SHRP-S-360

Concrete Bridge Protection, Repair, and Rehabilitation Relative to Reinforcement Corrosion: A Methods Application Manual

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Strategic Highway Research Program
National Research Council
Washington, DC 1993

SHRP-S-360
ISBN 0-309-05616-0
Contract C-103
Product No. 2036

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October 1993

key words:
bridges
concrete
concrete removal
corrosion
costs
protection
rehabilitation
repair
service lives

Strategic Highway Research Program
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1.5M/NAP/1093

Acknowledgments

This work would not have been possible without the enormous contributions of the bridge, materials, and maintenance engineers of the state departments of transportation. We also thank the SHRP state coordinators, who graciously assisted us by providing source information contained in this document. Special consideration is extended to SHRP staff, the Expert Task Group, the Advisory Committee, and Technical Contract Manager Mr. Joseph Lamond.

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Abstract

This manual is intended as a practical guide for state highway agency personnel who are faced with the day-to-day task of cost-effectively protecting, repairing, and rehabilitating concrete bridges exposed to chloride-laden environments. As a practical guide, the manual addresses the chloride-induced corrosion of the reinforcing steel because the protection, repair, and rehabilitation methods presented are based on a working knowledge of the corrosion processes. Methods are presented to estimate the service life and remaining service life of concrete bridge components. Economic models are presented to enable selection of the most cost-effective methods (i.e., those with minimum life-cycle cost) from the menu of protection, repair, and rehabilitation methods.

These methods include standard physical, chemical, and experimental protection, repair, and rehabilitation methods. Each method is described with respect to limitations, estimated service life, estimated construction price or cost, construction procedures, quality assurance and construction inspection methods, and material performance specifications. In addition, rapid bridge deck protection, repair, and rehabilitation methods are presented.

Two mechanized concrete removal methods, milling and hydrodemolition, are compared to the traditional method, pneumatic breakers. The three concrete removal methods are discussed with respect to labor- and capital-intensive operations, work characteristics, and quality management and control. The advantages of combining the strengths of the three removal methods are also presented.

Executive Summary

Chloride-ion-induced corrosion damage of reinforced concrete bridges is the single most costly deterioration mechanism facing state highway agencies in the United States. Approximately 40% of the current backlog of highway bridge repair and rehabilitation costs is directly attributed to the chloride-ion-induced corrosion of steel-reinforced concrete bridge components. This manual is presented as a practical guide to state highway agencies that are faced with the day-to-day task of protecting, repairing, and rehabilitating concrete bridge components exposed to chloride-ion-laden environments.

Limited resources demand that public facilities be maintained in a cost-effective manner, with minimum life-cycle cost. To minimize the life-cycle costs of concrete bridge components exposed to chloride environments, one must have a working knowledge of the corrosion mechanism, be able to estimate service lives of new or rehabilitated bridge components, estimate the remaining service life of existing bridge components, and know the service life of protection, repair, and rehabilitation methods. In addition, construction prices must also be estimated. With estimates of service lives and construction prices, standard engineering economic analysis can be used to select the most cost-effective protection, repair, and rehabilitation methods.

This manual presents the processes and mechanisms that initiate the chloride-induced corrosion of reinforcing steel in concrete. The application of the physical and chemical methods used to protect, repair, and rehabilitate concrete bridge components is founded on knowledge of these corrosion processes. The processes are used to develop deterioration models that can be used to estimate service life and remaining service life. Also presented are economic models for the replacement and protection, repair, and rehabilitation of bridges and together with the service life models the economic models can be used to select the most cost-effective strategy. Examples are presented to illustrate how the models are used. Electrochemical methods are not discussed.

The manual also presents standard and experimental protection, repair, and rehabilitation methods for concrete bridge components exposed to chloride-laden environments. For special cases in which traffic disturbance must be kept at a minimum, rapid methods for bridge deck protection, repair, and rehabilitation are presented. Protection methods include concrete sealers, coatings, membranes, and polymer concretes. Repair methods include patching with portland cement concrete, polymer concrete, and high-early-strength hydraulic cement concretes. Shotcreting methods are also presented. Latex-modified, low-slump dense, microsilica, and polymer concretes and asphaltic concrete over preformed membranes are used in both repair and rehabilitation methods; these methods include patching, overlays, encasement, and jacketing. The rehabilitation methods present criteria and procedures for increasing or maximizing service over the repair procedures. Each method is described with respect to limitations, estimated service life in various chloride environments, estimated construction price or cost, construction procedures, and quality assurance and control procedures.

Concrete removal techniques using pneumatic breakers, milling machines, and hydrodemolition are presented, as are the strengths of combining these techniques. Descriptions, work characteristics, and quality management and control procedures are presented for the three concrete removal techniques. The utility of each method is demonstrated in removing concrete from bridge components: decks, substructure elements, and superstructure elements.

1

Introduction

1.1 Background

Chloride-ion-induced corrosion of the reinforcing steel is the most destructive cause of the early deterioration of concrete bridges in the United States; hence its inclusion in the Strategic Highway Research Program (SHRP). The SHRP structural concrete research plan included four topic areas:

1. Condition Assessment Techniques

2. Electrochemical Rehabilitation Methods

3. Physical and Chemical Protection, Repair and Rehabilitation Methods

4. Methods Selection Decision Model

The capstone project, the decision model, determines from a menu of chemical, electrical, and physical methods the most cost-effective protection, repair, and rehabilitation methods for the assessed condition of a bridge. This manual presents, in summary form as a research implementation report, the results of the research project, "Concrete Bridge Protection and Rehabilitation: Chemical and Physical Techniques."

For further insight into the methods, procedures, and processes presented in this manual, readers are referred to the seven other SHRP research findings reports included under the "Concrete Bridge Protection and Rehabilitation: Chemical and Physical Techniques" project:

1. Service Life Estimates (1)
2. Price and Cost Information (2)
3. Feasibility Studies of New Rehabilitation Techniques (3)

4. Techniques for Concrete Removal and Bar Cleaning on Bridge Rehabilitation Projects (4)
5. Rapid Concrete Bridge Deck Protection, Repair, and Rehabilitation (5)
6. Corrosion Inhibitors and Polymers (6)
7. Field Validation (7)

1.2 Definitions

Certain terms in this manual have specific meanings with respect to the processes discussed. Users of this manual should make these definitions part of their working vocabulary.

Protection method: a non-electrochemical method used to significantly reduce the rate of ingress of chloride ions into concrete. Protection methods are limited to concrete elements that are <u>not critically contaminated</u> with chloride. Sealers, coatings, and polymer overlays are normally thought of as protection methods. However, hydraulic cement concrete overlays constructed with low-slump dense, microsilica, or latex-modified concrete can be used as protection methods.

Repair method: a method that restores a deteriorated concrete element to a service level equal to or almost equal to the as-built condition. No effort is made to prevent or significantly retard deterioration mechanisms. A typical example is patching a concrete bridge deck with hydraulic cement concrete where the surrounding concrete is above, at, or near the chloride threshold level and where corrosion will accelerate along the perimeter of the patch. Another example, which is normally thought of as a <u>rehabilitation</u> method but is really a <u>repair</u> method, is the overlaying method used by most transportation agencies: the overlaying of a bridge deck where the top one-half inch of the deck is milled off, spalled and patched areas are repaired, and the deck is overlaid with low-permeability concrete, but chloride-contaminated concrete is left in place. Corrosion continues under the overlay. Thus, the deck is considered <u>repaired</u> because no efforts were made to significantly reduce the corrosion deterioration process.

