishara investigated the relation between cracking in latex-modified concrete overlays and their exposure in 92 installations in Ohio, Michigan, Kentucky, and West Virginia (Fig. C-4) (*16*). The cracking in the overlays increased as their exposure increased. He also compared cracking in continuous spans with cracking in simply supported spans. The continuous spans exhibited more transverse and random cracking than the simply supported spans, but the difference was not significant. However, the majority of the overlays included in Bishara's investigation (*16*) were placed on existing decks, and therefore they exhibited a greater degree of drying shrinkage cracks and reflective cracks than would be expected for new construction. Reflective cracks in an overlay are likely due to cracks existing in the substrate prior to overlaying. The substrate cracks initiate cracks in the overlay, they extend under the stress of traffic impact, and eventually reach the surface. Reflective cracking in concrete overlays placed on newly constructed decks can be minimized by minimizing cracking in the underlying deck and by repairing any cracks that develop prior to overlaying.

REPAIR OF CRACKS

Cracks in concrete decks may be sealed by scrubbing the mortar portion of a concrete mix into the cracks or by feeding a low-viscosity, low-modulus polymer into the cracks with gravity, injection, or both. Laboratory tests by Smatzer and Zandar on latex-modified concrete (*60*) demonstrated that scrubbing the mortar portion of the concrete into the cracks does not seal the cracks well against chloride penetration. The same tests showed the greater effectiveness of sealing with a polymer (epoxy) penetrating sealer (Fig. C-5). Field investigations by McKeel in Virginia (*43*) showed that for a crack in a bare deck that was routed to a minimum depth of ¼ in. and filled with a low-viscosity, low-modulus epoxy adhesive, the chloride content was 0.15 lb/cu yd at a depth of 1 to 1½ in. An unsealed section on the same crack and at the same depth had a chloride content of 1.07 lb/cu yd.

The effectiveness of sealing cracks with polymers depends greatly on whether there is dirt in the cracks, because dirt prevents bond with the concrete. Some polymers are also sensitive to moisture and may not develop a satisfactory bond when the concrete is wet. Cracks should be cleaned out with compressed air, dried, and sealed as soon as they form. Sealing cracks with polymers is effective if the cracks are not active.

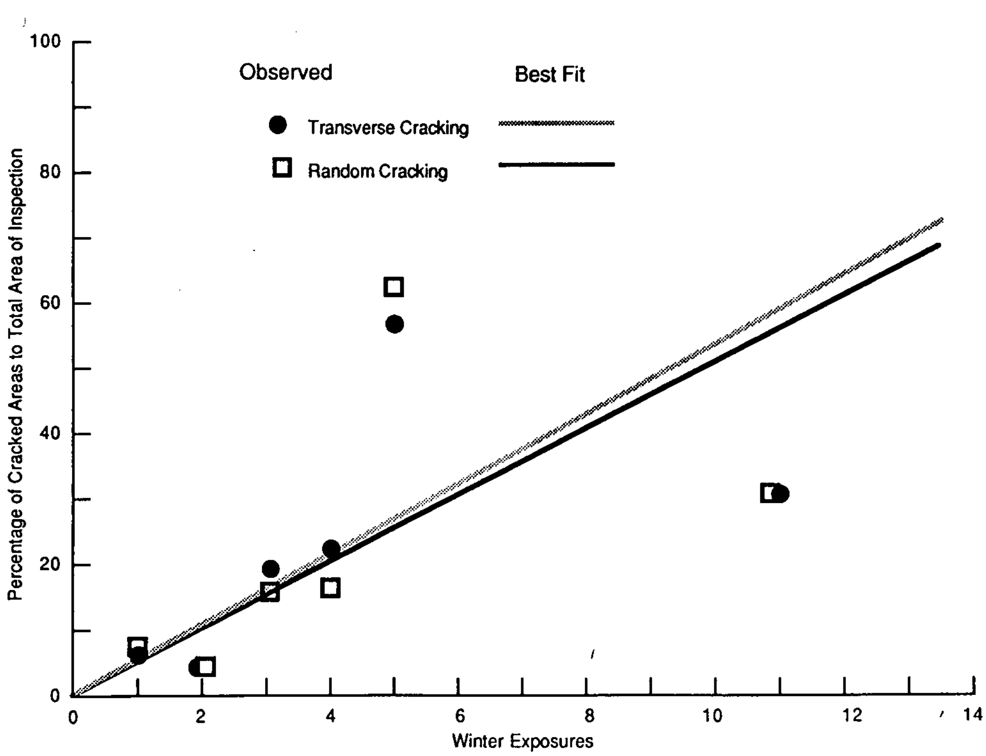

Figure C-4. Percentage of cracked areas to areas for 92 latex-modified concrete overlays. (Adapted from Ref. 16)

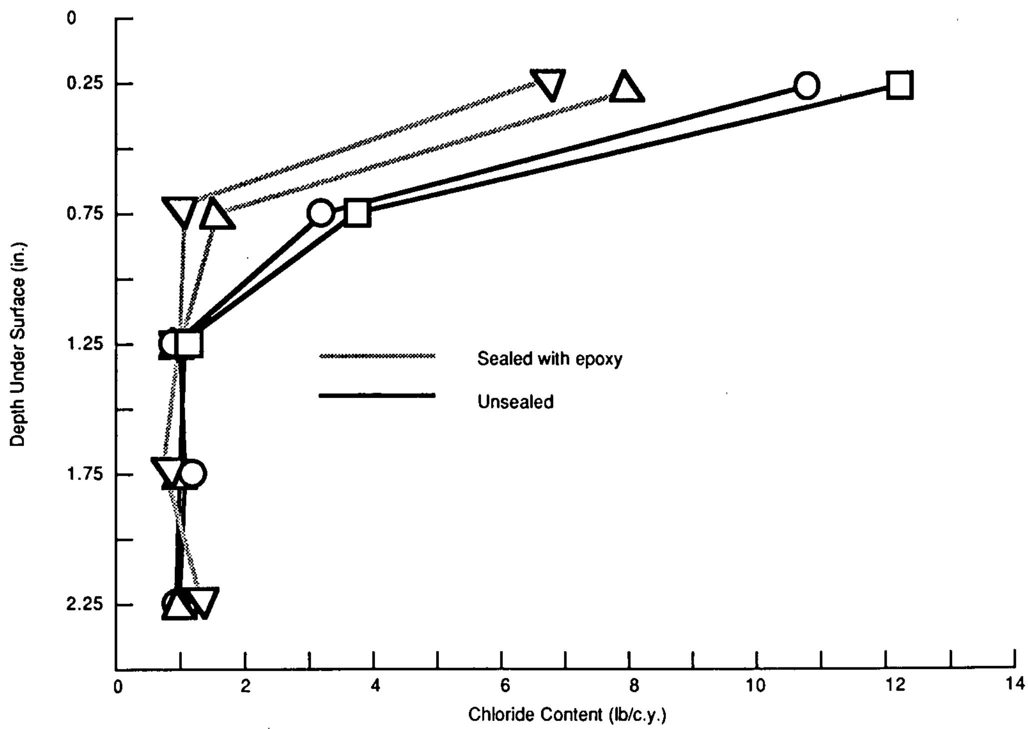

Figure C-5. Chloride penetration into sealed and unsealed cracks in latex-modified concrete after exposing to 50 cycles of ponding with calcium chloride, freezing, thawing, and partial drying. (Adapted from Ref. 60)

The widths of active or moving cracks change markedly with flexure or thermal cycling. No criteria has been established to determine if cracks are active, though cracks that extend through the depth of the slab are usually active (*7*). Even with the use of a low-modulus polymer material, filling of an active crack may initiate cracking elsewhere in the concrete at a later age. Such actions are a result of the small widths of most cracks and the relatively rigid nature of hardened polymer materials. The polymer material cannot strain effectively, yet is strong enough that it does not fail, and therefore it causes cracking of the adjacent concrete. Active cracks in bridge decks need to be identified and treated differently. Active cracks may be sealed by routing and then filling with a flexible asphaltic material that can elongate considerably (*61*).

CONTRIBUTION OF CRACKING TO BRIDGE DECK CORROSION

Bare Decks with Cover Depth ≥ 3 Inches

Transverse cracking is the most prevalent type of cracking in bare decks and has several causes. Transverse cracks in bare decks generally follow the line of the uppermost transverse bars; their depths may extend to the level of the bar or even through the depth of slab; and they can form at any location in the span (*43*). Transverse cracks, if not repaired, facilitate the diffusion of chlorides and carbon dioxide into the concrete and render the transverse steel anodic relative to the protected steel. The length of the active anodic transverse steel will be roughly equal to the length of the transverse crack located directly above the steel or close to it. Transverse cracks can also activate the longitudinal steel that is immediately below the transverse steel. In the longitudinal steel, however, the length of active steel is small and equal to about three bar diameters (*62*).

According to Bazant (*63*), increases in diffusivity in cracked concrete are proportional to the cube of the crack width and, according to Beeby (*62*), although in the early years of service small crack widths (less than 0.004 in.) may prevent the diffusion of corrosive substances, long-term exposure, such as 10 years, negates such effects. The amount of time before corrosion-induced deterioration begins is a function of the rate of corrosion, the depth of the bar, the diameter of the bar, and the strength of the concrete. The higher the electrical resistivity between the anodic and the cathodic steel, the lower is the rate of corrosion and the greater the time to the onset of deterioration. With a low water/cement ratio concrete is more impermeable so that its moisture content is reduced and its electrical resistivity increased. However, decreases in the water/cement ratio also increase the electrical resistivity independent of changes in the water content (*64*). Limitation of the water/cement ratio to 0.45 provides a reasonable compromise between desirable corrosion resistance properties for a concrete and workability. A low water/cement ratio concrete is a stronger concrete and one more resistant to internal fracture. The rate of corrosion can also be reduced by reducing oxygen diffusion at the major cathode, which is the bottom reinforcing mat. That reduction can be achieved by providing a greater depth of cover for the bottom mat. A depth greater than 1 in. is appropriate.

The mode of corrosion-induced cracking in concrete depends on bar depth, bar spacing, and bar diameter (*63*). Decks supported on longitudinal girders and with 3 in. or more of cover will have cracks emanating from the corroding bar and parallel to the surface. Those cracks cause delaminations near the bar but not spalling. For cover depths of 3 in. or more, the amount of time between the initiation of corrosion and delaminiation depends greatly on the rate of corrosion, but may be assumed to be 3 years for a typical bridge deck (*35*). The fatigue stresses caused by wheel loads are worst at locations where the concrete is distressed internally by corrosion, and therefore traffic plays a role in propagating deterioration, eventually turning delaminations into spalls. However, in cracked concrete corrosion products can leach out along the cracks, releasing the pressure that the corrosion products might otherwise exert upon the concrete (*63*). This explains why coincidence of deterioration and transverse cracking is more evident in the decks of older bridges.

One possible scenario is that transverse cracks will cause deterioration of the surrounding concrete sometime after 15 years of service. Typically after 5 to 10 years of service 100 ft of transverse cracking develops per 1,000 sq ft of deck area, and more cracking may develop later. Thus, the percentage of deck area affected by corrosion-induced deterioration may be calculated by knowing the width of the strip of concrete centered along the transverse crack, which may initially delaminate. If this width is assumed equal to the spacing of the transverse bars (usually 6 in.), the total area that may delaminate because of the corrosion after 15 years of service will be about 5 percent of the deck area.

Decks Overlaid with Concrete

Unlike bare decks, in which transverse cracking is the most prevalent type of cracking, low-slump dense and latex-modified concrete overlays are highly susceptible to other types of cracking that do not always penetrate into the first stage construction. However, these cracks may deepen and promote the intrusion of salt into the first stage construction in about 10 years and may cause contamination of the concrete surrounding the top steel mat, which is generally placed 1½ in. below the bonded interface. Performance information shows that the possibility of 150 ft of cracking per 1,000 sq ft of bridge deck area exists after only 5 to 10 years of service. If it is assumed that each crack can contaminate concrete up to 2 in. from it in each direction of the horizontal surface, the total area of the deck that may be contaminated and deteriorated before the bridge's 50-year service period is over is 5 percent of the deck area.

Bare Decks Built with Epoxy-Coated Bars

This system should resist corrosion even when transverse cracks develop along, and extend down to the bar. However, because flexing of the structure causes transverse cracks to open and close at the bar, the coating may wear away, especially at the bar's ribs (Fig. C-6a). Furthermore, under repeated flexing, the crack width at the bar may become as large as the width at the surface and make wear of the coating even worse (Fig. C-6b).

In decks built with epoxy-coated bars, a cover of 2 to 2.5 in. is usually specified. This condition minimizes the possibility that subsidence cracks will occur directly over the bar. However, despite the absence of this type of cracking, vertical planes of weakness directly above the bar may be present due to initial strains in the plastic concrete in that region. Later, transverse flexural and shrinkage cracking will probably form in the weakened concrete present above the transverse bar.

The extent of the damage that epoxy coating in the transverse bar may endure can be determined by knowing the width of the crack at the transverse bar. If longitudinal reinforcing steel is provided in the bridge deck and distributed so that the surface width of the transverse cracks caused by flexure and shrinkage is limited to δ_1 (Fig. C-6a), the cracks' width at the transverse bar will be δ_2, where $\delta_2 < \delta_1$. However, over time, the bond between the longitudinal steel and the concrete breaks down because of repeated flexing. Eventually, the crack width at the level of the bar becomes about the same as the crack width at the surface (*49*). The deeper the longitudinal bar is embedded in the concrete, the wider the cracks will be at the surface. If δ_1 is 0.01 in., a measurement typical of that taken in decks with epoxy-coated bars after 7 years of service (*43*), then δ_2, the crack width at the surface of the transverse bar, will eventually also become 0.01 in. A logical extension of the hypothesis presented here is that if the epoxy coating wears away, the width of the damaged coating on the bar's surface will be 0.01 in. and the length will be the crack length, which is probably several feet.

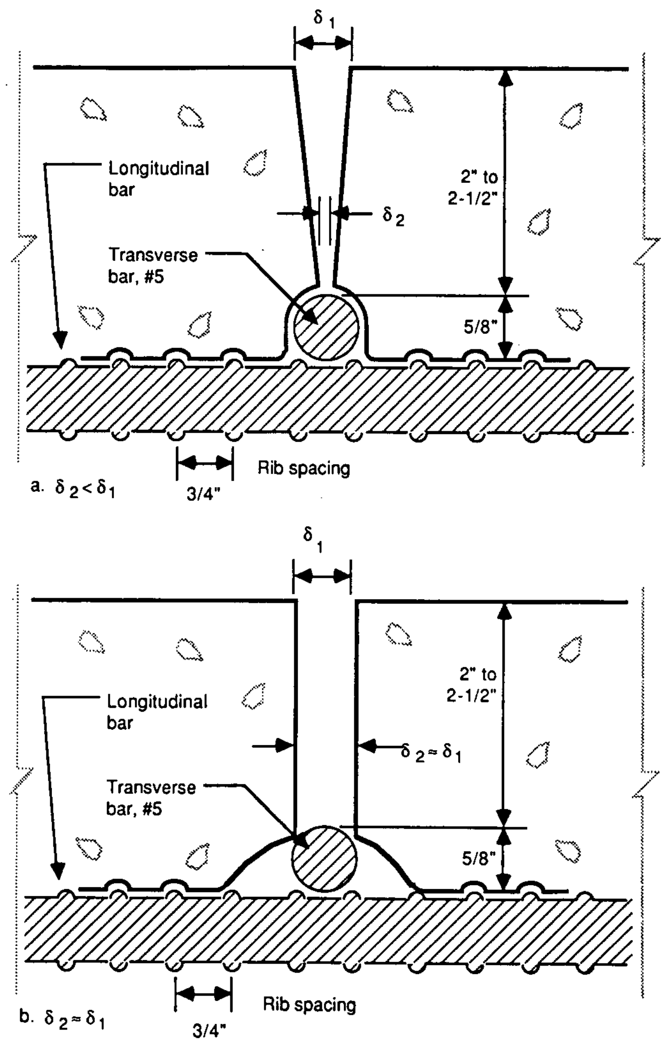

Figure C-6. Exaggerated schematic diagram of breaking of concrete bond and widening of crack at bar surface.

Thus, the damaged area will be 0.51 percent of 1 ft of the No. 5 bar's surface area. FHWA's recommended limits on damaged areas of coating are 0.25 percent with only the top mat coated and 2 percent with both top and bottom mats. Thus, the likely areas of worn epoxy coating are tolerable only when both mats are epoxy coated.

APPENDIX D

BRIDGE DECK DETERIORATION DUE TO EXCESSIVE FLEXIBILITY

The possibility of deck concrete deterioration due to a number of factors, including flexibility, concrete overlays, composite and noncomposite construction, and continuity, was investigated. Concrete overlays become an integral part of the deck structure and should be included in the structural evaluation of a bridge deck. However, because of limited design data, it is recommended that further testing be conducted on concrete overlays. Testing should include material mechanical properties and composite flexural specimens.

This investigation was mainly concerned with the structural behavior of girder bridges with composite and noncomposite reinforced concrete decks. Steel and precast concrete girder bridges were the primary structure types investigated because they represent the greatest number of federally aided highway bridges with the exception of reinforced concrete culverts (*65*). The most frequently used types of short- and medium-span bridges are composite precast prestressed concrete girder bridges, composite steel girder bridges, and noncomposite steel girder bridges. Other bridge types are not analyzed in detail because they are custom designed (i.e., reinforced and post-tensioned concrete box-girder bridges).

Longitudinal and transverse flexibility of the different girder bridges with and without concrete overlays was investigated. It is assumed that the concrete overlays act compositely with the base concrete because of the surface preparation typically specified for overlay construction (*66*). Concrete overlay material is assumed to be either latex-modified concrete (LMC) or low-slump dense concrete (LSDC). Because of a lack of information, the materials properties of the overlay are conservatively assumed to be similar to that of the base concrete.

An attempt is made to assess the effect of flexibility on cracking in the negative moment region of continuous span bridges and deck slabs.

DEFINITION OF FLEXIBILITY

The midspan curvature, $\rho = 1/R = M/EI$, due to live load is used to compare the stiffnesses of deck slab and girder units. Dead load being a sustained load is a static condition and varies with structure type. The magnitude of dead load would only seem to affect the natural period of vibration of the structure. Therefore, because live load moments will be the same for similar structures, live load curvature (behavior) is a better indicator of bridge flexural performance.

The bending moment, *M*, incorporates the effects of length and end restraints on the members investigated. The flexural rigidity, a product of *EI*, provides a measure of member stiffness because it involves both the material and cross-section properties of the member. It can be shown that the midspan deflection is directly proportional to the span and the curvature is inversely proportional to the span.

EVALUATION OF FLEXIBILITY

An attenpt was made to use standard designs, available from various sources (*67, 68, 69*), with similar conditions noted as follows: same spans, same girder spacing, similar concrete deck, same live loading, and same end restraint conditions.

The following design guides and assumptions were used to evaluate the different bridge types:

- AASHTO "Standard Specifications for Highway Bridges" (Ref. *70*)
- WSDOT "Bridge Design Manual" (Ref. *71*)
- HS-20-44 Truck or Land Loading, as appropriate, with impact
- Unshored construction for the girder bridges
- No live load reduction for three or more lanes
- Interior girder design using a live load distribution factor of $S/5.5$
- Service load stresses and deflections computed for live load and impact only, using transformed sections
- Load factor resistance design (LFRD) method used for steel members
- Ultimate strength design method used for reinforced concrete members
- Service load design method used for prestressed concrete members

Flexibility in two directions was evaluated: longitudinal direction flexibility, which considers the girder, concrete deck structure, and end restraint conditions; and transverse direction flexibility, which considers a continuous concrete deck structure and reinforcing steel.

TRANSVERSE FLEXIBILITY

The following concrete deck slab system with and without a concrete overlay was investigated (see Fig. D-1):

Girder spacing:	9-ft average
Structural slab (f'_c = 4000 psi):	7½-in. thickness
Reinforcing steel:	No. 5 at varying spacing
Concrete overlay:	2-in. thickness

Procedure

The analysis procedure consisted of selecting the required reinforcing steel for the plain concrete deck from Ref. *71* and then determining the cracked transformed section properties. Cracked transformed section properties were used because the slab will be cracked under live loading conditions. The analysis of the deck sections with a concrete overlay was similar except that the amount of reinforcing steel required for the plain concrete deck was used.

The dead load and live load moments were then determined from AASHTO (Section 3.24.3.1 of Ref. *70*). The curvature was calculated as

$$\frac{M\ (\text{Live Load} + \text{Impact})}{EI}$$

Results

Although various combinations of slab effective span and reinforcing were investigated because of the different girder types, the curvatures at midspan for the plain concrete deck were very similar, $\cong 1.5 \times 10^{-4}$ rads/in. The curvatures at midspan for the decks with concrete overlays showed a similar pattern with curvatures of $\cong 7.6 \times 10^{-5}$ rads/in. The addition of a concrete overlay decreased the midspan curvature by a factor of $\cong 2$. The curvatures at the support were $\cong 2.3 \times 10^{-4}$ rads/in. Because the overlay is added to the "tension" side of the deck slab, it will crack and hence not affect curvature at the support significantly. The differences in curvature between the midspan and support regions are due to changes in stiffness caused by the location of the reinforcement within the cracked transformed section. The effect of this varying stiffness upon moment distribution was not investigated.

CONCRETE DECK SLAB INVESTIGATION

Because of the surface preparation performed prior to placing, the overlay will bond structurally to the base concrete and hence act compositely until the bond is broken. When subjected to repeated loadings, there are three potential problems caused by this structural behavior: (1) horizontal shear or bond failure of the concrete overlay, (2) crack amplification in the overlay because of increased cover over the reinforcing steel, and (3) differential shrinkage and vertical thermal gradients.

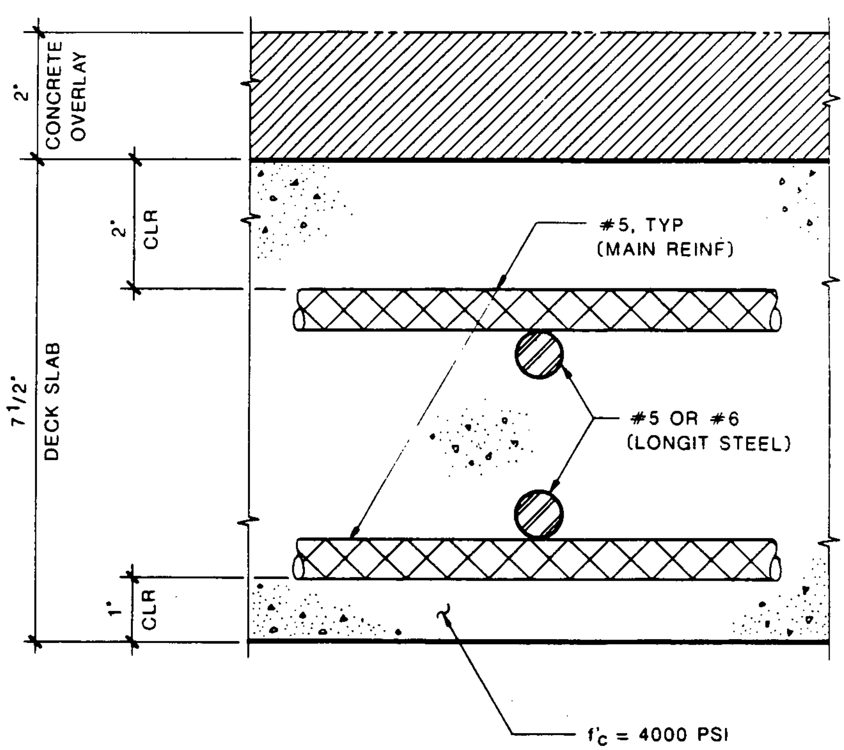

Figure D-1. Deck slab details.

Bond Failure

Failure of the overlay can be initiated at a number of locations within the composite member: (1) within the overlay, (2) at the bond line, and (3) within the base concrete.

Failure in the overlay can be related to poor material quality within the overlay. This can be as a result of poor construction practices. Failure at the bond line can be related to poor surface preparation. Failure in the base concrete can result from poor material quality at the surface and also as a result of the surface preparation techniques used (i.e., scarifying equipment can cause microcracking in the base concrete due to pounding of the machine). This microcracking can cause planes of weakness near the surface of the base concrete.

The addition of an overlay to the base concrete raises the neutral axis of the transformed cracked concrete section to very near the bond line (interface) between the two materials. The neutral axis is the location of maximum shear and hence the most likely zone for a structural failure. Some failures have been observed at the bond line or within ¼ to ½ in. of the bond line in the base concrete.

A horizontal shear analysis by VQ/It was performed for a concrete deck slab with a concrete overlay. Standard AASHTO wheel loads (Ref. *70*) and an allowable interface shear (V_u) of 80 psi were used. Based on this work, the failure is most likely to occur in the transverse direction as the horizontal shear stresses are much greater in this direction than in the longitudinal direction. Debonding would occur in the positive moment region of the slab, typically in and between the wheel paths. Debonding in the negative moment region would not be likely because the overlay concrete would probably fail in tension prior to bond failure.

Crack Amplification

The critical cracking region is in the top of the deck over the supports (negative moment region). The overlay does not increase the stiffness of the slab in the negative moment region (after cracking), but does increase the stiffness at midspan. The crack width in a reinforced concrete member is related to the tensile stress in the reinforcing steel and the concrete cover. The standard AASHTO expression (Formula 8–61, Ref. *70*) for determining flexural reinforcement distribution was derived for members with concrete cover over the tensile reinforcing steel typically less than 2 in. The formula would be applicable to the base concrete section but, when extrapolated to the extreme tension fiber of the overlay, the crack width will be magnified because the reinforcing steel is fixed and the concrete cover increased. It is suggested that the Gergely-Lutz expression (Section 10.6.4 of Ref. *74*) be used to determine crack widths because of this large increase in cover. The most significant parameter in the equation is β (ratio of distances to the neutral axis from the extreme tension fiber and from the centroid of the main reinforcement). For typical applications, β ranges from 1.20 to 1.35, but will be much greater for members with increased cover. Using this expression, crack widths of 0.017 in. ($Z = 137$, per Ref. *70*) for the plain concrete deck and 0.32 in. ($Z = 185$, per Ref. *70*) for the deck with a concrete overlay were calculated at the extreme tension fibers. The crack width for the plain concrete deck is acceptable; however, the wide crack at the surface of the overlay would present a serviceability problem because it would allow the ingress of water and chlorides to the base concrete and therefore negate the purpose of the overlay.

Differential Shrinkage and Transverse Thermal Gradients

Interface shear stresses can be generated at the base concrete to overlay surface by differential shrinkage and thermal gradients. Shrinkage of the overlay occurs after much of the drying shrinkage in the base slab has occurred. This differential shrinkage can cause interface shear stresses near the exterior perimeter of the deck slab of ≅ 100 psi (*72*). The magnitude of these stresses can be computed by applying an effective modulus of elasticity to the overlay concrete. The magnitude of interface shear stresses drops rapidly away from the perimeter to nearly zero in the bays between interior bridge girders.

Differential shrinkage does, however, generate axial tensions in the overlay of nearly 350 psi. The distribution of these axial stresses within the slab is shown in Figure D-2. The magnitude of axial stresses is reduced by lowering the total drying shrinkage strain of the overlay.

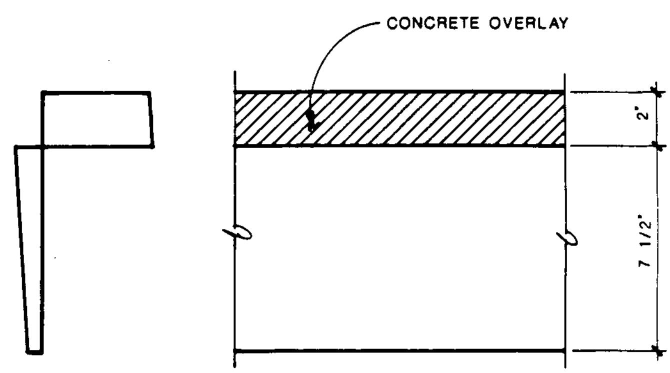

Figure D-2. Distribution of stresses in bridge decks due to differential shrinkage.

Differential temperatures between the top fiber of the overlay and the bottom fiber of the base slab can also create interface shear stresses between the overlay and the base slab. These temperatures can be as high as 25 °F (*73*). The magnitude and distribution of these interface shear stresses are similar to those caused by differential shrinkage.

Axial stresses can also be created. When the surface of the deck slab is hotter than the slab soffit, axial compressive stresses are created in the deck slab. If the surface is colder than the soffit, the reverse is true. Axial tensile stresses are also created in the bridge slab even without overlays when the deck slab cools less than the supporting superstructure elements. A bridge deck slab drop of 15 °F (*73*) can result in an axial stress of ≅ 400 psi.

Differential shrinkage and thermal gradients do not create high interface shear stresses in the deck slab to overlay surface except near the perimeter of the slab. Significant axial tensile stress, however, can be generated. These axial stresses could

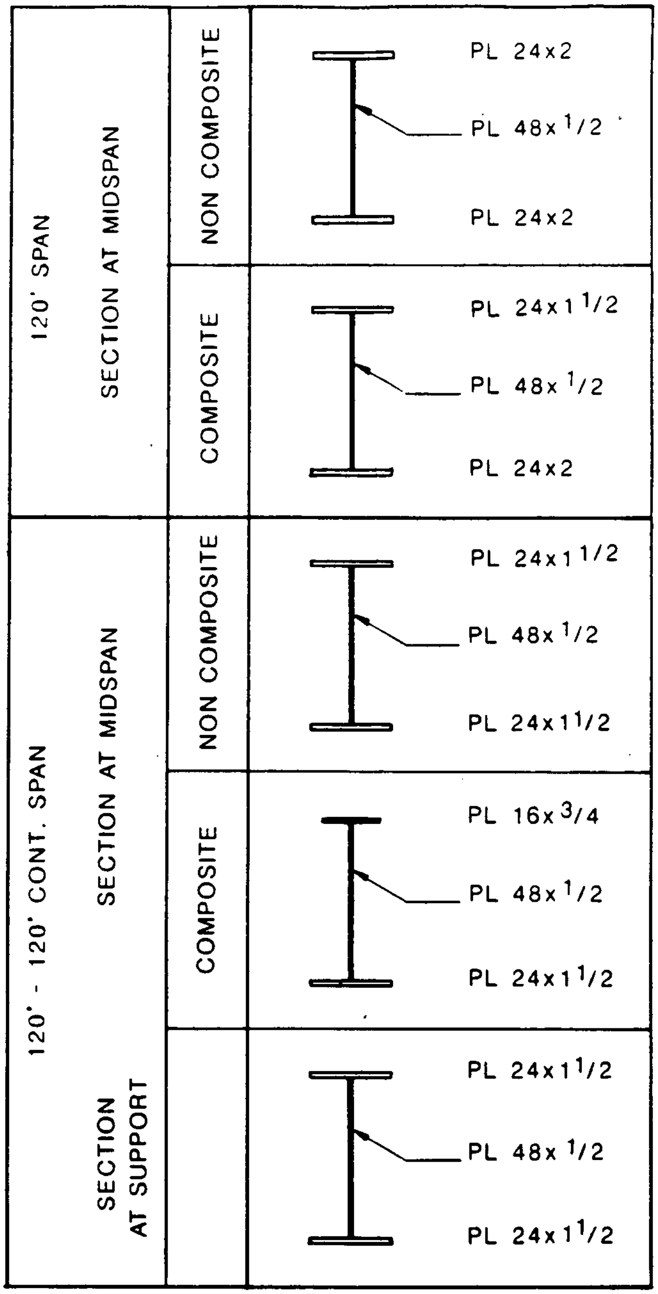

Figure D-3. Plate girder sections.

produce cracking, which can lead to deterioration of the base slab by allowing water or chlorides to penetrate through the overlay.