Rehabilitation method: a method that corrects the deficiency that resulted in the assessed deteriorated condition. A typical example is the overlaying of a bridge deck with microsilica concrete where one-half inch of the top surface of the concrete is milled off, spalled areas are repaired, delaminated concrete is removed and repaired, all areas where the corrosion potentials are more negative than 250 mV as measured by a copper copper sulfate half-cell are removed and repaired, and a microsilica concrete is placed over the entire deck. The original deficiency has been corrected by removing all the concrete that will continue to corrode the reinforcing steel under the overlay, and the more chloride-permeable as-built concrete has been replaced with a low-permeability concrete. Thus, rehabilitation has been achieved: the more permeable and chloride-contaminated concrete has been replaced with a less permeable concretes and thus the service life of the deck has been significantly increased.

Critical chloride contamination: the degree of chloride contamination of the cover concrete such that after the concrete is protected, the chloride content at the reinforcing steel level will come to an equilibrium value of at least 0.2 lb of acid-soluble chloride per cubic yard of concrete (0.12 kg/m^3) less than the corrosion threshold level (total acid-soluble minus the acid-soluble background level).

Corrosion chloride threshold level: the degree of chloride contamination of concrete that will activate the corrosion process. The chloride threshold level is estimated at 1.2 lb of acid-soluble chloride (total minus background) per cubic yard of concrete (0.71 kg/m^3). Note: that this is the contamination level. Concrete aggregates contain some acid-soluble chloride that may or may not participate in the corrosion process. The acid-soluble chloride content of the aggregates is commonly referred to as the background chloride content. A typical chloride content of concrete aggregates in some parts of the United States is 0.5 lb of acid-soluble chloride per cubic yard of concrete (0.29 kg/m^3) (8). Thus, a reasonable chloride threshold level estimate is 1.7 lb of acid-soluble chloride per cubic yard of concrete (1.0 kg/m^3) if the background chloride content is not available.

Deck: traffic riding surface.

Superstructure: beams or girders and diaphragms that support the deck.

Substructure: piers, pier caps, or columns that support the superstructure elements.

Cost: determined by classical engineering estimating techniques. Cost is the sum of all materials, labor, equipment, mobilization, and engineering costs at the specified reference time. Effects of regional economy, and environmental impact are not included in the cost, nor is the effect of construction on road users. The costs presented in this report are the national average costs for mid-year 1991.

Price: determined by a systematic examination of archival prices that selected departments of transportation paid for the work performed. Price includes the effects of regional economics and profit. The prices presented in this report are the national average prices for mid-year 1991.

1.3 Scope and Purpose

This manual addresses the protection, repair, and rehabilitation of the single most destructive deterioration mechanism for reinforced concrete bridges in the United States: corrosion of reinforcing steel. In this endeavor, standard and new (experimental) protection, repair, and rehabilitation methods are presented. Limitations, estimated service lives, costs or prices, construction procedures, quality assurance and inspection programs, and construction methods and material specifications are included for each method. In addition to standard and experimental methods, rapid methods for bridge deck protection, repair, and rehabilitation are presented. Finally, for concrete removal, pneumatic breakers, milling, and hydrodemolition and combined concrete removal methods are addressed as they relate to the repair and rehabilitation of chloride-induced corrosion deterioration of reinforced concrete bridge components.

A procedure is included for selecting the most cost-effective method based on life-cycle costing for those projects that may not require, or for which transportation agencies may not wish to employ, the full decision model. Also included is a primer on the corrosion deterioration process for reinforced concrete bridge components exposed to chloride-ion-bearing environments. Understanding how corrosion parameters for steel in concrete influence the deterioration process is mandatory for the effective application of the methods presented in this manual.

This manual is presented as an application guide to those faced with the day-to-day task of cost-effectively protecting, repairing, and rehabilitating concrete bridges exposed to chloride-ion-bearing environments. Procedures must be strictly followed to maximize the service life of the presented methods. In this way, transportation agencies will maximize the benefit and minimize the life-cycle costs of the methods presented for their clients, the general public.

1.4 Report Structure

This manual is intended as a user's guide for transportation agency personnel who are responsible for the day-to-day task of cost-effectively maintaining our nation's reinforced concrete bridges. The manual addresses a single deterioration mode of concrete bridges: chloride-ion-induced corrosion of the reinforcing steel. In this regard, chapter 2 includes a primer on the chloride-ion-induced corrosion of steel in concrete because a working understanding of the corrosion process is necessary to fully appreciate the procedures presented in the manual. The objective of each procedure is to maximize the service life of protection, repair, and rehabilitation methods included in the manual. Only in chapter 4 are methods discussed in which service life is sacrificed for time in the rapid protection, repair, and rehabilitation of concrete bridge decks. In chapter 3, methods are presented in which overlays are placed over chloride-contaminated concrete, but these methods are identified as repair methods, not rehabilitation methods, because the cause of the deterioration has not been addressed.

Chapter 2 also includes an economic decision aid for selecting the most cost-effective (minimum life-cycle cost) methods for maintaining concrete bridges exposed to chloride-laden environments. Deterioration models are also presented in chapter 2 for unprotected bare concrete and for overlays constructed with low-slump dense concrete (LSDC), latex-modified concrete (LMC), and microsilica concrete (MSC). These models can be used to determine the effects of delayed rehabilitation and to estimate the time to rehabilitation.

Chapters 3 and 4 present standard and experimental methods, respectively, for the protection, repair, and rehabilitation of reinforced concrete bridge components exposed to chloride ions. Each method is discussed with respect to limitations, step-by-step construction procedures, material specifications, price or cost, and estimated service life. The price or cost is based on the national average for mid-year 1991. For estimating price or cost for a different time or location, the reader is referred to the report " Price and Cost Information" (2). Estimated service lives are presented for various environmental exposure conditions rated extremely severe, severe, moderate, and mild.

Chapter 5, "Rapid Deck Treatment Methods," has the same structure as chapter 3, "Standard Methods," and chapter 4, "Experimental Methods."

Chapter 6, "Concrete Removal Methods," addresses the equipment, work characteristics, and quality management and control of pneumatic breakers, milling machines, and hydrodemolition methods for concrete removal for repair and rehabilitation projects. The chapter also presents the benefits of combining such technologies as milling and breakers; hydrodemolition and breakers; and milling, hydrodemolition, and breakers. Concrete removal technology selection criteria are also presented in chapter 6.

For ease of use, each chapter includes its own references.

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2

Method Selection

2.1 Background

This chapter presents a primer on the corrosion of steel in concrete, the rate of deterioration of reinforced concrete bridge components exposed to chloride ions, and a method for selecting cost-effective (minimum life-cycle cost) methods for the protection, repair, and rehabilitation of concrete bridge components exposed to chloride ions. These three independent but interrelated areas are presented as basic information to the manual's users, state departments of transportation and local government personnel who are charged with the day-to-day task of maintaining the nation's concrete bridges.

A basic understanding of the corrosion mechanism of reinforcing steel in concrete provides the knowledge needed for addressing the cause of the deterioration, not just the symptom. By addressing the cause, the user will maximize the service life of the selected methods. A description of the basic corrosion deterioration processes provides a means to estimate the rate of deterioration and thus a means to estimate the remaining life of deteriorating concrete bridge components. Through the coupling of the basic corrosion mechanism and the rate of deterioration with an economic decision model, the user can determine the lowest life-cycle cost of maintaining concrete bridges, the influence of delayed maintenance (preventive and corrective maintenance), and whether to protect, repair, rehabilitate, or replace a reinforced concrete bridge.

The corrosion mechanism, rate of deterioration, and economic models are presented to be used as applied knowledge with illustrative examples in Appendix A. The user is advised to develop a working knowledge of the material presented in this chapter in order to develop an in-depth understanding of the methods, procedures, and limitations presented in chapters 3, 4, and 5.

2.2 Corrosion of Reinforcing Steel in Concrete

It has been said that reinforced concrete is the ideal composite construction material. The concrete has a high compressive strength, is weak in tension, and is environmentally stable, whereas steel has a high tensile strength and spontaneously corrodes in the earth's moist, oxygen-rich environment but is environmentally stable in concrete. Concrete's high-alkali environment passivates the steel (reduces the spontaneous corrosion activity of steel to nil), and prevents the steel from spontaneously corroding in the oxygen-limited, moist environment of the concrete.