LONGITUDINAL FLEXIBILITY

The following girder bridges were investigated. A girder investigation matrix is provided in Table D-1.

80-Ft Simple Spans and 80-Ft–80-Ft Continuous Spans

- AASHTO Type IV precast girder with composite slab, f_c' = 6,000 psi

Table D-1. Girder investigation matrix.

	SHORT-SPAN STRUCTURES			
	80 ft		80 ft - 80 ft	
Type	Noncomposite	Composite	Noncomposite	Composite
Precast Girder	--	AASHTO Type IV	--	AASHTO Type IV
Wide-Flange Section	W36 x 300*	W36 x 280	W36 x 280	W36 x 245
Plate Girder Section	--	--	--	--

* 70-ft span

	MEDIUM-SPAN STRUCTURES			
	120 ft		120 ft - 120 ft	
Type	Noncomposite	Composite	Noncomposite	Composite
Precast Girder	--	AASHTO Type VI	--	AASHTO Type VI
Wide-Flange Section	--	--	--	--
Plate Girder Section	**	**	**	**

** See Figure D-3 for details.

- W36 rolled shape with composite slab, ASTM A36 steel
- W36 rolled shape with noncomposite slab, ASTM A36 steel

120-Ft Simple Spans and 120-Ft–120-Ft Continuous Spans

- AASHTO Type VI precast girder with composite slab, f_c' = 6,000 psi
- Built-up plate girder with composite slab, ASTM A36 steel
- Built-up plate girder with noncomposite slab, ASTM A36 steel

Procedure

The analysis procedure consisted of selecting the appropriate girder for the span under investigation and then determining the cracked transformed section properties. This procedure was used for both the plain concrete deck and the deck with a concrete overlay. Composite member details are shown in Figure D-4. The live load moments were then determined from Ref. 75. The curvature was calculated as

$$\frac{M\ (\text{Live Load} + \text{Impact})}{EI}$$

Results

The steel girder systems are more flexible than the precast concrete girder systems at midspan by a factor of $\cong 2$ for the shorter spans and $\cong 1.7$ for the longer spans. The magnitudes of curvature at midspan range from $\cong 2.1 \times 10^{-5}$ rad/in. to $\cong 4.0 \times 10^{-6}$ rad/in. The results of the analysis are shown in Figure D-5.

The steel and precast concrete girders in the two-span continuous bridge systems are of approximately equal stiffness at

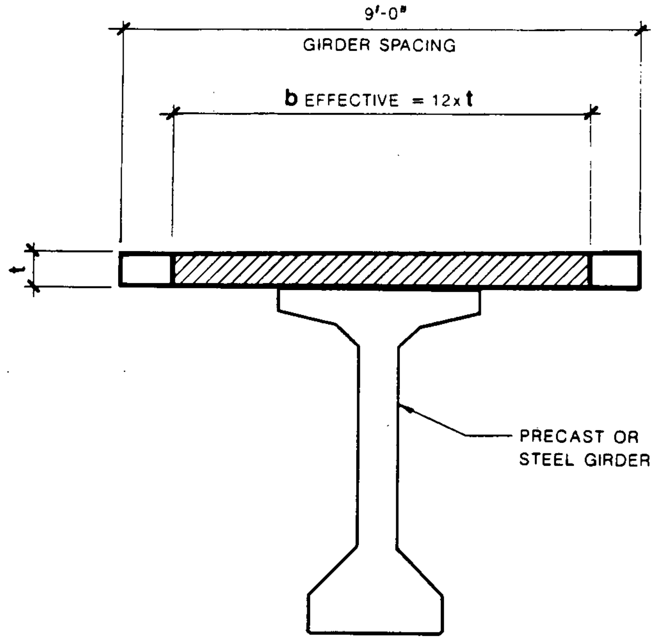

Figure D-4. Composite member details.

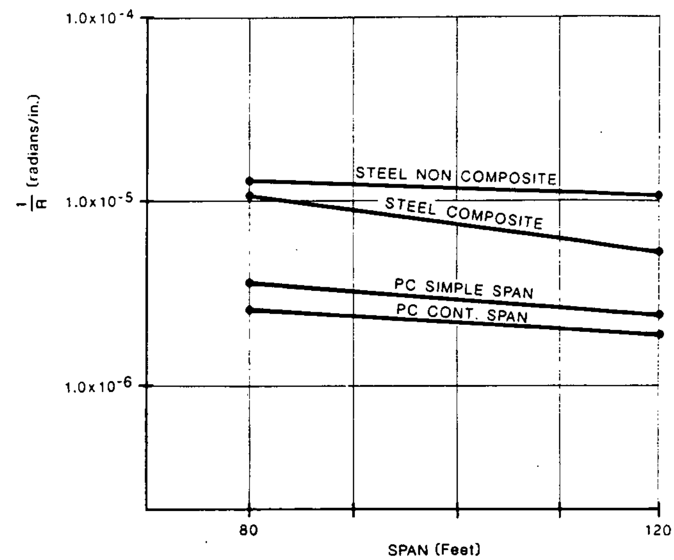

Figure D-5. Curvature vs. span.

the interior support (negative moment region). The magnitudes of curvature range from $\cong 1.7 \times 10^{-5}$ rad/in. to $\cong 1.2 \times 10^{-5}$ rad/in. for the 80-ft–80-ft and 120-ft–120-ft systems, respectively.

The concrete overlay does not affect longitudinal flexibility at midspan as significantly as in the transverse direction. The girders become $\cong$ 20 percent stiffer in the longitudinal direction at midspan with the concrete overlay.

CRACKING IN CONTINUOUS STRUCTURES

The following additional assumptions were used in evaluating the potential crack regions:

- Concrete deck reinforcing steel resists live load and impact only, i.e., unshored construction.
- Moment envelopes were developed for 80-ft–80-ft and 120-ft–120-ft continuous composite steel and precast concrete girders from Ref. *75*.
- Moment envelopes were developed for 80-ft–80-ft and 120-ft–120-ft continuous noncomposite steel girders from Ref. *75*.

Procedure

The analysis procedure consisted of determining the required negative moment (live load only) reinforcing steel and then determining the uncracked transformed section properties for the girders. An allowable concrete tensile stress of 100 psi was selected assuming 2 million cycles of loading.

The potential region of cracking was then determined by checking stresses along the length of the girders. Crack widths were calculated at the supports by use of the Gergely-Lutz expression (Section 10.6.4 of Ref. *74*). See Figure D-6.

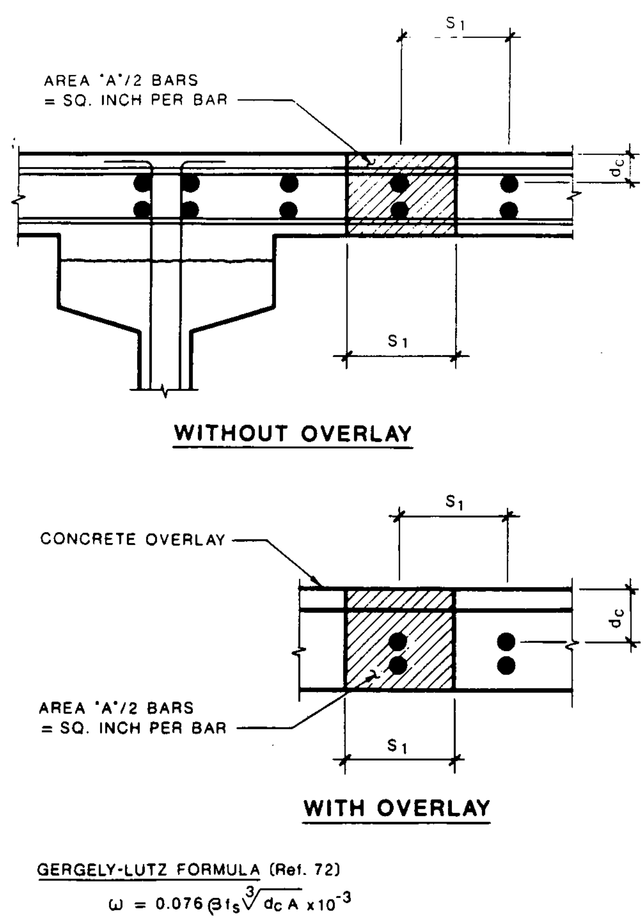

Figure D-6. Crack width calculation.

Results

There is a larger potential crack region for steel girder bridges than for concrete girder bridges primarily because the concrete girder systems are stiffer (i.e., greater *EI*). Although, as higher concrete strengths are used in precast girders, they too will become less stiff and the potential crack regions will increase (*76*). Regions of cracking extended for ≅ 40 percent of the span on each side of the support for precast girder systems and for ≅ 60 percent of the span on each side of the support for steel girder systems.

Crack widths at the support are similar for all of the steel and precast girder bridges, although the quantities of reinforcing steel are different. This can be attributed to a limit in reinforcing steel stress of ≅ 20 ksi (Ref. *70*). The effect of the concrete overlay is minimal when calculating the crack width because β (Section 10.6.4 of Ref. *74*) changes very little. The β in the longitudinal direction is related to the overall girder depth and hence a 2-in. overlay does not contribute significantly to a change in β. Maximum crack widths of 0.010 in. for the plain concrete deck and 0.012 in. for the deck with a concrete overlay were calculated. These crack widths are much less than those in the transverse direction and fall well within acceptable limits.

FINDINGS

Girder bridge systems are typically stiffer in the longitudinal direction than in the transverse direction. In the transverse direction, stiffnesses are determined by the slab thickness and span. Steel and concrete girder bridge systems have equal stiffness in the transverse direction.

Concrete girder bridge systems are stiffer in the longitudinal direction than the steel girder bridge systems; hence, the regions of potential cracking in the longitudinal direction of continuous spans are greater for steel bridges.

The most likely areas for structural problems occur in the transverse direction. The slab is likely to crack longitudinally due to live load and impact for both simple and continuous spans because the slab is continuous over the girders. Additionally, if the slab is overlayed, there is also a possibility of the overlay debonding in and between the wheel paths in the transverse direction.

An attempt was made to select typical bridges for structural analysis, but the conclusions formed using AASHTO specifications and design aids are in contradiction to field experience. Longitudinal cracking is not as prevalent as transverse cracking (Refs. *1, 2, 37*). Some possible explanations for this discrepancy are:

- AASHTO load distribution factors and design assumptions may be too conservative
- The observed deck slabs had not been subjected to the design loadings.
- The methods used to calculate cracking and crack width may not be appropriate for bridge deck slabs.
- The deck slab is supported on "flexible" members (bridge griders) in the transverse direction and hence some moment redistribution will take place, reducing the negative moment and increasing the positive moment.

The computed transverse crack widths in the concrete deck of continuous bridges are similar for steel and precast girder bridges because the crack width is a function of the allowable reinforcing steel stress in the deck, which is similar for either type of structure. The zone of cracking though would be larger for a steel girder bridge.

The concrete overlays present a structural problem. They are bonded to the concrete deck slab, yet do not contain any additional reinforcing and thus do not contribute to the stiffness of the girder in the negative region. They make a significant contribution to transverse direction midspan stiffness, ≅ 100 percent increase, but are less significant in the longitudinal direction, ≅ 20 percent increase. Crack widths at the surface of the concrete overlay are related to the reinforcing steel stress in the deck concrete and the concrete cover which has been effectively increased by up to 2 in., thus magnifying the crack width.

APPENDIX E

COST EFFECTIVENESS OF PROTECTIVE STRATEGIES

This appendix evaluates the cost effectiveness of the most commonly practiced protective strategies, as well as combinations of these strategies. In order to make the evaluation meaningful, the lifetime costs of the strategies, including major maintenance, were calculated using a mathematical model. The basic cost items used were those typical in Washington State (see Table E-1). Obviously, the lifetime cost evaluations presented here are valid only for bridges which fit the study's assumptions and may not necessarily represent every individual case. However, the procedure outlined in this appendix can be employed to calculate and compare the cost effectiveness of individual construction alternatives if the corresponding basic cost items and the bridge's service condition are known.

DESCRIPTION OF COST-EFFECTIVENESS MODEL

The cost-effectiveness model consists of eight alternative bridge deck constructions, as shown in Figure E-1. Alternative I is a basic, unprotected concrete deck typical of those supported

Table E-1. Typical Washington bridge deck construction related costs.

Description of Construction Item	Cost ($)
7-1/2" thick concrete deck (supported by pre-stressed girders)	12/s.f.
Ready-Mix 4000 psi concrete	60/c.y.
Portland cement	6/bag
Cost of epoxy-coating rebar	0.15/lb.
Latex-modified concrete overlay, 1-1/2" thick	2.85/s.f.
Low slump dense concrete overlay, 2" thick	2.60/s.f.
Scarification of deck for concrete overlay	0.75/s.f.
Waterproofing membrane	0.89/s.f.
Asphalt concrete pavement, 1.8" thick	0.32/s.f.
Removal of existing asphalt concrete pavement	0.45/s.f.
Traffic control (concrete overlay)[1]	2.00/s.f.
Traffic control (asphalt concrete/membrane system) [2]	1.00/s.f.

(1) An average traffic control cost when the bridge deck repair is the primary purpose of the construction. Based on a limited sampling, the traffic control cost for latex-modified concrete overlays' has ranged from $0.19/s.f. to $4.65/s.f. The cost for low-slump dense concrete overlay is essentially the same as that for latex-modified concrete overlay.

(2) Traffic control cost for asphalt concrete/membrane projects when the sole purpose of the contract is bridge deck construction has been as high as $1.08/s.f.

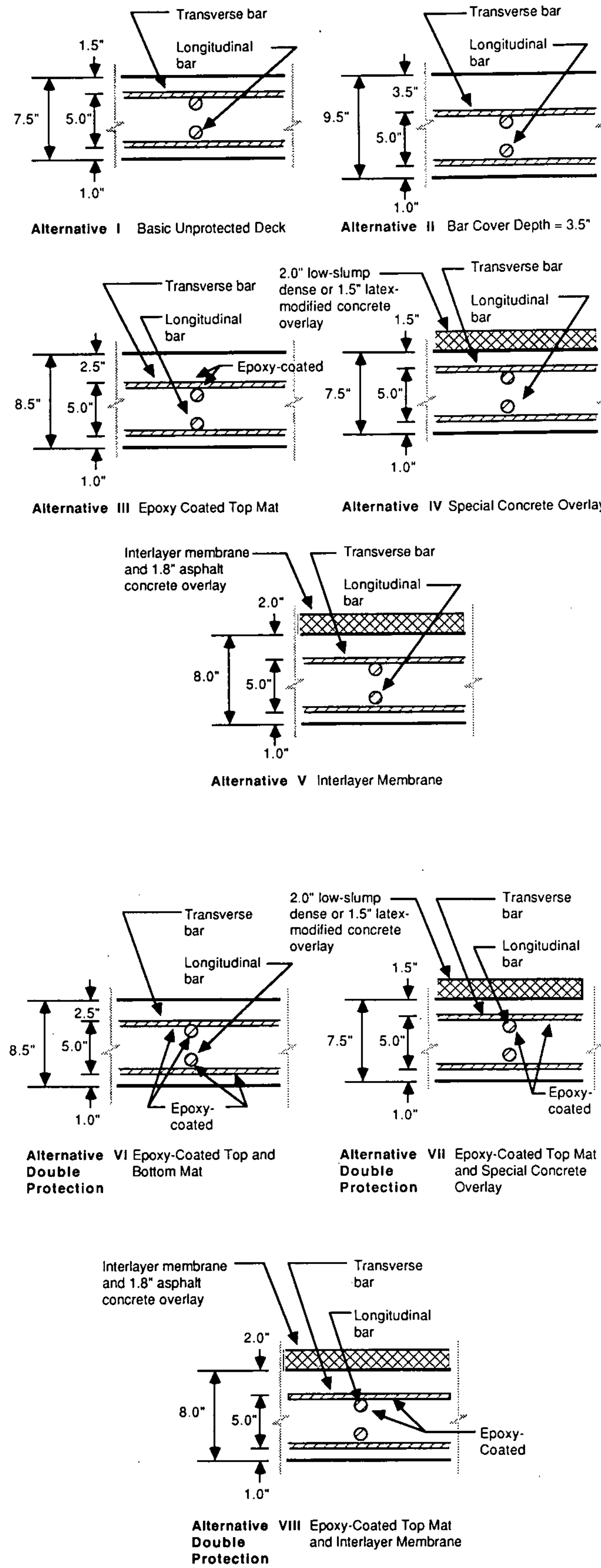

Figure E-1. Most commonly practiced singly and doubly protected bridge deck construction alternatives.

by longitudinal girders. This alternative utilizes 1.5 in. of clear concrete over the uppermost bar and is not presently used in corrosive environments because of its short service life. However, this alternative was extensively used in the past. Alternatives II through V, or singly protected decks, are the basic deck (Alternative I) plus one of the commonly used protective strategies, with possible adjustment in cover thickness. The protective system for Alternative II is 2 additional inches of bar cover, making the total cover thickness 3.5 in. Alternative III is epoxy-coated reinforcing steel in the top mat with a 2.5-in. cover, 1 in. more than the basic deck. Alternative IV employs either a 2-in. low-slump dense concrete or a 1.5-in., latex-modified concrete overlay as the second stage of the basic deck construction. The use of either a 2-in., low-slump dense concrete or a 1.5-in., latex-modified concrete does not significantly alter the cost-effectiveness model since typical initial costs and service performances for those two alternatives are similar. The protective system for Alternative V is the application of an interlayer waterproofing membrane and 1.8 in. of asphaltic concrete as the second stage of basic deck construction, with 2 in. of concrete cover.

Alternatives VI through VIII are the doubly protected bridge deck constructions most commonly used nationwide. Alternative VI is a basic deck with both its reinforcing mats epoxy-coated and its cover thickness adjusted to 2.5 in. Alternative VII is a basic deck with an epoxy-coated top mat bar plus the application of either a 2-in., low-slump dense concrete overlay or a 1.5-in., latex-modified concrete overlay as the second stage of construction. Alternative VIII also includes epoxy-coated reinforcing steel in the top mat but has an interlayer membrane and 1.8 in. of asphaltic concrete as the second stage of construction. Additionally, the cover thickness of Alternative VIII is 2 in., 0.5 in. more than the basic deck.

Lifetime Cost Comparison

To evaluate the cost effectiveness of the bridge deck construction Alternatives II through VIII, their lifetime costs were determined and compared. The effective service period of each alternative was assumed to be 50 years, and the salvage value of each of the structures at the end of its service life period was assumed to be negligible. Additional assumptions were that all cracks were sealed and that no corrosion-induced deterioration from salt could occur during the 50 years of service (see Appen. B and Appen. C). The total cost of each project includes the initial construction cost plus maintenance costs throughout the service period. For the purpose of this model, only the major maintenance costs of overlaying and resurfacing the decks were considered. The costs of routine maintenance, such as crack sealing or surface patching, and costs incurred to users because of delays during maintenance were not considered in this model.

In order to evaluate the lifetime cost of each alternative, consideration was given to the timing of maintenance activities as well as the effects of interest and inflation rates on the timing of expenditures. The total present worth project cost for each alternative was determined by translating all maintenance costs to their present worth before summing them. The following formula was used:

$$DF = \frac{1}{(1 + EI)^N}$$

where DF = discount factor, EI = effective interest rate = interest rate − inflation rate, and N = number of years to maintenance.

For this model an annual interest rate of 10 percent and an inflation rate of 5 percent were used, giving an effective interest rate equal to 5 percent. The total present worth project cost for every two alternatives (excluding Alternative I) was then compared in a matrix so that the additional cost of one alternative relative to the other was obtained.

CONCRETE COVER = 3.5 IN.—ALTERNATIVE II

The initial cost of this alternative includes the basic deck plus the cost of 2 in. of additional concrete cover. Placement of the additional concrete should not require significant labor costs. The adoption of Alternative II might reduce the amount of transverse bar in the bottom mat because the concrete section would be greater. That reduction would offset additional labor costs, if any. If the cost of the basic deck is assumed to be $12.00/sq ft and the material cost for ready mix concrete of 4,000 psi is assumed to be $60/cu yd, the initial cost of Alternative II is $12.00/sq ft + $0.37/sq ft = $12.37/sq ft (Table E-2).

The amount of time between construction and maintenance of the bare deck is a function of the severity of the environment, especially the number of freeze-thaw cycles, the amount of salt used, and the volume of traffic. Under average conditions, distress such as scaling, rutting, and lack of skid resistance will take 25 years to create conditions requiring an overlay. The deck might be overlaid with a 2-in. conventional concrete (cement factor of 6) after 25 years. If the cost of installing a 2-in. conventional concrete overlay (not including material costs) is assumed to be approximately the same as the cost of installing a 2-in. low-slump dense concrete overlay with a cement factor of 8 (although the installation of the latter involves special equipment), the difference in the overall cost would mainly correspond to savings in the cost of cement and would be $0.07/sq ft of overlay. If the overall cost of a low-slump dense concrete overlay is $2.60/sq ft, cost of the conventional concrete overlay is $2.53/sq ft. The total cost of the operation, including the cost of traffic control and deck scarification, is $5.28/sq ft. Once the maintenance cost is discounted to present worth, the total project cost is $13.93/sq ft (Table E-2). In this model the effective service period of the overlay is assumed to be 25 years.

EPOXY-COATED TOP-MAT REINFORCING STEEL—ALTERNATIVE III

The initial cost of this alternative is the sum of the cost of the basic deck ($12.00/sq ft), the cost of providing 1 in. of additional concrete cover ($0.19/sq ft), and the cost of epoxy-coating the top mat bar ($0.60/sq ft). The latter figure is based on the assumption that the bridge deck contains 4 lb/sq ft of top mat steel and that the cost of epoxy coating that steel is $0.15/lb. The total initial cost of Alternative III is $12.79/sq ft. Because this strategy uses a bare deck, the maintenance plan discussed for Alternative II is also a possibility here. If the deck is overlaid with 2 in. of conventional concrete with a cement factor of 6 after 25 years, and that overlay lasts for another 25 years, the maintenance costs including the cost of scarifying, overlaying, and traffic control are $5.28/sq ft and they occur in the 25th year of service. When the latter figure is discounted to present worth, the total present cost is $14.35/sq ft (Table E-2).

SPECIAL CONCRETE OVERLAYS—ALTERNATIVE IV

The initial cost of this strategy is the cost of the basic deck ($12.00/sq ft) plus the cost of installing a special concrete overlay ($2.53/sq ft, the average cost for latex-modified and low-slump dense concrete) in the second stage of construction, or a total of $14.73/sq ft (Table E-2). The latter figure does not include the cost of scarification and traffic control.

The effective service period of the concrete overlay depends on the amount of surface distress, such as scaling and wear that occurs and can be assumed to be 25 years. Thus, the bridge is likely to be resurfaced with concrete 25 years after its construction, and that surface is likely to last for another 25 years. Resurfacing could be accomplished by applying an overlay of the same nature as the initial overlay after scarifying the deck to a depth of ¼ in. Consideration should also be given to the accumulation of dead loads. The second concrete overlay would cost $5.48/sq ft, including the cost of scarification and traffic control. This brings the total present worth cost to $16.35/sq ft after discounting the maintenance cost to present worth.

Note, however, that the life expectancy of concrete overlays is also a function of their bond with the substrate. Therefore, the effective service life of concrete overlays may be less than 25 years because of debonding and stripping of the overlay caused by various factors, such as the presence of an initially weak bond and high traffic exposure. Defects in the bond in-

Table E-2. Lifetime cost of different bridge deck construction alternatives.

		Alternative	Cost per Sq. Ft. of Deck Area (typical 1986 cost, $) Initial Cost	Lifetime Cost 12 years	Lifetime Cost 25 years	Lifetime Cost 38 years	Lifetime Cost 50 years	Present value of 50 years lifetime cost **
Basic Deck	I	7.5" thick deck, * 1.5" cover	$12.00	—	—	—	—	—
Basic Deck Plus Single Protection	II	Alt. I plus 2" additional cover	12.37		Resurf. w/ 2" conc. 5.28			$13.93
	III	Alt. I plus 1" additional cover plus epoxy-coated top mat	12.79		Resurf.w/ 2" conc. 5.28			14.35
	IV	Alt. I plus 1.5" latex or 2.0" low-slump overlay	14.73		Resurf.w/ 1.5" latex or 2" l.s. conc. 5.48			16.35
	V	Alt. I plus 0.5" additional cover plus 1.8" AC/membrane	13.30	Resurf. w/ 1.8" AC/membrane 2.66	Resurf. w/ 1.8" AC/membrane 2.66	Resurf. w/ 1.8" AC/membrane 2.66		15.98
Basic Deck Plus Double Protection	VI	Alt III. plus epoxy-coated bottom mat	13.39		Resurf. w/ 2" conc. 5.28			14.95
	VII	Alt. IV plus epoxy-coated top mat	15.33		Resurf.w/ 1.5" latex or 2" l.s. conc. 5.48			16.95
	VIII	Alt V plus epoxy-coated top mat	13.90	Resurf. w/ 1.8" AC/membrane 2.66	Resurf. w/ 1.8" AC/membrane 2.66	Resurf. w/ 1.8" AC/membrane 2.66		16.58

* Alternative I (unprotected deck) is not considered as a possible strategy. ** Assuming 10% interest rate and 5% inflation rate.

terface may require complete removal of the overlay or rebonding of the overlay with epoxy injection. If the concrete overlay is scarified to its full depth, damage may result to the structure and other techniques, such as removal of the concrete through hydrodemolishing, may give better results. Iowa and Kansas have experimented with epoxy injection for rebonding overlays.

INTERLAYER MEMBRANE/ASPHALTIC CONCRETE OVERLAY—ALTERNATIVE V

The initial cost of this alternative is the sum of the cost of the basic deck ($12.00/sq ft), the cost to provide 0.5 in. of additional concrete cover ($0.09/sq ft), and the cost of an interlayer membrane covered with at least 1.8 in. of asphaltic concrete overlay ($1.21/sq ft), or a total of $13.30/sq ft (Table E-2). This strategy can provide 50 years of chloride-free service provided that the interlayer membrane and the asphaltic concrete are removed and replaced when worn. Heavy traffic volumes may affect flexible membrane systems placed on rigid concrete decks. Repeated flexing of the membrane, combined with its aging and brittleness, may result in its failure. Other types of distress, usually experienced prior to failure to the membrane, are wear and rutting of the overlay under wheeltracks, and debonding and stripping of the overlay caused mainly by the accumulation of water above the interface. The model presented in Table E-2 assumes that the protective system will be removed and replaced 12 to 13 years after installation. Thus, three successive removal and replacement operations are needed for the projected 50 years of service and each operation costs $2.66/sq ft, including traffic control for this type of reconstruction. The total present worth cost is $15.98/sq ft. Note also that the cost incurred to the user is not included in the total costs. If there is high traffic use, the cost incurred to the user because of partially closing the bridge during the three major maintenance operations may govern the overall cost effectiveness of this strategy.

When asphaltic concrete and membrane systems are removed, care must be taken not to disturb the concrete surface, because the next membrane will not be compatible with a disturbed concrete surface. One alternative for solving this problem is the application of a ½- to 1-in. leveling asphaltic concrete course on the disturbed concrete deck, followed by the application of the membrane and the wearing course. The leveling course may also protect the concrete deck in the next membrane removal operation. Some agencies may experience premature surface failure of the asphaltic concrete wearing course. This may happen under very high traffic exposure. In these circumstances removal and replacement of the upper portion of the asphaltic concrete wearing course with the membrane intact may be possible. This plan may require wearing course thicknesses of at least 2 in.

Table E-3. Comparison of the lifetime cost of bridge deck construction alternatives.[1]

	Alternative No.[2]	Single Protection				Double Protection		
Alternative No.[2]	Protective Strategy	II	III	IV	V	VI	VII	VIII
		Cover thickness = 3.5"	Epoxy-coated top mat bar	Special conc. overlay	Interlayer membrane	Epoxy-coated top & bottom mats	Epoxy-coated top mat & special conc. overlay	Epoxy-coated top mat & interlayer membrane
Single Protection II	Cover thickness = 3.5"	100 / 100	103 / 100	117 / 100	115 / 100	107 / 100	122 / 100	119 / 100
Single Protection III	Epoxy-coated top mat bar	100 / 103	100 / 100	114 / 100	111 / 100	104 / 100	118 / 100	116 / 100
Single Protection IV	Special conc. overlay	100 / 117	100 / 114	100 / 100	100 / 102	100 / 109	104 / 100	101 / 100
Single Protection V	Interlayer membrane	100 / 115	100 / 111	102 / 100	100 / 100	100 / 107	106 / 100	104 / 100
Double Protection VI	Epoxy-coated top & bottom mat	100 / 107	100 / 104	109 / 100	107 / 100	100 / 100	113 / 100	111 / 100
Double Protection VII	Epoxy-coated top mat & special conc. overlay	100 / 122	100 / 118	100 / 104	100 / 106	100 / 113	100 / 100	100 / 102
Double Protection VIII	Epoxy-coated top mat & inter-layer membrane	100 / 119	100 / 116	100 / 101	100 / 104	100 / 111	102 / 100	100 / 100

1 The table determines percent additional cost of one alternative relative to the other.
2 See Figure E-1 for detailed description of the alternative bridge deck construction.

DOUBLE PROTECTION—ALTERNATIVES VI THROUGH VIII

Alternatives VI through VIII (Fig. E-1) are classified as double protection. The following is a brief description of these alternatives:

Alternative VI — Epoxy-coated top mat (Alternative III) + epoxy-coated bottom mat
Alternative VII — Special concrete overlays (Alternative IV) + epoxy-coated top mat
Alternative VIII — Interlayer membrane/asphaltic concrete overlay (Alternative V) + epoxy-coated top mat

The costs for Alternatives VI, VII, and VIII are similar to the costs for Alternatives III, IV, and V, respectively, except for an additional initial cost of \$0.60/sq ft for epoxy-coating either the bottom or top mat (Table E-2). Thus, the total present worth cost for Alternative VI is \$14.95/sq ft, the cost for Alternative VII is \$16.95/sq ft, and the cost for Alternative VIII is \$16.58/sq ft.

Highway agencies have used double protection because they lack long-term field experience with the different strategies possible. At this time some conservatism, such as the use of double protection, is reasonable for building certain types of structures. Such structures are those whose integrity can be seriously impaired by corrosion deterioration and those for which repair of corrosion damage would be costly, either because of the complexity of the repair or because of high traffic volumes (with their consequent high user costs). Although some agencies use double protection on every new construction, others have developed factors for determining whether a structure should be given double protection. Those factors are: (a) type of structure, (b) geographical location of structure, (c) impact of possible deck repair on traffic flow, and (d) extent of salt use.

Factor "a" may apply to structures using prestressing steel in the deck. On these structures, because epoxy-coating of the prestressing steel may not be feasible, a chloride-proofing overlay for protection of the prestressing steel may be required while the bar in the top mat is epoxy-coated. Another reason for double protection of these structures is the complexity of their repair. Factor "b" represents the importance of the transportation link and the public's dependence on the structure. Factor "c" stands for the cost to the travelling public resulting from closure of the bridge due to repairs, and factor "d" represents the severity of the exposure to salt. The question of whether conditions warrant application of double protection can only be answered subjectively by considering those four factors for each newly designed bridge deck as well as the additional costs of double protection.

LIFETIME COST COMPARISON OF BRIDGE DECK CONSTRUCTION ALTERNATIVES

The total present worth costs of the seven commonly used, singly and doubly protected bridge deck construction alternatives are given in Table E-2. For the singly protected decks, protection with a concrete cover equal to 3.5 in. is the least expensive alternative, followed by protection with an epoxy-coated top mat, an interlayer membrane, and a special concrete overlay. For the three double protection alternatives, epoxy coating both mats is the least expensive strategy and epoxy coating the top mat in conjunction with a special concrete overlay is the most expensive strategy. Interestingly, the cost of epoxy coating both mats may be lower than the cost of protecting the deck with only a special concrete overlay or asphaltic concrete and an interlayer membrane.