A few conditions carbonation of the concrete and the presence of the chloride ion destroy the protective (passive) layer formed on the steel surface in concrete. The chloride ion is present in seawater and in snow and ice melt water where sodium chloride (table salt) and calcium chloride have been used as deicer salts. When the chloride ion in the concrete pore water reaches a corrosion initiation level or threshold level at the concrete-steel contact surface, the steel begins to spontaneously corrode or rust. The chloride ion in solution reaches the steel either through cracks or by diffusion through the concrete's pore water. In either case, when the chloride ion reaches the threshold level of about 1.2 lb acid-soluble chloride ion per cubic yard of concrete (0.71 kg/m^3), the steel spontaneously rusts. The natural rusting of steel in chloride-ion-contaminated concrete takes place as follows (1).

$$Fe \rightarrow Fe^{++} + 2e^- \tag{1}$$

Iron changes to a positively charged iron ion and releases two negatively charged electrons at the corroding site.

$$Fe^{++} + 2Cl^- \rightarrow FeCl_2 \tag{2}$$

Iron ion complexes with the chloride ion at the corroding site.

$$FeCl_2 + H_2O + OH^- \rightarrow Fe(OH)_2 + H^+ + 2Cl^- \tag{3}$$

Iron chloride complex reacts with water and the hydroxyl ion in the water at the corroding site and forms iron hydroxide, leaving one hydrogen and two chloride ions in the pore water at the corroding site, even in the absence of oxygen. The chloride ion in the pore water is now free to complex with more iron and continue the spontaneous corrosion process.

$$2Fe(OH)_2 + \tfrac{1}{2}O^- \rightarrow Fe_2O_3 + 2H_2O \tag{4}$$

The iron hydroxide and oxygen ion in the concrete pore water at the corroding site react to form rust plus water. The rust is several times larger than the original iron. The increase in size causes internal pressure at the corroding site and the concrete cracks, resulting in spalls (potholes) and delaminations.

At the noncorroding site, the following reaction takes place (1).

$$\frac{1}{2}O_2 + H_2O + 2e^- \rightarrow 2OH^- \tag{5}$$

Oxygen in concrete pores reacts with a water molecule and the two negatively charged electrons released in equation 1, which have flowed through the steel to the noncorroding site. The hydroxyl ion then diffuses through the concrete pore water and reacts with the iron as shown in equation 3. Thus the electrical circuit (galvanic cell or battery) is complete, corrosion continues, and additional rust will form.

The reactions described by these equations are extremely important. They help us understand the factors that influence the rate at which corrosion takes place, the time to cracking of the concrete, and subsequent spalling and delamination.

The following factors influence the rate of corrosion:

1. *Chloride ion content at the reinforcing steel* - the higher the soluble chloride content the higher the rate of corrosion. As shown in equation 2, the greater the amount of chloride ion above the threshold level, the greater the amount of chloride ion that will be available to complex with the iron ion.

2. *The concrete pore water must be continuous to complete the ionic circuit* as shown in equations 3 and 5. Otherwise, the hydroxide ion produced at the noncorroding site (equation 5) cannot diffuse to the corroding site and take part in the corrosion reaction (equation 3). This would create an open electrical circuit and the corrosion rate would be reduced to zero. The reduction of the corrosion rate to zero or near zero has been observed many times in the laboratory when the concrete is allowed to dry out at a temperature of about 70°F (21°C) at a relative humidity of about 50% to 60% over a period of a few weeks. As the concrete dries out, the resistance of the concrete increases. As the electrical resistance of the concrete increases, the corrosion rate decreases (2).

3. If the concrete pore system is saturated, the oxygen available at the noncorroding site becomes limited because the oxygen must diffuse through the concrete pore water. *A reduced oxygen content at the noncorroding site reduces the corrosion reaction* because the noncorroding and corrosion reactions must take place at the same rate. The corrosion potential at the corroding site becomes more negative when the corrosion rate is reduced. Thus there is an optimum moisture content for the corrosion reaction. An example is concrete piles in seawater; the pile below the low- water line corrodes extremely slow and little or no concrete damage takes place over the life of the structure. At the splash zone, concrete damage is very rapid where moisture and oxygen are plentiful at the noncorroding site (equation 5) and

oxygen is present at the corroding site (equation 4).

Figure 2.1 summarizes the corrosion process. Chloride ions migrate directly to the reinforcing steel through water-filled cracks, such as subsidence cracks. Otherwise the chloride ion diffuses through the cement matrix through the water-filled pores. *Where surface cracking exists, such as drying shrinkage cracking, the diffusion path is shortened and corrosion begins sooner.*

As with any electrochemical reaction, like that in a car battery, *the higher the temperature, the faster the corrosion reaction.* Thus the corrosion rate of steel in concrete is faster during the warm, moist spring and fall days and slower during the cold winter and dry summer days. For warm seacoast areas such as the Florida Keys, corrosion takes place at a rapid, nearly continuous rate. If moisture is trapped in chloride-contaminated concrete by a coating or membrane, corrosion damage will accelerate because sufficient moisture will always be present to support the corrosion process (2). It is almost impossible to exclude oxygen from moist concrete.

To rehabilitate a reinforced concrete bridge component damaged by chloride-ion-induced corrosion, the cause or corrosion process must be addressed. Thus critically chloride-contaminated sound concrete must be removed, or new corrosion sites will be initiated under the overlay or encasement, resulting in a significantly reduced service life (2). The critically contaminated sound concrete can be identified by chloride content and copper/copper sulfate half-cell measurements. Specifics are presented in the rehabilitation sections in chapters 3, 4, and 5. In addition, all rust must be removed from exposed reinforcing steel. The rust contains chloride ions that will allow the corrosion process to continue in newly patched areas (2, 3) unless the reinforcing steel is treated with a corrosion inhibitor before the backfill concrete is placed (4, 5).

2.3 Deterioration Rates

2.3.1 Unprotected Concrete Elements

Unprotected concrete bridge components are bare concrete elements constructed with black steel. Examples are decks constructed with reinforcing steel without an epoxy or galvanized coating, periodic sealer applications, or a membrane or polymer concrete overlay; or piers without periodic sealer or coating applications on the surface of the concrete. However, the same concrete may be a low-permeability concrete, such as low-slump dense concrete (LSDC), or a microsilica concrete (MSC). Chloride ions still diffuse through these low-permeability concretes, though at a slower rate.

For unprotected concrete elements with a 2 in. (5 cm) mean concrete cover depth in a severe chloride environment, the corrosion process is presented in figure 2.2. As shown, the time to rehabilitation is the sum of several quantities: the corrosion initiation period plus the time