Table E-3 compares total present worth costs of every two construction alternatives in a matrix so that the additional cost of one alternative can be determined relative to the other. For singly protected decks, cost differences vary from 2 to 17 percent. For doubly protected bridge decks, cost differences vary from 2 to 13 percent. When all the double protection alternatives are compared with the single protection alternatives, it can be seen that the most expensive double protection alternative (epoxy coating the top mat in conjunction with a special concrete overlay) costs 22 percent more than the least expensive single protection alternative (cover thickness equal to 3.5 in.). The least expensive double protection alternative (epoxy coating both mats) costs about 9 percent less than the most expensive single protection alternative (special concrete overlay).

APPENDIX F

SURVEY OF CURRENT PROTECTION METHODS ON NEWLY CONSTRUCTED BRIDGE DECKS AND THEIR PERFORMANCE

To provide information about the protection of newly constructed bridge decks and their performance, a questionnaire (Fig. F-1) was mailed to all state highway departments and the District of Columbia in May of 1986. The questionnaire was intended to generate information on (1) rates of salt application, (2) types of protective systems used, (3) properties of bridge deck concrete and protective systems, (4) methods of testing and condition evaluation, and (5) the performance of protective strategies. Responses from 45 states and the District of Columbia were received by September 15, 1986. This appendix outlines those responses, tabulates them, and comments on them. Some of the responses were incomplete and some may have been inaccurate. Consequently, the tables in this appendix may contain errors and omissions. However, the tables are sufficiently reliable to reflect overall practices and performance nationwide.

USE OF DEICING SALTS

Details of the state highway agencies' policies for highway salt application are given in Table F-1. Only five out of 46 respondents indicated that their policy is not to use salt. Among those state agencies using salt, nine use salt only in part of the state. The rest of the agencies use salt throughout their states. Salt application rates as high as 30 tons per lane-mile were reported for some states in the northeast. These figures generally reflect roadways. However, bridges may receive more salt because they are in a special environment. Salt may also be applied by other local agencies in addition to state agencies.

USE OF PROTECTIVE STRATEGIES

Table F-1 provides detailed information on the states' involvement in various types of bridge deck protective strategies, including double protection. The practices included in Table F-1 are current practices, both standard and experimental. However, after experimenting for 10 years or more, some states have either discontinued or limited the use of certain types of protective strategies. This fact is reflected in the tabulation of the respondents' comments in the footnotes to Table F-1. Problems such as cracking in and debonding of concrete overlays, wear and stripping of asphalt overlays, or the ineffectiveness of some types of sealers in preventing chloride penetration may account for those changes. Among the protective strategies used as standard practice, epoxy coating bars is the most popular. Forty-one states epoxy coat their bars, although they may also use other systems depending on the nature of the exposure and the importance of the bridge. Some states also use epoxy-coated bars in conjunction with a different type of protective strategy, such as a low permeability concrete overlay, interlayer membrane, or surface sealer. Presently, six states use double protection on every bridge deck they construct, and 19 states use it only on selected bridge decks that have been identified as critical transportation links.

The information in Table F-1 has been condensed further into Table F-2 and compared to the results of a similar nationwide survey conducted in 1977 (*7*). Forty-eight states responded to that survey. As indicated in Table F-2, the use of epoxy-coated bars and concrete sealers has increased substantially between

Name of Respondent________________________ State_____________________

Title___________________________________ Phone_______________

SURVEY OF CURRENT PROTECTION PRACTICE ON NEWLY CONSTRUCTED BRIDGE DECKS AND COMPLETELY REPLACED BRIDGE DECKS*

1. Does your agency use de-icing salts on bridge decks?

   [ ] Yes

   [ ] No (if no, go to question 4)

2. What is the extent of bridge deck salt usage in your state?

   [ ] Is salt usage limited only to parts of the state?

   [ ] Is salt usage widespread?

3. Please specify the approximate maximum, minimum, and average salt usage on your agency's bridges in terms of tons per lane-mile per year, if the records are available. If the records are not available, please indicate the approximate maximum, minimum, and average number of salt applications per year on your agency's bridges.

   Maximum tons per lane-mile per year_____ Maximum times per year_______________

   Minimum tons per lane-mile per year_____ Minimum times per year_______________

   Average tons per lane-mile per year_______ Average times per year________________

4. Does your agency use any of the bridge deck protective systems listed in the following table as a standard procedure, as an experimental procedure, or not at all? Please indicate by checking (√) in the appropriate boxes.

*Questions 4 through 14 apply only to the protection of newly constructed bridge decks and completely replaced bridge decks.

Figure F-1. Questionnaire mailed to all states in May 1986.

Protective Systems	Standard Procedure	Experimental Procedure	Not Tried
Concrete Cover ≥ 3"			
Epoxy Coated Rebar			
Galvanized Rebar			
Low-slump Dense Conc. Overlay			
Superplasticized Dense Conc. Overlay			
Latex-modified Concrete Overlay			
Ultrathin Polymer Concrete Overlay			
Concrete Sealers			
Asphalt Conc./ Membrane System			
Cathodic Protection			

5. Does your agency use any bridge deck protective system not listed in the preceding table as a standard procedure?

   [ ] Yes (if yes, please specify____)

   [ ] No

6. What is the protective system most commonly used by your agency as the standard procedure? Please rank in order (1 is most commonly used, 6 is least commonly used):

   [ ] Cover depth > or = to 3 inches

   [ ] Expoxy coated rebar

   [ ] Low-slump dense concrete overlay

   [ ] Latex-modified concrete overlay

   [ ] Asphalt concrete/membrane system

   [ ] Others, please specify__________________

7. Does your agency apply any combination of two protective systems (double protection) on a bridge deck?

   [ ] Yes

   [ ] No (if no, go to question 10)

   If the answer to question 7 is yes, please indicate any possible combinations of protective systems by checking (√) in the appropriate boxes in the following table:

Protective System / Protective System	Latex-modified Conc. Overlay	Low-slump Dense Conc. Overlay	Asphalt Conc./ Membrane System	Concrete Cover ≥ 3"	Epoxy-coated Rebar
Epoxy-coated Rebar					X
Sealing Concrete Surface	X	X	X		

8. Does your agency apply any combination of two protective systems not indicated in the preceding table on a bridge deck?

   [ ] Yes (if yes, specify)__________________________

   [ ] No

Figure F-1. Continued

9. If your agency is practicing double protection, is double protection applied on

   [ ] every bridge deck?

   [ ] only selected bridge decks?

10. Does your agency construct bare decks with no epoxy coating (or galvanized) rebar?

   [ ] Yes

   [ ] No (if no, go to question 11)

   If the answer to question 10 is yes, please specify the following practices employed by your agency:

   a. Maximum bridge deck water/cement ratio______________________________

   b. Design target and minimum clear concrete cover thickness, inches____________

11. Does your agency construct bare decks with epoxy coated rebar?

   [ ] Yes

   [ ] No (if no, go to question 12)

   If the answer to question 11 is yes, please specify the following practices employed by your agency:

   a. Maximum bridge deck concrete water/cement ratio________________________

   b. Design target and a minimum clear concrete cover thickness, inches___________

12. Does your agency construct bridge decks with low-slump dense concrete overlays as the second stage of construction?

   [ ] Yes

   [ ] No (if no, go to question 13)

   If the answer to question 12 is yes, please specify the following practices employed by your agency:

   a. Maximum water/cement ratio of bridge deck (first stage construction)______________________

   b. Design target and minimum clear concrete cover thickness of first stage construction, inches_____

   c. Maximum water/cement ratio of overlay (second stage construction)______________________

   d. Maximum slump of overlay, inches______________________________________

   e. Minimum thickness of the overlay, inches____

13. Does your agency construct bridge decks with latex-modified concrete overlay as the second stage of construction?

[ ] Yes

[ ] No (if no, go to question 14)

If the answer to question 13 is yes, please specify the following practices employed by your agency:

a. Maximum water/cement ratio of bridge deck (first stage construction)________________

b. Design target and minimum clear concrete cover thickness of first stage construction, inches_____

c. Maximum slump of overlay, inches________________

d. Minimum thickness of overlay, inches________________

14. Does your agency construct bridge decks with asphalt concrete/membrane systems as the second stage of construction?

[ ] Yes

[ ] No (if no, go to question 15)

If the answer to question 14 is yes, please specify the following practices employed by your agency:

a. Minimum thickness of asphalt concrete, inches________________

b. The most commonly used type of interlayer membrane (check the appropriate box):

[ ] Liquid applied-in-place system

[ ] Preformed sheet system

c. Describe the nature of interlayer membrane checked in 14b________________

____

Figure F-1. Continued

15. Does your agency use any of the bridge deck test methods listed in the following table as a routine procedure, an experimental procedure, or not at all? Please indicate by checking (√) in the appropriate boxes.

Test Method		Routine Procedure	Experimental Procedure	Not Tried
Visual Inspection	Spalling			
	Cracking			
	Scaling			
	Wear			
Chloride Content Determination				
Delamination Detection				
Half-cell Corrosion Detection				
Electrical Resistivity Testing				
Depth of Rebar Cover Survey (Pachometer)				
Rapid Chloride Permeability Testing				

16. Has your agency collected field condition data on protected new bridge decks that are 10 years or older? Please check (√) the appropriate boxes in the following table for the type of protective system and type of data available.

Data / Protective System	Chloride Penetration	Rebar Half-cell Potential	Deck Concrete Delamination	Overlay Debonding	Overlay Stripping	Concrete Scaling	Surface Wear
Concrete Cover ≥ 3"							
Epoxy-coated Rebar							
Low-slump Dense Conc. Overlay							
Latex-modified Conc. Overlay							
Asphalt Conc. Membrane System							

17. Identify by placing an asterisk (*) in the appropriate boxes of the preceding table the unsatisfactory conditions that your agency has experienced with protected new bridge decks regardless of their age.

18. Has your agency collected field condition data on rehabilitated/protected bridge decks that have 10 years or more of rehabilitation/protection? Please check (√) the appropriate boxes in the following table for the type of protective system and type of data available.

Data / Protective System	Rate of Chloride Penetration in Overlay	Overlay Debonding	Overlay Stripping	Concrete Scaling	Surface Wear
Low-slump Dense Conc. Overlay					
Latex-modified Conc. Overlay					
Asphalt Conc. Membrane System					

19. Identify by placing an asterisk (*) in the appropriate boxes of the preceding table the unsatisfactory conditions that your agency has experienced with rehabilitated/protected bridge decks regardless of the age.

20. Please identify the name and telephone number of the individual to contact regarding the bridge deck condition data collected by your agency.

Figure F-1. Continued

Name________________________________ Phone______________

Table F-1. Survey of deicing salt applications and type of bridge deck protection strategies.

State	Salt Used: Policy on state highways	Salt Used: Geographically widespread	Extent of salt applications per year: Tons/lane-mile max	Tons/lane-mile min	Tons/lane-mile avg	Times max	Times min	Times avg	Protective Strategy: Conc. cov. ≥ 3"	Coated bar: Epoxy	Coated bar: Galvanized	Concrete overlay: Latex-modified	Concrete overlay: Low-slump dense	Concrete overlay: Superplasticized dense	Concrete overlay: Ultrathin polymer	Interlayer membrane/AC	Concrete sealer	Cathodic protection	Double protection +
Alabama	Yes	No								S							S	E	NP
Alaska	Yes	Yes								S I		E				S I [1]	S [1]	E	SD
Arkansas	Yes	No				10	0	2		S			E				E [2]		NP
Connecticut	Yes	Yes	15.73	2.75	8.16			[3]		S I, II		E				S I	S II	E	ED
Delaware	Yes	Yes								S I		S I				E		E	SD
Florida	No									S I [4]	E	[5]					S I	E	SD
Hawaii	No																		NP
Idaho	Yes	Yes	1.3	0.5					E	S I, II	E	S I	S [6]			S II [6]	E	E	SD
Illinois	Yes	Yes	15 [7]	2.4	6.5	93	12	50		S I		I				S [8]			SD [9]
Indiana	Yes	Yes	12	6	8	45	25	30	E III	S I, II, III	E	S I	S II			[1]	S III		SD
Iowa	Yes	Yes	10	4	5	102	24	63		S	E	E	E		E	E	E		NP
Kansas	Yes	Yes	10.73	1.44	4.21	17	2	6	E IV	S I,II, III, IV		E I [10]	S II			E	E III [11]		SD
Kentucky	Yes [12]	Yes			7.5			30 [13]		S I,II		S I	S II			[14]	E	E	ED [15]
Louisiana	Yes	Yes				5 [16]				E		E	E		E	E			NP
Maine	Yes	Yes	8.0	5.6	6.4	20	15	17	S II, III	E I,II		S I		S		S	S III	E	SD
Maryland	Yes	Yes			2.82 [50]					S			E						NP
Massachusetts	Yes [17]	Yes								S I,II	E	E I				S II			ED
Michigan	Yes	Yes			27			80 [18]	II	S I,II	[19]	S I	E	E [20]		S		E	ED
Minnesota	Yes	Yes	13.2 [21]	1.9 [21]	4.3 [21]	47	39	43.5	S III	S I,II, III	E	S I	S II			S [22]	S III	E	SD
Mississippi	No [23]	No																	NP
Missouri	Yes	Yes	[51]	[51]	[51]				III	S I,II, III		I	II				S [2]		SD
Montana	No [24]	No							E	S		E	E					E	NP
Nebraska	Yes [25]	Yes								S I	[26]	[27]	[27] I			[27]		[27]	SD
Nevada	Yes	No	[28]	[28]	[28]					S									NP

+ See the matching Roman numerals under "Protective Strategy" for the type of double protection.
Legend: E – Experimental procedure, S – Standard procedure, ED – Every deck doubly protected, SD – Selected decks doubly protected, NP – Not practicing double protection.

State	Salt Used		Extent of salt applications per year						Protective Strategy										Double protection +
			Tons/lane-mile			Times				Coated bar		Concrete overlay							
	Policy on state highways	Geographically widespread	max	min	avg	max	min	avg	Conc. cov. ≥ 3"	Epoxy	Galvanized	Latex-modified	Low-slump dense	Superplasticized dense	Ultrathin polymer	Interlayer membrane/AC	Concrete sealer	Cathodic protection	
New Hampshire	Yes	Yes	20.4	7.3	15.9			90		S I		E	E	E		S I	E		ED
New Jersey	Yes	Yes	6.5	2.75	4.5	44	30	37		S I	E	S I	E			S	E		SD
New York	Yes	Yes	30	5	17.5					S I,II	E	I	II			S			SD [29]
North Carolina	Yes	No			[30] 1.38	[31]	[31]	[31]		S	E	E	E			E			NP
North Dakota	Yes	Yes	[32]	[32]	[32]					S			S				E		
Ohio	Yes	Yes	6.3	0.5	5.5				IV	S I,II,III,IV	E	S I	II	S	E	S III	E IV	E	SD
Oklahoma	Yes	Yes	1.2	0.85	1.0					S I		E	S I			[33]	S [34]	E	SD
Oregon	No									S I	E	S	E		E	S	S I	E	SD [35]
Pennsylvania	Yes	Yes	6.25	0.75	3.5	50	10	30		S	E	S [36]	S [36]	E	E	S [36]	E	E	NP
Rhode Island	Yes	Yes						25		S I,II		S I				S II		E	SD
South Carolina	Yes	No				5	0	0.5											NP
South Dakota	Yes	No	5.9	0	0.19					S		E [37]	E [37]						NP [37]
Tennessee	Yes	No	1.0	0.63	0.38	8	5	3		S		E	E	E	E	S	E		
Texas	Yes	No	.15 [38]	0	[39]	15	0	[39]		S I,II			E	E	E	S I	S II [40]	E	SD
Utah	Yes	Yes	9	0.1	2.5	60	2	25		S		E [41]	E [41]	[42]			E	[42]	NP
Vermont	Yes	Yes	18	6	12.7	[43] 140	[43] 80	[43] 105		S I	E		E			S I			ED
Virginia	Yes	Yes			0.13	5		2		S		E [44]	E	E	E	[44]	E	E	NP
Washington	Yes	No	[45] 2.5			20				S I,II		S I		E [46]	E	S II	E	E	SD
West Virginia	Yes	Yes						100		S			E			E	E	E	NP
Wisconsin	Yes	Yes	30	8	12	205	50	85		S			E			E	S	E	NP
Wyoming	Yes [47]	Yes			8.9					S I,II	E	E I					E II		SD
Dist. of Columbia	Yes	Yes				12	4	8	S [48]	S							E [49]		NP

+ See the matching Roman numerals under "Protective Strategy" for the type of double protection.