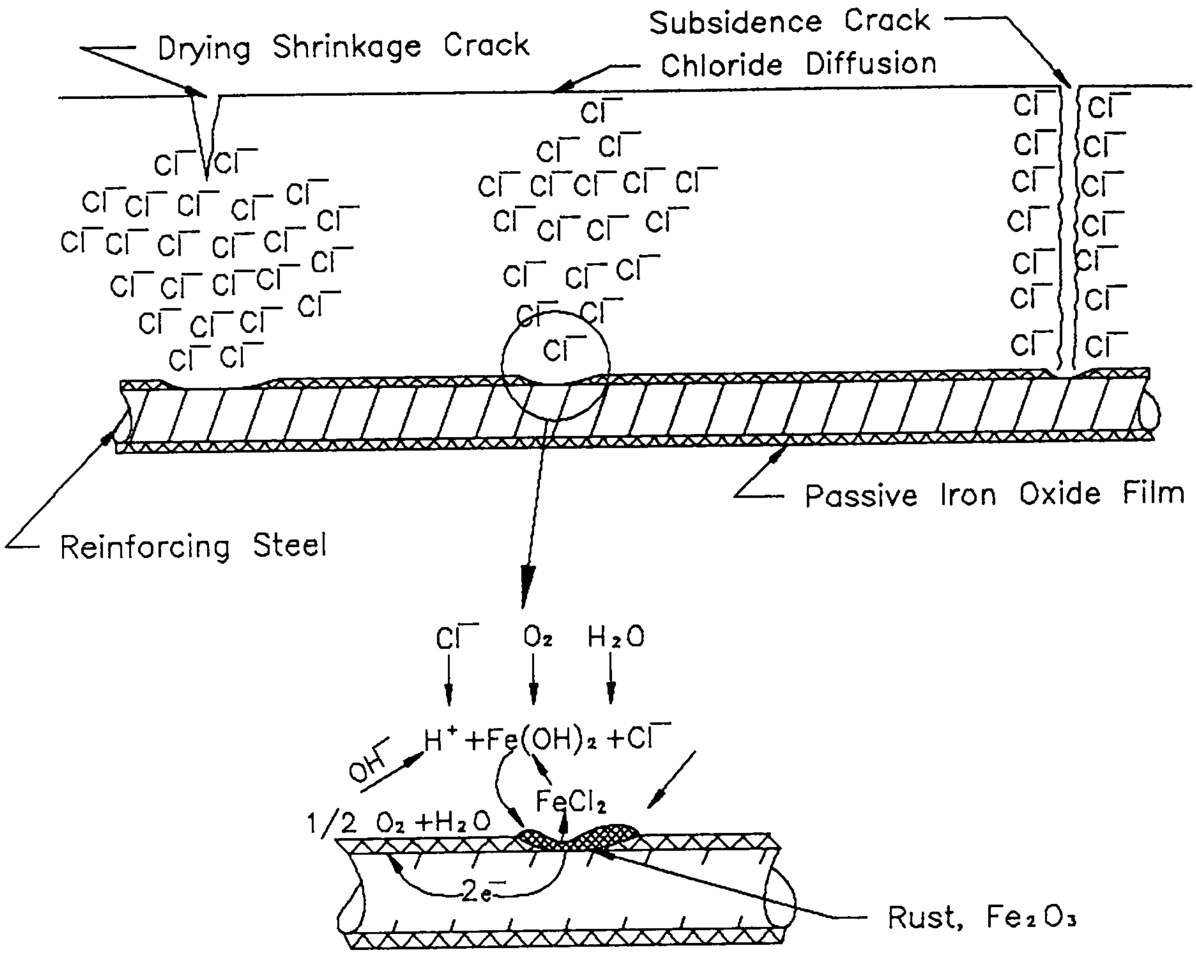

Figure 2.1 Chloride-Ion-Induced Reinforcing Steel Corrosion Process in Concrete

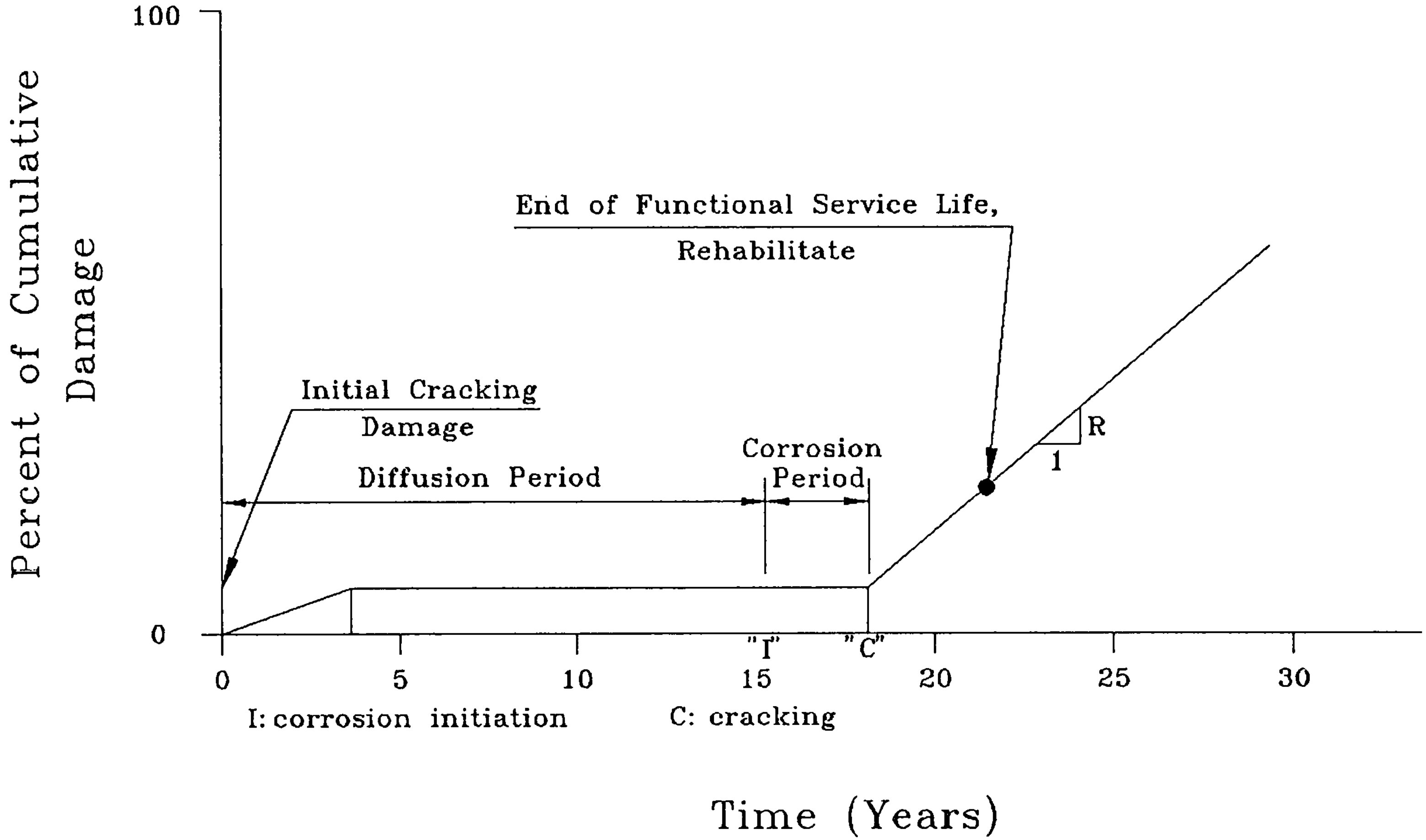

Figure 2.2 **Chloride Corrosion Deterioration Process for a Concrete Element with a Mean Cover Depth of 2 in. (5.08 cm)**

to cracking, plus the rate of deterioration (R) multiplied by the length of time until the end of functional service life is reached (6). Note that the chloride diffusion time is much longer than the corrosion period (time to cracking). The length of the diffusion period is a function of the mean concrete cover depth and the variation of the concrete cover depth around the mean cover depth. The cover depth variation is normally distributed (bell-shaped curve) (6), meaning that 50% of the steel cover is less than the mean cover depth, in this case 2 in. (5 cm). A measure of the magnitude of the spread of the normal distribution is the standard deviation. The larger the cover depth standard deviation, the more variable the distance between the reinforcing bars and the concrete surface. The standard deviation for reinforcing bar cover depths normally ranges from 0.2 to 0.4 in. (0.5 to 1.0 cm) (2).

For the case presented in figure 2.2, the chloride diffusion corrosion initiation point "I" represents the group of reinforcing steel bars at and below a specified cover depth. Because there is a range of concrete cover depths about the mean cover depth, the group of reinforcing steel normally chosen is the 2.5% group. That is, a cover depth is selected such that 2.5% of the reinforcing bars have a cover depth at or below the calculated cover depth. For the above case of a mean cover depth of 2 in. (5 cm) and a standard deviation of 0.3 in. (0.8 cm), the cover depth for 2.5% of the reinforcing steel bars is equal to 1.4 in. (3.6 cm), $2.0 - 1.96\ (0.3) = 1.4$. Note that 1.96 is the numerical factor for a normal distribution that represents the 2.5% fraction. Further details will be presented in section 2.4.1. Note also that cover depths are easily measured with a pachometer.

The cracking period, the time between corrosion initiation "I" and concrete cracking, "C", ranges between 2 and 5 years (2, 6). Concrete cover depth, reinforcing steel spacing and size, and concrete strength have little influence on the length of the cracking period (2). The rate of corrosion has a significant influence on the length of the cracking period (2).

The percent of cumulative damage presented in figure 2.2 is normally an ogive, or S-shaped curve (6). The ogive corrosion deterioration curve for concrete bridge components (figure 2.3) can be divided into four regions: the initial ill-defined rate, the initial near-steady-state rate, the final near-steady-state rate, and the reducing rate. The initial ill-defined deterioration rate is related to early concrete cracking, dry shrinkage, plastic shrinkage, and subsidence (transverse) cracking. Chloride ions penetrate the cracks and initiate the corrosion of the reinforcing steel. The magnitude of the initial ill-defined corrosion deterioration related to early concrete cracking is a function of the quality of the construction, concrete curing (drying and plastic shrinkage), and concrete cover depth (subsidence cracking). The initial and final near-steady-state rates are a function of the rate of chloride diffusion, the severity of the chloride exposure conditions, the distribution or variation of the concrete cover depth, and the rate of corrosion. For a given concrete bridge component, the initial and final near-steady-state deterioration rates are fairly predictable because (a) the rate of chloride diffusion depends on the concrete's water/cement ratio, degree of consolidation, and ambient temperature; (b) the amount of chloride exposure is a function of the mean average snowfall (2) or seawater chloride content; (c) construction methods result in a predictable (normal) distribution of the concrete cover depths; (d) corrosion rate is dependent on a given environment.

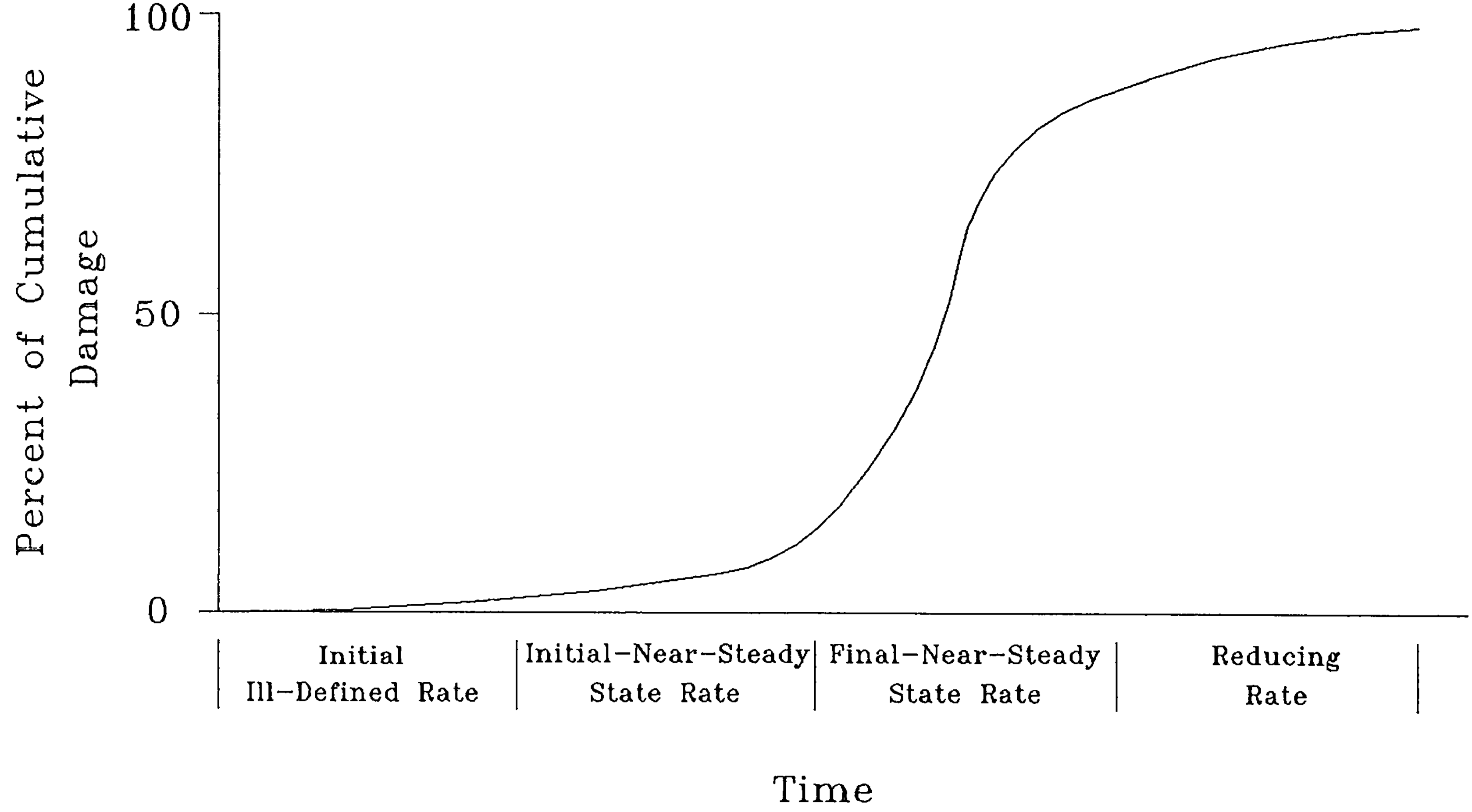

Figure 2.3 Cumulative Corrosion Deterioration of Reinforced Concrete Bridge Components Versus Time

The initial near-steady-state rate of deterioration occurs as the chloride content builds up to a near-constant contamination level (2). The chloride diffuses through the concrete and initiates corrosion and spalling occurs. As the component ages, more of the concrete becomes chloride-contaminated, involving more of the reinforcing steel, and the rate of deterioration increases (the final near-steady-state region). As the cumulative damage increases, the undamaged area decreases, and thus the rate of deterioration decreases (reducing rate region). The dividing line between the initial ill-defined rate and the initial near-steady-state rate is about 2.5% cumulative damage (2, 6). The initial near-steady-state rate (R) has been estimated at 2.1% per year from 2.5% to 40% cumulative damage (6). Also, a relationship based on engineering opinion and validated with historical performance data has been developed for the rate of deterioration from about the 3% damage level to the end of functional service life (2). Details are presented in sections 2.4.1 and 2.4.2.

Three methods are used for estimating the end of functional service life for concrete bridge components exposed to chloride environments. The first and second methods make use of the rate of chloride diffusion through concrete; the third method uses a relationship between the present corrosion damage and damage rate. All three methods require a definition for the end of functional service life. For bridge decks, the end of functional service life is based on riding quality or pavement roughness, which is in turn based on the level of damage in the worst traffic lane (2). The level of damage is the sum total percent of spalled and delaminated concrete and asphalt patches. The level of damage in the worst traffic lane that defines the end of functional service life ranges between 9% and 14% of the surface area (2).

For concrete substructure units, column and cap surfaces, the end of functional service life has not been defined because the decision to rehabilitate the substructure components is often dependent on the decision to rehabilitate the bridge deck (2). Percentage of substructure damage (spalls plus delaminations) at the time of deck rehabilitation has ranged from 4% to 60% of the surface area (2, 3). Here again, the decision to rehabilitate would be based on the damage level of the worst component and its structural integrity. The damage level of the worst substructure component ranged from 20% to 60% of the surface area (3).