Legend: E – Experimental procedure, S – Standard procedure, ED – Every deck doubly protected, SD – Selected decks doubly protected, NP – Not practicing double protection.

Table F-1. Continued

Footnotes:

1- No longer used.
2- Linseed oil as a standard procedure.
3- Number of storms, 22.
4- Coastal environment.
5- Patching only.
6. Has not been used very frequently.
7- 5-year average. Bridges receive more salt because of frost problems.
8- On PCC deck beams and segmental concrete girder bridges.
9- Linseed oil protective coat only on post-tensioned box girder construction.
10- In specifications, not used much.
11- Chemtrete.
12- Approximately 500 lb. per lane-mile per application.
13- Assume 10 storms, 3 applications each.
14- Have been used on a limited basis, not satisfied with results.
15- All new projects and selected rehabilitations.
16- In north and south of state bridges receive 5+/year and 1-/5 year, respectively.
17- Standard application, 300 lb. per lane-mile per application
18- 100 times and 60 times in the south and north of state, respectively.
19- Two bridges.
20- Micro silica additive concrete is used as standard procedure.
21- Usage figures are for roadways. Usage on bridges would be higher.
22- System used seldom.
23- Official policy is to not use de-icing salts. However it is rumored that some deicing salts are occasionally boot-legged.
24- Cities apply salt.
25- A mix of 1 salt to 4 aggregate. Application of salt-aggregate mix at a rate of 600 to 800 lb. per lane-mile.
26- One structure.
27- Discontinued.
28- Max., min., and avg. amount of annual sand-salt used is 5.7, 4.6, and 5.1 c.y. per lane-mile, respectively, with a ratio of sand to salt of 5 to 1.
29- Epoxy-coated bar in conjunction with latex-modified or low-slump dense concrete overlay, but the practice being discontinued.
30- Average for mountain areas based on salt used for all routes. Primary and secondary roads in mountain areas get 3 times and less than the average salt use, respectively.
31- The part of state excluding mountain areas has 1 to 3 salt applications per year.
32- Limited quantity – 8000 tons per year statewide.
33- Not practiced, unsatisfactory performance.
34- Chemtrete, all bridges.
35- Only on bridge widening.
36- Discontinued about 10 years ago.
37- No longer used.
38- Considers the total number of highway lane-miles for the highest salt use.
39- The average is probably only a little more than zero. Any salt used is usually mixed with sand at a rate of 100 lb. salt per c.y. of sand.
40- Linseed oil sealer for double protection.
41- No longer used, unsatisfactory results.
42- First experimental project advertised.
43- Represents several applications per storm.
44- Standard rehabilitation procedure.
45- Certain locales can have much higher salt application rate.
46- For deck construction and approach slab.
47- Sand-salt mix of approximately 5% salt.
48- 3" cover tried once, as non-experimental.
49- Once, with Chemtrete.
50- 36 storms.
51- Approximately 300 lb. chemicals per mile per application. Yearly number of applications depends on severity of storms and type of road. Yearly, maximum, minimum, and average storms are 20, 8 and 15 storm-days, respectively.

Table F-2. Number of responses from States regarding their involvement in Protective Strategies.

Protective Systems	Standard Procedure [1]	Experimental Procedure [1]	Total 1986	Total 1977 [2]
Concrete Cover ≥ 3"	2	4	6	9
Epoxy Coated Bars	41	2	43	26
Galvanized Bars	0	14	14	11
Low-slump Dense Conc. Overlay	6	16	22	20
Superplasticized Dense Conc. Overlay	2	7	9	---
Latex-modified Concrete Overlay	12	12	24	20
Ultrathin Polymer Concrete Overlay	0	9	9	---
Concrete Sealers	10	16	26	13
Asphalt Conc./ Membrane System	14	7	21	33
Cathodic Protection	0	21	21	5

1. Discontinued or limited use not included (see footnotes for Table F-1).
2. See Ref. 7 for 48 states responding to TRB's 1977 survey.

Table F-3. Responses from States regarding preferred protective strategies.

Strategy \ Popularity * Ranked	I	II	III
Concrete Cover ≥ 3"	3	7	7
Epoxy-coated Bars	37	5	1
Low-slump Dense Conc. Overlay	0	17	5
Latex-modified Conc. Overlay	0	19	4
Asphalt Conc. Membrane System	5	17	6

* (I) = First Preference
(II) = Second Preference
(III) = Third Preference

1977 and 1986. This is also true for cathodic protection, although the latter has been mainly used experimentally on salt-contaminated existing decks. The use of low-slump dense concrete, latex-modified concrete, and galvanized bars has increased slightly. On the other hand, strategies involving bar covers ≥ 3 in. and interlayer membranes are not being used as extensively as they were in 1977. The facts that epoxy-coated bars are easy to use and have performed satisfactorily account for that trend.

Because some states use more than one type of protective system, the survey asked the states to rank their most commonly employed protective systems. The results are given in Table F-3. Almost all of the states responding to the survey reported epoxy-coated bar as the preferred system. Interlayer membranes ranked second, although substantially lower than the epoxy-coated bar. The popularity of low-slump dense and latex-modified concrete was about the same and only slightly lower than that of interlayer membrane. Comparing these results with a similar survey conducted in 1977 (*7*) indicates that epoxy-coated bar is substantially more popular at the present time than it was in 1977. However, states do not favor low-slump dense and latex-modified concrete as much at the present time. The popularity of the interlayer membrane system has stayed about the same, although its use has decreased.

The use and popularity of protective strategies are influenced by both their performance and ease of application. Interestingly, their use by the states can be higher than their popularity among the states. This case was noticed with the interlayer membranes in 1977 (*7*). The strategy was the most used then. A similar case is evident at the present with the epoxy-coated bar, although the difference between its use and popularity is not as large. While 43 states use epoxy-coated bar, 37 consider it the most preferred system (Tables F-2 and F-3). Some states still prefer interlayer membrane over epoxy-coated bar (Table F-3). This is due to a lack of confidence that some agencies still have in the long-term performance of epoxy-coated bar, despite the fact that the system is easy to apply.

QUALITY OF PROTECTIVE STRATEGIES

Detailed information on the quality of protective strategies is provided in Table F-4.

Bare Decks

Some states, especially those in the south, where deicing salts are not applied, still build bare decks containing black steel. However, generally the depth of cover is not less than 2 in. and the specified water/cement ratio is below 0.50.

Table F-4. Survey of properties of bridge deck protective strategies practiced.

State	Bare Decks: Minimum cover (in.)	Bare Decks: Maximum W-C ratio	Bare Decks w/ Epoxy Bars: Minimum cover (in.)	Bare Decks w/ Epoxy Bars: Maximum W-C ratio	Concrete overlaid decks, First stage: Minimum cover (in.)	Concrete overlaid decks, First stage: Maximum W-C ratio	Concrete overlaid decks, Second stage, Minimum overlay thickness: Latex	Concrete overlaid decks, Second stage, Minimum overlay thickness: Low-slump	Decks with interlayer membrane: Type of[+] mem-brane	Decks with interlayer membrane: AC thickness (in.)
Alabama	2 [1]	0.44	2 [1]	0.44						
Alaska			2-1/2/ 2 [2]	0.44					L [3]	2 [3]
Arkansas	2-1/4 [4]	0.44 [4]	2-1/4	0.44						
Connecticut									P *	2-1/2 *
Delaware			2-1/2	0.40	2	0.40	1-1/4			
Florida	2	0.41	2	0.41						
Hawaii	1-1/2									
Idaho			2-1/2 [5]	0.44	1/ 1-1/2	0.44	1-1/2		L & P	2
Illinois			±2-1/4	0.48					P	2
Indiana			2	0.49	2 2	0.49 0.49	1-1/2	 2-1/2		
Iowa			2-1/2	0.41						
Kansas			2-1/2	0.44	1 3/4	0.44 0.44	1-1/4	 2		
Kentucky			2-1/2	0.44	1 1/2	0.44 0.44	1-1/2 *	 2 *		
Louisiana	2	0.48 [6]								
Maine	3	0.46			2	0.46	1-1/4		P	3
Maryland			2-1/2	0.45						
Massachusetts									P	2-1/2
Michigan			3	0.44		0.44	1-1/2 *			
Minnesota	3	0.44	3	0.44	1-1/2 1	0.44 0.44	1-1/2	 2	P	2-1/4
Missouri			3	0.40	1-1/4 3/4	0.44 0.44	1-3/4 *	 2-1/4 *		
Mississippi	2	0.49								
Nebraska	2-1/2 ±1/4	0.44	2-1/2 ±1/4	0.44	1-1/2 (±1/4)	0.44		2		
Nevada	2-1/2	0.53	2-1/2	0.44						
New Hampshire									P *	2 *

\+ Type most commonly used, * In conjunction with epoxy-coated bar only

Legend: L – Liquid applied-in-place membrane, P – Preformed sheet membrane

Table F-4. Continued

State	Bare Decks		Bare Decks w/ Epoxy Bars		Concrete overlaid decks				Decks with interlayer membrane	
					First stage		Second stage			
							Minimum overlay thickness			
	Minimum cover (in.)	Maximum W-C ratio	Minimum cover (in.)	Maximum W-C ratio	Minimum cover (in.)	Maximum W-C ratio	Latex	Low-slump	Type of mem- brane +	AC thickness (in.)
New Jersey			2-1/2	0.44	1-1/2	0.44	1-1/4			
New York			2-1/2	0.44					P	2-1/2
North Carolina	2-1/2	0.43	2-1/2	0.43						
North Dakota			2-1/2	0.44	1	0.44		1-1/2		
Ohio			3	0.50	2	0.50	1-1/4		L	2-1/2
Oklahoma	2	0.49	2	0.49	1	0.49		1-1/2		
Oregon			2-1/2	0.40	1-1/2 1-1/2	0.40 0.40	1-1/2	1-1/2	L	1-1/2
Pennsylvania			2-1/2	0.43	2-1/2 2-1/2	0.43 0.43	1-1/4	2	P	2-1/2
Rhode Island			2-1/2	0.50	2	0.50	1-1/4		P	3
South Carolina	2	0.44								
South Dakota			2-1/2	0.45						
Tennessee			2-1/2						P	3
Texas	2	0.44	2	0.44	2	0.44		1-3/4	L	1-1/2
Utah			2	0.45						
Vermont			2-1/2	0.44					P *	2-1/2 *
Virginia				0.45						
Washington			2-1/2 ±1/4	0.40 ±0.02	1-1/2 ±1/4	0.40 ±.02	1-1/2 *		L *	1.8 *
Wisconsin			2-1/2	0.45/0.42						
Wyoming			2-1/2	0.44						

+ Type most commonly used, * In conjunction with epoxy-coated bar only

Legend: L – Liquid applied-in-place membrane, P – Preformed sheet membrane

Footnotes:

1- In coastal area, epoxy-coated bars; except coastal area, uncoated steel.
2- 2-1/4"/2" precast concrete.
3- Only as repair or retrofit.
4- Rural road, low traffic.
5- 2-1/2"+ chip seal.
6- W-C ratio of 0.43 when water reducer and air-entraining admixtures are used.

Bare Decks with Epoxy-Coated Bars

The depth of cover for this system varies from 2 in. to 3 in. A majority of the states adopt 2½ in. of cover and a majority of the states specify a water/cement ratio of 0.44. However, specified water/cement ratios range from a low of 0.40 to a high of 0.50.

Concrete Overlaid Decks

States generally specify that the water/cement ratio for first stage construction (i.e., the underlying deck) be the same as that for decks containing epoxy-coated bar. The depth of cover for the first stage construction varies among states from 1 to 2 in. for latex-modified concrete overlays and from ½ to 2 in. for low-slump dense concrete overlays. The thickness for the second stage construction (i.e., the overlay) varies from 1¼ to 1½ in. for latex-modified concrete and from 1½ to 2½ in. for low-slump dense concrete overlays.

Interlayer Membrane

States use two interlayer membrane types, a liquid applied-in-place and a preformed sheet membrane. The thickness specified for the asphaltic concrete overlay ranges from 1.8 to 3 in.

TEST METHODS

Table F-5 summarizes the states' use of different bridge deck physical test methods. Except for the rapid chloride permeability test method, which is relatively new and indicates the potential for permeability, most of the states employ all the bridge deck condition determination techniques listed. However, the extent of their use varies considerably, and many states use the test methods shown only in research or special projects. Some techniques, such as visual inspection and delamination detection, are used mainly for diagnosis (i.e., to plan repair or rehabilitation). The remainder are employed to evaluate performance.

Table F-5. Number of responses from States regarding their use of bridge deck physical test methods.

Test Method		Routine Procedure	Experimental Procedure
Visual Inspection	Spalling	44	
	Cracking	44	
	Scaling	42	
	Wear	38	2
Chloride Content Determination		37	7
Delamination Detection		40	3
Half-cell Corrosion Detection		26	14
Electrical Resistivity Testing		13	18
Depth of Bar Cover Survey (Pachometer)		25	16
Rapid Chloride Permeability Testing		5	8

Table F-6. Number of responses from States regarding unsatisfactory conditions on Protected New Bridge Decks.

Protective System \ Data	Chloride Penetration	Bar Half-cell Potential	Deck Concrete Delamination	Overlay Debonding	Overlay Stripping	Concrete Scaling	Surface Wear
Concrete Cover ≥ 3"	MN 1	0	0			MN 1	0
Epoxy-coated Bar	MN 1	0	0			MN 1	0
Low-slump Dense Conc. Overlay	UT 1	0	UT 1	UT 1	UT 1	MN 1	0
Latex-modified Conc. Overlay	IN UT 2	0	IN UT 2	DC IN UT WA 4	UT 1	IN 1	0
Asphalt Conc. Membrane System	CT DC IN 3	CT 1	CT IN 2	DC IN IA MS NY OK VT 7	IN IA MN MS NY OK WA WI 8		CT IL IN IA OK 5

Legend:

CT - Connecticut
DC - District of Columbia
IL - Illinois
IN - Indiana
IA - Iowa
MN - Minnesota
MS - Missouri
NY - New York
OK - Oklahoma
WA - Washington
WI - Wisconsin

PERFORMANCE OF PROTECTIVE STRATEGIES

Nationwide, bridge decks protected immediately after their construction have generally not shown bar corrosion-induced deterioration after 10 or more years of service (Table F-6). Three states have experienced delaminations in some decks protected by low-slump dense concrete overlays, latex-modified concrete overlays, or interlayer membranes (Table F-6).

However, many states have reported dissatisfaction with the durability of bridge deck concrete and with overlays applied to bridge decks. Shown in Table F-6 are the states that have been dissatisfied with certain protective strategies and the types of deck distress experienced. Shown in Table F-7 are the states that have been dissatisfied with the performance of certain types of rehabilitated protected decks. The distress categories used in Table F-7 are generally not dependent on the rehabilitation techniques and represent decks protected immediately after their construction, as well as decks protected after some time in service. From Tables F-6 and F-7, it can be seen that the most frequent problems have been debonding and stripping of asphaltic concrete overlays paved over interlayer membranes, and to a lesser degree, debonding and stripping of latex-modified and low-slump dense concrete overlays. Some states have been dissatisfied with the rate of chloride penetration into concrete overlays. Scaling and wear of concrete surfaces have also been experienced, although not frequently. Surface wear has been especially frequent in asphaltic concrete overlays.

Table F-7. Number of responses from States regarding unsatisfactory conditions on Rehabilitated Protected Decks.