2.3.2 Repaired Elements

Repaired concrete bridge elements are components that have been patched and overlaid or encased and in which the sound, chloride-contaminated concrete has been left in place. Thus, the rate of deterioration is dependent on the rate of corrosion of active corrosion sites and the initiation of new corrosion sites in critically chloride-contaminated areas. In this case, the corrosion process is in an advanced near-steady-state rate and should be predictable. The rate-of-damage relationship developed for unprotected elements was applied to repaired elements and validated with historical performance data (2). Details are presented in section 2.4.2.

2.3.3 Rehabilitated Elements

Rehabilitated reinforced concrete bridge elements are those elements where the spalled areas have been patched, the delaminated areas and all areas with a corrosion potential more negative than 250 mV to the copper/copper sulfate (CSE) half cell have been removed and patched, and the entire surface has been overlaid or encased with low-permeability concrete. That is to say, the cause of the corrosion process has been addressed. Thus, methods using corrosion inhibitors are also rehabilitation methods because they will address the cause of the problem.

Since the deteriorated and critically contaminated concrete has been removed, we start at year zero with sound, bare, uncontaminated concrete. Thus we may use the same method we used for unprotected bare concrete components to determine the length of the diffusion, cracking, and deterioration periods to estimate the service life of rehabilitated components (see figure 2.1). Details are presented in section 2.4.3.

2.4 Estimating Service Lives

2.4.1 Unprotected Concrete Elements

For unprotected concrete bridge elements, one of three methods can be used to estimate the service life of the deteriorating components. The first two methods estimate the service life or remaining service life using the concepts of chloride diffusion period, corrosion cracking period, and deterioration period. For these methods, the present chloride exposure age, the mean concrete cover depth and standard deviation, and the chloride diffusion constant must be known or estimated. The third method uses a deterioration rate relationship to determine the remaining life of a deteriorating component. The present corrosion damage level must be known to estimate the remaining life or to estimate the economic impact of delaying rehabilitation of the component. The following presents the service life estimation procedures for unprotected bare concrete components exposed to chloride environments.

2.4.1.1 Diffusion-Cracking-Deterioration Model

The procedure first requires a definition for the end of functional service life of concrete bridge components. For bare decks, the end of functional service life based on corrosion damage (spalls plus delaminations plus asphalt patches) that influence riding quality is 9% to 14% of the worst traffic lane (see section 2.3). The worst bridge traffic lane is typically the right lane. Left lanes, acceleration and deceleration lanes, and refuge lanes or shoulder lanes typically show fewer signs of damage than the right traffic lane. However, should the cover depth for any of these lanes be significantly less than for the right lane, the lane with the lowest cover depth will be the worst traffic lane. For concrete bridge substructure components (columns and pier caps), the end of functional service is dependent on structural safety considerations. Estimates of 20% to 60% damaged service area (spalls plus delaminations) of the worst component could be used as the end of functional service life for substructure components. However, it may be more economical to replace the columns or pier caps if the surface area damage exceeds 40% (7). Thus, a damage level of 20% to 40% of the worst component surface area will be used in this manual for substructure components that are to be rehabilitated.

Given a definition of the end of functional service life for concrete bridge components, the first step in estimating the service life is to determine the constructed parameters (concrete cover depth mean and standard deviation) needed to estimate the length of the diffusion period. The length of the diffusion period is related to the rate of chloride diffusion, the magnitude of the chloride contamination that is causing the chloride diffusion, and the mean and standard deviation of the concrete cover depth. The concrete cover depth must be measured. A minimum of 40 measurements must be taken for each bridge surface area unit of less than 8,000 ft^2 (740 m^2) (8). A bridge surface area unit comprises the riding surface of a deck span, column, and pier cap. For unit areas greater than 8,000 ft^2 (740 m^2), additional measurements are to be taken in proportion to the area greater than 8,000 ft^2 (740 m^2). For example, a 14,000 ft^2 area would require a minimum of 70 random measurements; (14,000/8,000)40 = 70). An area of 200 ft^2 (18 m^2) would require 40 random measurements whereas an area of 16,000 ft^2 (1,480 m^2) would require 80 random measurements. The mean (average) and standard deviation would be calculated for the bridge element being treated.

The rate of chloride diffusion is dependent on the chloride diffusion constant and the chloride contamination level that is causing the diffusion. Both the chloride diffusion constant and driving chloride contamination level are to be determined from chloride content measurements. A set of chloride contents for ¼-¾, ¾-1¼, 1¼-1¾, 1¾-2¼, 2¼-2¾, and 3¾-4¼ in. for an average depth of ½, 1, 1½, 2, 2½, and 4 in. (1.27, 2.54, 3.81, 5.08, 6.35, and 10.16 cm) is to be taken for each 600 ft^2 (55 m^2) of surface area to determine the chloride diffusion parameters for a bridge surface area unit. A minimum of three sets of chloride contents are to be taken for each bridge surface area unit. The 4 in. (10.16 cm) depth value is considered the background chloride level if it is on the portion of the curve parallel to the depth axis. Background chloride content is to be subtracted from each of the above four chloride contents. The ½ in. (1.27 cm) depth measurement is the driving chloride content, *Co*, causing the chloride diffusion (6). The chloride contents are to be acid-soluble chloride contents measured in accordance with the standard American Association of State Highway and Transportation Officials (AASHTO) method, AASHTO T260, or the method presented in reference 8. The chloride diffusion constant for each bridge surface area unit is to be calculated in accordance with the procedure presented in reference 2. The chloride diffusion constant calculation method determines the best chloride diffusion constant based upon the minimum cumulative sum of errors squared for sets of chloride content measurements for each bridge surface area unit using Fick's Second Law (2). The standard solution to Fick's Second Law used in the determination of the diffusion constant is as follows:

$$C_{(x,t)} = Co\left(1 - erf\frac{X}{2\sqrt{D_c t}}\right) \tag{6}$$

where $C_{(x,t)}$ = Chloride concentration at depth x after time t for an equilibrium chloride concentration Co at the surface (for this case the equilibrium chloride constant is at 0.5 in. [1.27 cm] below the concrete surface)

erf = Error function (from standard mathematical tables) (9)

D_c = Chloride diffusion constant

Table 2.1 presents a group of three sets of chloride content measurements from which the chloride diffusion constant is calculated for a bridge surface area unit. The driving chloride concentration level, *Co*, is the chloride content (Measured - Background = Difference) at the ½ in. (1.27 cm) depth for each sample, respectively.

Table 2.1 Example of Chloride Contents for Calculation of Bridge Component Specific Diffusion Constant (D_c) and Driving Chloride Diffusion Concentration (*Co*).

	Acid Soluble Chloride Content (lb/yd³)					
Sample Number:	D1		D2		D3	
Depth (in.)	Measured	Diff.	Measured	Diff.	Measured	Diff.
½	10.8	10.3	14.6	14.0	12.6	12.2
1	3.3	2.8	3.8	3.2	8.4	8.0
1½	1.8	1.3	2.0	1.4	3.9	3.5
2	1.3	0.8	1.4	0.8	1.7	1.3
2½	1.2	0.7	1.2	0.6	1.3	0.9
4	0.5	0.0	0.6	0.0	0.4	0.0

Notes: D_c is calculated for each sample location using the ½ in. Diff. depth *Co* with the associated Diff. chloride content at the specific depths. It is assumed that 4 in. is background chloride content. It would be best to measure the chloride content until the measurements as a function of depth are the same on the portion of the curve parallel to the depth axis. 1.0 lb/yd³ = 0.59 kg/m³.

In the absence of a diffusion constant D_c and a driving chloride concentration *Co* for a specific bridge component and for planning purposes, the average values presented in tables 2.2 and 2.3 may be used (2). Users should select a diffusion constant D_c that is closest to the state's climatic conditions (temperature and precipitation snow and rain levels) and similar bridge concrete specifications (water/cement ratio and consolidation specifications).