Data / Protective System	Rate of Chloride Penetration in Overlay	Overlay Debonding	Overlay Stripping	Concrete Scaling	Surface Wear
Low-slump Dense Conc. Overlay	NY, SD, UT — 3	MN, MS, NV, TN, UT, WA — 6	TN, UT — 2	MN, WA — 2	WA — 1
Latex-modified Conc. Overlay	IN, NJ, NY, SD, UT, WY — 6	IN, MS, NV, TN, UT, VI, WA, WY — 8	TN, UT — 2	IN, WA — 2	WA, WY — 2
Asphalt Conc. Membrane System		CT, DC, IN, MN, MS, NV, VI, WA — 8	CT, DC, IN, MN, MS, WA — 6		CT, IL, IN, NV, NJ, TN, WA — 7

Legend:

CT - Connecticut
DC - District of Columbia
IL - Illinois
IN - Indiana
MN - Minnesota
MS - Missouri
NV - Nevada
NJ - New Jersey
NY - New York
SD - South Dakota
TN - Tennessee
UT - Utah
VI - Virginia
WA - Washington
WY - Wyoming

APPENDIX G

INTERVIEWS AND FIELD INSPECTIONS OF SELECTED STATE TRANSPORTATION DEPARTMENTS

During the course of this investigation, in the summer and fall of 1986, interviews were held with representatives of the New York, Minnesota, Pennsylvania, and Washington state transportation departments followed by visual inspections of a number of protected bridge decks in each state. The objectives of that portion of this research were (1) to discuss bridge deck design and construction procedures, (2) to discuss any problems in the performances of protected decks, (3) to visually appraise the condition of selected sites, and (4) to explore the possibility of these transportation departments participating in the second phase of the research by presenting sites suitable for detailed field testing and for conducting the associated physical testing.

NEW YORK

Environment

More than 50 freeze-thaw cycles per year can be expected in the southern part of the state. The north, although it has lower temperatures, may experience fewer freeze-thaw cycles. The use of deicing salts is widespread due to the state's severe climate. The maximum salt use in terms of tons per lane-mile per year is about 25–30, the minimum is 5–6, and the average is 15–20 tons per lane-mile per year.

Protective Strategies

Epoxy coating the bars is virtually the only protective system presently used. The top mat reinforcing steel and steel in traffic barriers are epoxy coated. In marine environments the bottom mat steel is also epoxy coated. The cover thickness to the top mat coated bar is specified as 2½ in. and the maximum water/cement ratio for bridge deck concrete is specified as 0.44. The New York State Department of Transportation is evaluating the possibility of epoxy coating both mats on every structure due to the relatively small increase in overall cost. The thickness of the epoxy coating after its curing is specified as 7 ± 2 mils, and the coating is considered continuous when it does not have more than 2 holidays, pinholes not visible to the naked eye, per linear foot of coated bar. The flexibility and adhesion of the coating are tested by bending the bar through 120 deg (after rebound) around a 6-in. diameter mandrel and inspecting the coating on the bent bar for cracking. At the job site any damage to the coating, regardless of its size, is cleaned and repaired with a patching material compatible with the coating.

Coverage with an asphaltic concrete/membrane is the only other protective strategy, besides epoxy coating the bars that is allowed for new bridge deck construction. However, the former strategy is generally not used because debonding and stripping have required periodic removal and replacement of systems. The most commonly used type of interlayer membrane has been a preformed sheet system, which consists of fabric impregnated with asphalt applied to the concrete deck with an asphalt tack coat and overlaid with two asphaltic concrete courses for a total thickness of 2½ in.

The alternative of a bar cover depth equal to 3¼ ± ¼ in. was adopted in 1974 in conjunction with uncoated steel due to a lack of enthusiasm for membranes. That alternative is not presently used. Low-slump dense and latex-modified concrete overlays are used only for existing unprotected decks. A minimum thickness of 2 in. is specified for low-slump dense concrete overlays, and a minimum thickness of 1.5 in. is specified for latex-modified concrete overlays. Double protection, or a combination of epoxy-coated bars and a concrete overlay (low-slump dense or latex-modified concrete), was used on selected bridge decks in the past. However, the practice of double protection is now considered redundant and has been discontinued.

Performance

Performance information was obtained through interviews as well as site visits to a number of protected decks (Table G-1).

Scaling

Concrete scaling has not posed a problem, although a few decks have shown signs of scaling. During the site visits some scaling was noticed on a bare deck built in 1981 with epoxy-coated bars and on a low-slump dense concrete overlay placed in 1983. Latex-modified concrete overlays have shown scaling in small isolated areas of a few decks only, and there has been no extension of the affected areas after about 9 years of service.

In conventional and low-slump dense concrete, lack of air entrainment in some concrete batches has been blamed for scaling in certain areas. One major factor identified as a cause of concrete scaling is late season curing combined with early salting. A 2-month service period has been recognized as the minimum amount of time before decks should be salted. Silane-based sealers are used in late season construction when early salting is anticipated.

Wear

During the site visits, it was noticed that transverse grooves tined into a plastic concrete surface for skid resistance had disappeared in a lane which carried higher traffic than the adjacent lane. The surface was a low-slump dense concrete placed in 1976–1977. To provide better skid qualities, concrete overlays are usually textured with turf dragging instead of tining. Texturing may also be accomplished by cutting grooves about ¼ in. deep in the hardened concrete. For better skid qualities, high friction aggregates are used in cement concrete and asphaltic concrete surfaces.

Cracking

Cracking was detected by visually inspecting a number of bare decks containing epoxy-coated bars as well as decks overlaid with low-slump dense concrete. Construction, material, and structural factors all contributed, more or less, to the cracking. In low-slump dense concrete overlays, false setting cement has been identified, in one case, as the cause of the problem. However, the major factor has been the curing process. Presently, New York's low-slump dense concrete overlays are not seriously cracked, mainly because of a more controlled curing process. The failure to apply wet burlap within 30 min after the concrete has been placed can cause the work to be rejected. Plastic shrinkage in latex-modified concrete is a concern, and the department is investigating the causes of the problem. Cracked areas in concrete overlays are removed and replaced when the depth of the cracking exceeds the mid-depth of the overlay. Otherwise, cracked areas are sealed with a silane-based sealer.

Debonding and Stripping

Debonding and stripping have not been a problem with low-slump dense and latex-modified concrete overlays. Shear bond strength values of 500 to 900 psi have been obtained from latex-modified concrete overlays after about 3 years of service. However, debonding and stripping in asphaltic concrete overlays covering waterproofing membranes have been a problem, requiring removal and replacement of the system every 5 to 8 years. Blistering under the membrane has been identified as a factor contributing to debonding.

Chloride Impermeability

The chloride impermeability of low-slump dense and latex-modified concrete overlays has not been satisfactory. Studies conducted by the New York State Department of Transportation's Research and Develoment Bureau have shown that after about 5 years of service the average chloride profiles in the low-slump dense concrete overlays are about the same as the average chloride profiles in concrete decks built with a specified water/cement ratio of 0.44. The department's future research will include determination of the void structure of low-slump dense concrete mixes and the influence of curing procedures on the chloride impermeability of this special mix.

MINNESOTA

Environment

Despite the state's cold climate, the annual number of freeze-thaw cycles are low. As many as 15 freeze-thaw cycles have been experienced. The maximum and minimum annual deicing salt applications for the roadways are 13.2 and 1.9 tons per lane-mile, with an average value of 4.3. Salt use on bridges, however, is higher due to early morning frost, which requires salt applications during spring and fall periods even when there is no precipitation.

Table G-1. Bridge deck field observations in New York.

Bridge No.	Year Built (State accepted)	Type of Structure	No. of Spans	ADT	Type of Protection (1)	Year Protected	Condition of Deck
1-07086-9 (E. Bound)	1981	Steel beams (continuous)	12	33,434 (1985)	ECR	1980	Wide transverse cracks at 5 to 6 ft. intervals, especially around the supports. Longitudinal cracks in east end. Some concrete disintegration around a patched area close to shoulder.
1-07086-9 (W. Bound)	1981	Steel beams (continuous)	12	33,434 (1985)	ECR	1980	Wide transverse cracks at 5 to 6 ft. intervals, especially around supports. One area with longitudinal and random cracking repaired with epoxy injection. Surface scaling along width of deck in some areas.
1-07247-9	1983	Prestressed concrete beams	3	----	ECR	1984-85	Extensive longitudinal cracks spaced uniformly. Some transverse cracks.
1-07246-9	1983	Steel beams (simple)	1	----	ECR	1983	One small area with map cracking.
1-07309-0	1983	Steel beams (continuous)	2	----	ECR	1983	One transverse crack. One area along the width of the deck with rough surface.
1-09288-1	1970	Steel beams (continuous)	18	33,408 (1985)	LSDC	1983	Random cracking. Transverse grooves wearing.
1-09288-2	1970	Steel beams (continuous)	18	33,408 (1985)	LSDC	1983	Wide transverse cracks at 10 ft. intervals and close to shoulder. Transverse grooves wearing
1-09260-0	1968	Steel beams (simple)	2	12,340 (1980)	LSDC	1976-77	Extensive random cracking. Transverse grooves on the lane with higher ADT worn away.
1-09226-9	1971	Steel beams (simple)	1	21,353 (1976)	LSDC	1983	Narrow map cracking.
1-09228-1	1970	Steel beams (simple)	1	10,968 (1981)	LSDC	1983	Narow map cracking. A small scaled area. Tining not satisfactory.
1-09240-9 (E. Bound)	1971	Steel beams (simple)	1	12,527 (1984)	LSDC	1978	A few isolated cracks.

(1) ECR: Epoxy-coated bar; LSDC: Low-slump dense concrete overlay

Protective Strategies

New bridge decks are protected according to the category in which they fit. New bridge decks have been grouped into three categories, with each deck's protective system (including double protection) designed to be cost effective for its anticipated exposure to deicing chemicals.

Category	Protective System
I. Trunk highways with greater than 10,000 ADT, all interstate highways, all bridges within municipalities.	Epoxy-coated reinforcing bar and special concrete overlay (provide 3 in. of total concrete cover).
II. Trunk highways with 750 to 10,000 ADT.	Epoxy-coated reinforcing bar with 3 in. of structural concrete cover.
III. Trunk highways and all other highways with less than 750 ADT.	Provide 3 in. of structural concrete cover.

Epoxy-coating of bars is in conformance with AASHTO M 284, except that the film thickness is specified as 5 to 10 mils after curing. Recent specifications require epoxy coating of both mats in the decks of the Category I bridges. The specified water/cement ratio of the structural deck is 0.44. One inch of cover is to be provided over the top bar when it is overlaid with low-slump dense concrete and 1½ in. of cover when it is overlaid with latex-modified concrete, for a total of 3 in. of cover. Presently, the number of low-slump dense concrete overlays is higher than the number of latex-modified concrete overlays. Interlayer membranes with asphaltic concrete overlays are seldom used. Preformed membranes with 2¼ in. of asphaltic concrete overlay have been installed on some bridges in the past.

Performance

Information on the performance of the protected decks was obtained during the interviews and site visits (Table G-2).

Scaling and Wear

Wear has not been a problem, mainly because studded tires were banned in the early 1970s. Scaling in conventional concrete and low-slump dense concrete was noticed in some decks during the site visits. Most of the scaling had occurred in the gutter areas. In some decks the roadside surface of traffic barriers had also scaled. However, the deck scaling has not progressed far and has been limited to the upper concrete layer. To prevent scaling, conventional concrete is treated with linseed oil during late-season construction when the deck might be salted soon after construction.

Debonding and Stripping

Debonding in concrete overlays has been insignificant. Only two low-slump dense concrete overlays have debonded significantly; one of these had an epoxy bonding agent which hardened unexpectedly and caused the problem. In order to provide a satisfactory bond between the concrete and the overlay, the surface of the structural deck is textured in its plastic stage and sand blasted before the overlay is applied. The bonding agent is a grout scrubbed on the deck. Unlike concrete overlays, asphaltic concrete overlays covering the interlayer membranes have been debonded and stripped off the deck, limiting the deck's service period to about 10 years.

Chloride Permeability and Cracking

The chloride permeability of latex-modified concrete is considered good. Low-slump dense concrete overlays have not been totally chloride-proof. For example, data collected by the department gave an average of 1.88 lb/cu yd of chlorides in the top ½ in. of an underlying deck 9 years after it was overlaid with a 2-in. low-slump dense concrete (Bridge No. 27895). The deck was opened to traffic and salted after the overlay was installed. The chloride proofing abilities of membranes have been satisfactory, and basically no changes in the chloride content of decks covered by this sytem have been noticed after 10 years of performance.

Present cracking in low-slump dense and latex-modified concrete overlays does not pose a problem. Timely covering and curing of the overlays has been a factor in keeping the cracks at low levels. Some of the cracks have originated from the joints in the barriers. Minnesota has also been concerned over repeated flexing of structures and its role in propagating existing cracks. Cracks in concrete decks are usually sealed with a low-viscosity epoxy fed through the cracks by gravity.

PENNSYLVANIA

Environment

In the north of the state, annual freeze-thaw cycles can number well above 100. In central and southern Pennsylvania the annual freeze-thaw cycles can number around 50. The application of deicing salts is widespread in the State. The maximum and minimum number of salt applications per year on the bridge decks in Pennsylvania are approximately 50 and 10, with an average of 30.

Protective Strategies

Epoxy coating of bars is the only system presently specified for the protection of newly constructed bridge decks. Although in the past only top mat bars were epoxy coated, currently bottom mat bars, bars in parapets, and bars in pier caps are also epoxy coated. The possibility exists that pier caps located under expansion joints will be exposed to saline due to the unsatisfactory performance of water-proofing joints. Bars are epoxy coated in conformance with AASHTO M284. Two and one-half inches of clear concrete cover are built above the uppermost bar in the deck, and the maximum water/cement ratio specified for deck concrete is 0.43.

Although presently standard procedure for the protection of existing bridge decks, the practice of protecting newly constructed decks with latex-modified concrete overlays, low-slump dense concrete overlays, and asphaltic concrete/interlayer mem-

Table G-2. Bridge deck field observations in Minnesota.

Bridge No.	Year Built	Type of Structure	No. of Spans	ADT	Type of Protection (1)	Year Protected	Condition of Deck
9116	1964	Prestressed beams (continuous)	4	3,900	LSDC (2)	1984	Overlay scaled. Some longitudinal cracking.
9082	1958	Prestressed beams (simple)	2	24,900	LSDC	1976	Transverse cracks sealed with epoxy.
9081	1958	Prestressed beams (simple)	2	24,900	LSDC	1976	Transverse and random cracks sealed with epoxy.
9130	1960	Steel beams (simple)	4	52,350	LSDC (3)	1976	Cracks sealed with epoxy. Traction grooves worn in wheel tracks. A few leaching transverse cracks on underside of deck.
9131	1960	Steel beams (simple)	4	52,350	LSDC (3)	1976	Cracks sealed with epoxy. A few leaching transverse cracks on underside of deck.
27888	1972	Steel beams (continuous)	3	34,500	LSDC	1975 (4)	Deck scaling in the gutter area and barrier scaling.
27893	1972	Prestressed beams (continuous)	3	63,500	LSDC	1975 (4)	Small areas of scaling in gutter area and scaling of the barrier.
27895	1971	Prestressed beams (continuous)	2	63,500	LSDC	1975 (4)	Good condition.
27887	1972	Steel beams (continuous)	4	34,500	LSDC	1975 (4)	Some scaling in gutter areas and scaling in the barrier.
9528	1967	Prestressed beams (simple)	2	6,100	LSDC (5)	1986	Some random cracking.
62865	1971	Steel beams (continuous)	3	6,600	LSDC	1983 (4)	Some pattern cracking in one area.
19859	1979	Steel beams (simple)	6	12,500	LMC, ECR	1980	Good condition.

(1) ECR: Epoxy-coated bar
LSDC: Low-slump dense concrete overlay
LMC: Latex-modified concrete overlay

(2) Overlaying slotted cathodic protection system.
(3) Pozzolanic overlay, deck with galvanized bar.
(4) Opened to traffic after overlayed.
(5) An experimental migrating corrosion inhibiter admixed with overlay.

Table G-2. Continued.

Bridge No.	Year Built	Type of Structure	No. of Spans	ADT	Type of Protection (1)	Year Protected	Condition of Deck
19860	1979	Prestressed beams (simple)	6	12,500	LMC, ECR	1980	Scaling in gutter area and traffic barrier.
19825	1973	Prestressed beams (simple)	7	17,500	LMC (6)	1973	Good condition. Only two transverse cracks. No traction grooves built.
9779	1959	Steel beams (continuous)	2	19,500	LMC	1976	Two transverse cracks and one diagonal crack. Traction grooves wearing away.
9613	1962	Steel beams (continuous)	4	71,250	ACM	1971	Overlay patched all over. A few leaching cracks on underside of deck.
9614	1962	Steel beams (continuous)	4	71,250	ACM	1975	Deck has recently been resurfaced with an asphalt overlay.
27933	1964	—	3	123,100	ACM	1975	Deck has recently been resurfaced with an asphalt overlay.

Bridge No.	Year Built	Type of Structure	No. of Spans	ADT	Type of Protection (1)	Year Protected	Condition of Deck
9742	1962	—	4	24,750	ACM	1975	Deck has recently been resurfaced with an asphalt overlay.
9741	1962	—	4	24,750	ACM	1975	Deck has recently been resurfaced with an asphalt overlay.
27062	1978	Steel beams	2	10,600	ECR	1978	Deck scaled. Transverse cracks sealed with epoxy.
19015	1973	Prestressed beams (simple)	3	10,550	ECR	1973	Scaling in one area along the width of deck. Fine pattern cracking.
19016	1973	Prestressed beams (simple)	3	10,550	Black Steel, 2-1/2" cover	—	Scaling in one location along the width of deck.

(1) ECR: Epoxy-coated bar
LMC: Latex-modified concrete overlay
ACM: Asphalt concrete – membrane

(6) 3/4" mortar overlay.

branes was discontinued about 10 years ago when epoxy-coated bars were introduced for the construction of new decks. The practice generally involved the application of concrete overlays on scarified decks designed with 2.5 in. of initial concrete cover and a water/cement ratio of 0.43. This procedure required the application of 1¼ in. of latex-modified concrete overlay or 2 in. of low-slump dense concrete overlay. A specially mixed grout was used as a bonding element. When interlayer membranes were used, preformed sheet systems were overlaid with 2.5 in. of asphaltic concrete.

Performance

Performance information about the protected decks was collected from interviews with department personnel, site visits, and the department's general bridge deck evaluation program (Table G-3). The decks included in Table G-3 were protected either during their construction or before salt applications.

Table G-3. Bridge deck field evaluation in Pennsylvania in 1986.

Bridge No.	Year Built	Type of Protection (1)	Crack Type	Crack Rating (2)	Crack Comments	Surface Defects Comments	Rust Comments
LR 1073, I-380 WB	1982	LMC	Map	1	—	—	—
LR 1036, US 15 SB	1974	LMC	Map Transverse	1 1	—	—	Along transverse joint
LR 1012, I-84 EB	1977	LMC	Map	3	Tight	1 spall @ 1 s.f., 1 patch @ 4 s.f., & 1 patch @ 2 s.f. along joint	—
LR 1012, I-84 WB	1977	LMC	Map Long.	3 1	Tight Some with epoxy injection	Joint spalling on first exp. dam.	—
LR 1081, 283 EB	1978	LMC	—	0	—	—	—
LR 168, 435	1978	LMC	Map	1	Very few and tight	Spalls mostly on NB side and spalling in last joint NB.	Rust stains in center of spall.
LR 793, US 422 EB	1977	LMC	Block Transverse Map	4 5 1	—	Asphalt patch 2 s.f. & concrete patch 1.5 s.f.	—
LR 1011, I-176 NB	1977	LMC	Transverse Map	5 5 & 2	—	One large agg. pop-out.	—

(1) LMC: Latex-modified concrete overlay

(2)

Crack Intensity	Crack Extension: None	<10%	10-30%	>30%
> Hairline	0	4	5	6
Hairline	0	1	2	3

Table G-3. Continued

Bridge No.	Year Built	Type of Protection (1)	Crack Type	Crack Rating (2)	Crack Comments	Surface Defects Comments	Rust Comments
LR 793, US 422 WB	1977	LMC	Transverse	6	6" to 3' spacing over entire deck	Joint spalls patched: 2 asph. patches @ 8 s.f. & 2 conc. patches @12 s.f.	——
LR 141, I-78 & US 22	1977	LMC	Diagonal	4	West bound	Popout west bound.	One spot west bound.
			Transverse	4			
LR 1073, I-180 WB	1976	ECR	None	0	——	One small patch.	——
LR 1002, I-380 NB	1974	ECR	Transverse	2	——	Includes spalling, 6.5 s.f. patched.	High steel with spall, one area epoxy coat worn off.
			Map	1			
LR 1002, I-380 SB	1974	ECR	Transverse	2	——	——	——
			Map	1			
LR 184, US 94	1982	ECR	Transverse	2	Mostly transverse.	Popouts & surface defects due to poor finish.	——
			Longitudinal	1			
LR 1072, US 222	1974	ECR	Transverse	4	Mostly in travel lane, appr. 5' spacing.	Large spalls in travel lane & smaller spalls in passing lane.	——
			Map	1	Mostly in passing lane.		
LR 784	1978	ECR	Long.	4	In right wheel path travel lane EB center span.	——	Appeared to be minimal, west bound half of deck.
			Map	3	Very tight.		
LR 779, PA 100	1974	ACM	——	—	——	Bridge just recently overlayed.	——
LR 555, 145	1975	ACM	——	0	——	Wear and exposed aggregate.	——

(1) LMC: Latex-modified concrete overlay
ECR: Epoxy-coated bar
(1) ACM: Asphalt concrete-membrane.

(2)

Crack Extension: None	<10%	10-30%	>30%	Crack Intensity
0	4	5	6	> Hairline
0	1	2	3	Hairline

Scaling and Wear

Scaling in conventional, low-slump dense or latex-modified concrete surfaces has occurred rarely and only in isolated and localized areas. The scaling has been attributed to construction practices. However, low-slump dense concrete and latex-modified mortars have shown relatively higher scaling than latex-modified concrete.

Use of studded tires was disallowed for a period of time. However, in 1978 use was allowed again and wear of the surface under traffic was found to be less than that experienced previously. This lack of wear is mainly attributed to smaller cars, more effective regular tires, and the public's concern for damage to the pavement's surface. For texturing concrete surfaces, Pennsylvania specifies tining plastic concrete. However, in practice the concrete surface is often broomed.

Cracking

During an on-site visit, hairline, random shrinkage cracking was inspected on a 12-year-old latex-modified concrete overlay (Bridge LR 1005, I-81 NB), a 12-year-old bare deck containing epoxy-coated bars (Bridge LR 1005, I-81 SB), and a bare section of a 12-year-old deck containing galvanized bars (Bridge LR 1000, SB ramp to I-95). Information from Pennsylvania's general bridge deck condition evaluation program (Table G-3) indicated that transverse cracking was mainly evident in bare concrete decks (epoxy-coated bars) and map cracking (caused generally by shrinkage) in the concrete overlays (latex-modified and low-slump dense concrete). The department is researching the effectiveness of using low-viscosity polymers in sealing concrete cracks. Rubberized-asphalt sealant, however, is routinely used by maintenance crews to seal wide cracks in concrete.

Debonding and Stripping

Although debonding has not posed a problem, cases of debonding have occurred in concrete overlays. During an on-site visit, chain dragging on 12-year-old latex-modified concrete applied on a deck containing galvanized steel (Bridge LR 1000, SB ramp to I-95) revealed debonding near the expansion dam as well as in and around the overlays, which were saw cut and patched with latex-modified concrete. The saw cutting and patching were likely performed during the deck's construction because of an initially unsatisfactory bond. Asphaltic concrete overlays covering interlayer membranes, on the other hand, had debonded and stripped off the deck, especially when subjected to heavy traffic with a high percentage of trucks.

Corrosion Protection Characteristics

The data obtained through the department's general bridge deck condition evaluation program (Table G-3) show some incidents of spalling and patching associated with latex-modified concrete overlays applied on uncontaminated decks as well as on bare decks containing epoxy-coated bars. However, detailed field investigations are needed to determine if the problem was caused by corrosion in the reinforcing steel.

WASHINGTON

Environment

Washington State has three distinct climates: wet western, cold mountain, and dry eastern. Annual freeze-thaw cycles can number over 50 in Eastern Washington. For Western Washington, however, fewer than 50 annual freeze-thaw cycles can be expected. The department's deicing salt usage has been reduced by approximately 80 percent over the past 10 years. The department's current maximum salt use is about 2.5 tons per lane-mile per year. However, this figure does not include the salt use of certain local agencies. Also, bridges, because of their special environment, may be salted more than roadways.

Protective Strategies

Presently, epoxy-coating top mat bars in conjunction with 2½ in. of concrete over the epoxy-coated bars is required for all deck constructions. The 2½-in. cover includes 0.2 in. of traction striations in the roadway surface and a ¼-in. tolerance for the placement of reinforcing steel. The water/cement ratio of deck concrete is specified at 0.40 ± 0.02. In addition to the top slab reinforcing steel, traffic barrier bars are also epoxy coated. Beam or diaphragm stirrups, when reaching the top mat bar, are not epoxy coated. However, they are secured with 135-deg hooks. The thickness, continuity, and flexibility of the epoxy-coating are according to AASHTO M284. The maximum amount of damage to the coating is limited to 0.25 percent of the surface area of each bar. After placement of the coated bars and before placement of the concrete, the coated bars are inspected and patched with a compatible epoxy material in areas showing damage that exceeds the following size limitations:

1. An area of 0.05 sq in.
2. An area of 0.012 sq in. if the opening is within ¼ in. of another opening of the same or a larger size.
3. Damage 6 in. in length regardless of width or area.
4. An aggregate area of 0.50 sq in. in any 1-ft length.

The use of only latex-modified concrete, low-slump dense concrete, or asphaltic concrete/membrane to protect new decks has been discontinued. However, the use of 1½ in. of latex-modified concrete overlay is the prime strategy for protecting existing unprotected decks. Inter-layer membranes rank second for protecting existing decks. Generally a liquid, applied-in-place rubberized asphalt membrane in conjunction with 0.15-ft-thick asphaltic concrete is used for this purpose. Before the asphaltic concrete is placed, however, the membrane is covered with a polypropylene material. Double protection, or the combination of a top mat epoxy-coated bar and a latex-modified concrete overlay or an interlayer membrane, may be used when a bridge is a vital transportation link or repairs to the structure may be extremely difficult. Double protection with latex-modified concrete consists of a 1¾-in. design concrete cover with 1½ in. of overlay. The design concrete cover includes ¼ in. for scarifying the concrete deck for bond. Double protection with an interlayer membrane consists of 2 in. of concrete cover with 0.15 ft of asphaltic concrete overlay.

Table G-4. Field observations in Washington.

Bridge No.	Year Built	Type of Structure	No. of Spans	ADT	Type of (1) Protection	Year Protected	Condition of Deck
82/145	1978	Prestressed box girder (continuous)	2	120	ECR	1978	Diagonal cracks concentrated in the intermediate support and to a lower degree in the two ends.
82/147 S	1978	Prestressed beams (continuous)	3	9,558	ECR	1978	Numerous transverse cracks distributed over the deck.
80/147 N	1978	Prestressed beams (continuous)	3	9,558	ECR	1978	Transverse cracks over the deck but not extensive.
82/149	1978	Prestressed box girder (continuous)	2	120	ECR	1978	Good condition.
5/113	1970	Steel beams (simple)	2	3,000	ACM	1970	A longitudinal joint appears as a crack.
5/112 E	1971	Concrete box girder (continuous)	3	44,748	ACM	1971	Cracking of overlay at pavement seats. Longitudinal construction joints widened.
5/112 W	1970	Concrete box girder (continuous)	3	44,748	ACM	1970	Cracking of overlay at pavement seats. Longitudinal construction joints widened.
5/107 E	1970	Prestressed beams (continuous)	3	42,496	ACM	1970	Good condition.
5/107 W	1970	Prestressed beams (continuous)	3	42,496	ACM	1970	Good condition. Three transverse cracks with efflorescence underneath the deck.
82/30 N	1970	Concrete box girder (continuous)	5	9,000	ACM	1970	Asphalt overlay was removed, deck was scarified for a concrete overlay.
82/106 S	1969	Prestressed beams (simple)	1	9,516	ACM	1969	Good condition. Old surface overlaid with 1 inch asphalt concrete in 1984.
82/106 N	1969	Prestressed beams (simple)	1	9,516	ACM	1969	Good condition. Old surface overlaid with 1 inch asphalt concrete in 1984.

(1) ECR: Epoxy-coated bar; ACM: Asphalt concrete – membrane

Performance

Performance information was collected during interviews and from the department's bridge deck research program. Some information was also obtained during site visits (Table G-4).

Scaling and Wear

Scaling has been detected in some installations of both latex-modified and low-slump dense concrete overlays. The problem has been especially acute in gutter areas. The total disappearance of traction grooves has been noticed on concrete overlays after 7 years of service under an average daily traffic of about 10,000 and use of studded tires. Rut depth measurements are included in the routine survey of bridge decks. Some bare concrete decks in the Seattle area have shown rut depths of ½ in. or more after 20 years of service. Rutting has been more severe on asphaltic concrete overlays. Some bridges overlaid with asphalt/concrete membranes have had about 1 in. of ruts due to both wear and lateral movement of the asphaltic concrete under wheel loads. The department is planning to investigate methods for partially overlaying wheel ruts on bridge decks.

Debonding and Stripping

Debonding has occurred on some latex-modified and low-slump dense concrete installations immediately after overlaying and curing. The department may specify use of the overlay mix as a bond coat on scarified surfaces (wetted for latex-modified concrete) and the removal of coarse aggregate from the mix as the mix is broomed on the deck. After curing, the overlays are chain-dragged for debonding before acceptance. The debonded areas, if they exist, are removed and replaced. While stripping of asphaltic concrete overlays paved on liquid, applied-in-place membranes has been a problem in some installations, others have performed satisfactorily after 16 years of service, even under heavy traffic.

Chloride Impermeability and Cracking

Investigations by the department have shown that a rubberized asphalt membrane is effective in preventing chloride intrusion. The results of rapid chloride permeability tests performed on field samples from Washington installations brought the chloride impermeability of low-slump dense concrete overlays into question. Thus, the practice of overlaying with this type of concrete was discontinued in 1984 pending further investigations. Currently, the department is investigating both latex-modified and low-slump dense concrete overlays, including their chloride impermeability characteristics.

Regardless of the chloride permeability of sound concrete, the permeability of cracked concrete has been a concern. Both latex-modified concrete and low-slump dense concrete overlays have cracked in some installations. After 5 to 7 years of service, cracks in some overlays have reached the bonded interface and in some instances they have passed the interface due to the integrity between the overlay and the substrate. Initial overlay cracking has generally been considered plastic shrinkage cracking. The department's experience with latex-modified concrete has shown that continuously wetting the substrate to reduce its temperature as well as increasing the target slump to about 5.5 in. may minimize cracking. Cracks in latex-modified concrete may also be minimized by controlling curing procedures more carefully. The overlay should be covered by thoroughly saturated burlap as soon as possible. Ten to 20 ft of exposed concrete behind the screed has been suggested as a limiting value. Air gaps and wrinkles under the burlap may also cause cracking, due either to differential temperatures or evaporation. Forty-eight-hour wet curing instead of 24-hour curing is also suggested. The department requires a maximum evaporation rate of 0.15 to 0.20 lb/ft^2 when a latex-modified concrete is being overlaid. Cracks in the latex-modified concrete, when they occur, are sealed either by covering them with a slurry made of latex, cement, and water (sand may also be added), or by a polymer material (mainly epoxy) fed through the cracks by gravity. The latter procedure involves broadcasting sand over the tacky polymer for skid resistance. The slurry, however, is cosmetic, and recently use of the polymer material has been considered the only effective means for water and chloride-proofing the cracks.

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