Table 2.2 State Concrete Bridge Chloride Diffusion Constants (D_c)

State	Mean D_c, in²/yr (cm²/yr)
Delaware	0.05 (0.32)
Minnesota	0.05 (0.32)
Iowa	0.05 (0.32)
West Virginia	0.07 (0.45)
Indiana	0.09 (0.58)
Wisconsin	0.11 (0.71)
Kansas	0.12 (0.77)
New York	0.13 (0.84)
California	0.25 (1.61)
Florida	0.33 (2.13)

Table 2.3 presents *Co* ranges for severity of climatic exposure conditions (seacoast and inland structures exposed to deicer salts) (2). Users should select the *Co* value closest to the chloride exposure conditions of their state, salt usage (tons/lane - mile). Table 2.4 presents state salt usage for the United States (2).

Table 2.3 Corrosion Environment: Chloride Content Categories (*Co*)

Low	Moderate	High	Severe
$0 < Co < 4$ $(0 < Co < 2.4)$	$4 \leq Co < 8$ $(4 \leq Co < 4.7)$	$8 \leq Co < 10$ $(4.7 \leq Co < 5.9)$	$10 \leq Co < 15$ $(5.9 \leq Co < 8.9)$
Mean = 3.0 (1.8)	Mean = 6.0 (3.5)	Mean = 9.0 (5.3)	Mean = 12.4 (7.4)
EXAMPLE STATES: Kansas California	Minnesota Florida	Delaware Iowa West Virginia Indiana	Wisconsin New York

NOTE: 1.0 lbs/cy (0.59 Kg/m³)

Table 2.4 State Salt Usage

> 5.0 tons/lane-mile/year (4.5 metric tons/lane-mile/year)	2.5-5.0 tons/lane-mile/year (2.3-4.5 metric tons/lane-mile/year)	< 2.5 tons/lane-mile/year (2.3 metric tons/lane- mile/year)
Maine	New Jersey	Delaware
New Hampshire	Pennsylvania	North Carolina
Vermont	West Virginia	South Carolina
Rhode Island	Virginia	Georgia
Massachusetts	Kentucky	Florida
Connecticut	Tennessee	Louisiana
New York	Iowa	Alabama
Maryland	Minnesota	Mississippi
Ohio	Missouri	Texas
Indiana	Nebraska	Arkansas
Illinois	Kansas	North Dakota
Michigan		South Dakota
Wisconsin		Oklahoma
		New Mexico
		Colorado
		Wyoming
		Montana
		Arizona
		Utah
		Idaho
		Nevada
		California
		Oregon
		Washington
		Alaska
		Hawaii

For structures exposed to seawater or brackish water, table 2.5 presents the exposure categories for reinforced concrete bridges. The sea-brackish water categories, low, moderate, high, and severe, were given in table 2.3. For example, a structure exposed to a moderate sea-brackish water environment could be estimated to have a *Co* value of 6.0 lb/yd^3 (3.5 kg/m^3) from table 2.3 if a more accurate value is not available.

Table 2.5 Seawater/Brackish Water Exposure Categories for Reinforced Concrete Bridge Components

Component Location Relative to Seawater/Brackish Water	Exposure Categories			
	Low	Moderate	High	Severe
In contact with soil and/or water with a chloride content (ppm)	< 500	> 500, < 1,000	>1,000, < 2,000	> 2,000
Less than 12 feet (3.7 m) above the mean high water elevation with a chloride content (ppm).	< 500	> 500, < 1,000	>1,000, < 2,000	> 2,000
Over water, regardless of height above the mean high water elevation, with a chloride content (ppm)	<1,500	>1,500, < 3,000	>3,000, < 6,000	> 6,000
Within 0.5 mile (0.8 km) of any major body of water with a chloride content (ppm)	<3,000	> 3,000, < 6,000	>6,000, < 12,000	>12,000

Note: Chloride content of water and soil are to be determined in accordance with established ASTM test methods.

Table 2.6 presents 40 typical cover depth measurements for a reinforced concrete bridge component and the mean and standard deviation for the measurements.

Table 2.6 Cover Depth Measurements for a Reinforced Concrete Bridge Component

Measurement Number	Cover Depth (in.)	Measurement Number	Cover Depth (in.)
1	2.1	21	2.3
2	2.0	22	2.1
3	2.3	23	1.7
4	2.5	24	1.8
5	2.0	25	1.9
6	2.0	26	2.2
7	1.9	27	2.3
8	1.8	28	2.6
9	2.3	29	1.5
10	1.7	30	1.6
11	2.0	31	1.8
12	2.1	32	2.0
13	1.9	33	2.2
14	2.0	34	2.3
15	1.6	35	1.4
16	1.8	36	1.3
17	2.0	37	1.2
18	2.4	38	1.9
19	2.5	39	2.0
20	2.1	40	2.2

1 in. = 2.54 cm
Mean, $\overline{X}$ = 1.98 in. (5.03 cm)
Standard Deviation, σ_{n-1} = 0.32 in. (0.82 cm)

Examples for Unprotected Concrete Elements: Diffusion-Cracking-Deterioration Model

The following examples illustrate the use of the diffusion-cracking-deterioration model. Previously presented data will be used in the examples.

Example 1: Bridge Deck in Kansas: D_c = 0.12 in²/yr (from table 2.2). Co = 3.0 lb/yd³ (mean from table 2.3). End of functional service life for deck based on worst lane = 12% (central value between 9% and 14%). Average measured concrete cover depth $\overline{d}$ = 1.98 in. (from table 2.6). Standard deviation of measured cover depth = 0.32 in. (from table 2.6).

Visually observable damage level at end of initial ill-defined rate = 2.5%. Initial near-steady-state = 2.1% per year (6).

Length of diffusion period for 2.5% of the steel is determined from Fick's diffusion law.

$C_{(x,t)}$ = Co[1 - erf($X/\{2\sqrt{D_c t}\}$)], equation 6

$C_{(x,t)}$ = chloride corrosion threshold level, 1.2 lb/yd^3 (acid-soluble chloride content minus background chloride).

X = for a given percentage of reinforcing steel, in this case 2.5%, depth of rebar exposed to a chloride concentration equal to 1.2 lb/yd^3. The general equation to calculate X is as follows:

$$X = \overline{X} - \alpha\sigma_{n-1} \tag{7}$$

where $\overline{X}$ = average reinforcing steel cover depth, for this case 1.98 in.

σ_{n-1} = standard deviation of the reinforcing steel cover depth, for this case 0.32 in.

X is determined for the chosen percentage of reinforcing steel exposed to an acid-soluble chloride content of 1.2 lb/yd^3.

For this case, the percentage of steel is 2.5% and α = 1.96. Table 2.7 presents α values for various percentages of rebar exposed to 1.2 lb chloride per cubic yard of concrete. Note: *Co* was taken at a depth of ½ in. below the surface. Thus X should be equal to d (the pachometer-measured cover depth) minus ½ in. However, corrosion does not start the moment that the chloride content *at the top* of the rebar reaches 1.2 lb/yd^3 because it takes some time for the chloride to break down the passive oxide layer. It is reasonable to say that corrosion starts when the chloride content at ½ in. *below* the top of the rebar is equal to 1.2 lb/yd^3. Therefore, the depth of ½ in. below the concrete surface and ½ in. below the top of the rebar cancel out and $\overline{X}$ is equal to $\overline{d}$, the average pachometer-measured cover depth.

For this example, where 2.5% of the steel is actively corroding,

$d = \overline{d} - \alpha\sigma_{n-1}$
$d = 1.98 - (1.96)(0.32)$
$d = 1.35$ in.

and

$$C_{(d,t)} = Co\left(1 - erf\frac{d}{2\sqrt{D_c t}}\right)$$

$$1.2 = 3.0\left(1 - erf\frac{1.35}{2\sqrt{0.12\ in^2/yr\ t}}\right)$$

$$0.40 = 1 - erf\frac{0.675}{\sqrt{0.12t}}$$

$$erf\frac{0.675}{\sqrt{0.12t}} = 0.60$$

Table 2.7 Alpha Values for Calculating Rebar Cover Depth for a Cumulative Percentage of Rebar Equal to and Less Than the Calculated Cover Depth (Based on a Normal Distribution)

Cumulative Percentage	α	Cumulative Percentage	α
2.5	1.96	14.5	1.06
3.0	1.88	15.0	1.04
4.0	1.75	16.0	0.99
5.0	1.65	17.0	0.96
6.0	1.56	18.0	0.92
7.0	1.48	19.0	0.88
8.0	1.41	20.0	0.85
9.0	1.34	24.0	0.71
9.5	1.31	28.0	0.58
10.0	1.28	32.0	0.47
10.5	1.25	36.0	0.36
11.0	1.23	38.0	0.31
11.5	1.20	40.0	0.26
12.0	1.17	45.0	0.13
12.5	1.15	50.0	0.0
13.0	1.13		
13.5	1.10		
14.0	1.08		

Note: Use linear interpolation for percentages not listed.

where the only unknown, time to initiate corrosion, can now be solved. If erf $y = 0.60$, then $y = 0.5961$ (see table 2.8).

then

$$\frac{0.675}{\sqrt{0.12t}} = 0.5961$$

$$\sqrt{0.12t} = 1.1324$$

$$0.12t = 1.2824t = 10.7 \text{ years}$$

The time from initiation of corrosion to spalling is about 3 years (6). Thus the time for spalling to occur on 2.5% of the worst traffic lane is 13.7 years (10.7 + 3).

The time for continuous deterioration to take place from 2.5% to the end of functional service life at 12% is (12 − 2.5)/(2.1%/yr) = 4.4 years.

Table 2.8 **Error Function Values y for the Argument of y**

y	erf y	y	erf y	y	erf y
0.02	0.02256	1.02	0.85084	2.02	0.99572
0.04	0.04511	1.04	0.85865	2.04	0.99609
0.06	0.06762	1.06	0.86614	2.06	0.99642
0.08	0.09008	1.08	0.87333	2.08	0.99673
0.10	0.11246	1.10	0.88021	2.10	0.99702
0.12	0.13476	1.12	0.88679	2.12	0.99728
0.14	0.15695	1.14	0.89308	2.14	0.99753
0.16	0.17901	1.16	0.89910	2.16	0.99775
0.18	0.20093	1.18	0.90484	2.18	0.99795
0.20	0.22270	1.20	0.91031	2.20	0.99814
0.22	0.24430	1.22	0.91553	2.22	0.99831
0.24	0.26570	1.24	0.92051	2.24	0.99846
0.26	0.28690	1.26	0.92524	2.26	0.99861
0.28	0.30788	1.28	0.92973	2.28	0.99874
0.30	0.32863	1.30	0.93401	2.30	0.99886
0.32	0.43913	1.32	0.93807	2.32	0.99897
0.34	0.36936	1.34	0.94191	2.34	0.99906
0.36	0.38933	1.36	0.94556	2.36	0.99915
0.38	0.40901	1.38	0.94902	2.38	0.99924
0.40	0.42839	1.40	0.95229	2.40	0.99931
0.42	0.44747	1.42	0.95538	2.42	0.99938
0.44	0.46623	1.44	0.95830	2.44	0.99944
0.46	0.48466	1.46	0.96105	2.46	0.99950
0.48	0.50275	1.48	0.96365	2.48	0.99955
0.50	0.52050	1.50	0.96611	2.50	0.99959
0.52	0.53790	1.52	0.96841	2.52	0.99963
0.54	0.55494	1.54	0.97059	2.54	0.99967
0.56	0.57162	1.56	0.97263	2.56	0.99971
0.58	0.58792	1.58	0.97455	2.58	0.99974
0.60	0.60386	1.60	0.97635	2.60	0.99976
0.62	0.61941	1.62	0.97804	2.62	0.99979
0.64	0.63459	1.64	0.97962	2.64	0.99981
0.66	0.64938	1.66	0.98110	2.66	0.99983
0.68	0.66378	1.68	0.98249	2.68	0.99985
0.70	0.67780	1.70	0.98379	2.70	0.99987

Table 2.8 Error Function Values y for the Argument of y

y	erf y	y	erf y	y	erf y
0.72	0.69143	1.72	0.98500	2.72	0.99988
0.74	0.70468	1.74	0.98613	2.74	0.99989
0.76	0.71754	1.76	0.98719	2.76	0.99991
0.78	0.73001	1.78	0.98817	2.78	0.99992
0.80	0.74210	1.80	0.98909	2.80	0.99992
0.82	0.75381	1.82	0.98994	2.82	0.99993
0.84	0.76514	1.84	0.99074	2.84	0.99994
0.86	0.77610	1.86	0.99147	2.86	0.99995
0.88	0.78669	1.88	0.99216	2.88	0.99995
0.90	0.79691	1.90	0.99279	2.90	0.99996
0.92	0.80677	1.92	0.99338	2.92	0.99996
0.94	0.81627	1.94	0.99392	2.94	0.99997
0.96	0.82542	1.96	0.99443	2.96	0.99997
0.98	0.83423	1.98	0.99489	2.98	0.99997
1.00	0.84270	2.00	0.99532	3.00	0.99998

Note: Use linear interpolation for values not listed.

Therefore, the total time to rehabilitation for our bridge deck in Kansas with a mean cover depth of about 2 in. is:

Initial time to corrosion of 2.5% of deck	Plus time to spalling of 2.5% of deck	Plus time to deterioration of additional 8.9% of deck	= Total time
10.7 years	+ 4.4 years	+ 4 years	= 18 years

Example 2: Bridge deck in West Virginia. D_c = 0.07 in²/yr (from table 2.2). Co = 9.0 lb/yd³ (from table 2.3). End of functional service life = 11.4%. Average measured concrete cover depth, $\overline{d}$ = 1.98 in. Measured cover depth standard deviation = 0.32 in. Visually observable damage level = 2.5%. α = 1.96 (from table 2.7).

$$d = \overline{d} - \alpha\sigma_{n-1}$$
$$d = 1.98 - (1.96)(0.32)$$
$$d = 1.35 \text{ in.}$$

$$C_{d,t} = Co\left(1 - erf\frac{d}{2\sqrt{0.07t}}\right)$$

$$1.2 = 9.0\left(1 - erf\frac{1.35}{2\sqrt{0.07t}}\right)$$

$$0.13 = 1 - erf\frac{0.675}{\sqrt{0.07t}}$$

$$erf\frac{0.675}{\sqrt{0.07t}} = 0.87$$

$$erf\,y = 0.87 \qquad y = 1.0707\ (table\ 2.8)$$

$$\frac{0.675}{\sqrt{0.07t}} = 1.0707$$

$$\sqrt{0.07t} = 0.6304$$

$$t = 5.7\ years$$

Time to spalling for 2.5% of rebar = 3 years
Time for continued deterioration, (12 - 2.5)/2.1% = 4.2 years
Time to rehabilitation = 5.7 + 3 + 4.2 = 13 years

Example 3: Concrete pile in seawater with a chloride content of 750 ppm, in Florida. D_c = 0.33 in^2/yr (from table 2.2). *Co* = 6.0 lb/yd^3 (from table 2.3). End of functional service life = 30% (midpoint of 20% to 40%) spalling of chloride exposure zone, which is above the low tide mark and below a line 12 ft above the mean high tide elevation.

Average measured cover depth for exposure zone = 2.8 in. Standard observable damage level = 2.5%. α = 1.96 (from table 2.7).

$d = \overline{d} - \alpha\sigma_{n-1}$
$d = 2.8 - (1.96)(0.21) = 2.4$ in.

$$1.2 = 6.0\left(1 - erf\frac{2.4}{2\sqrt{0.33t}}\right)$$

$$erf\frac{1.2}{\sqrt{0.33t}} = 0.80$$

$$erf\, y = 0.80 \qquad\qquad y = 0.9063$$

$$\frac{1.2}{\sqrt{0.33t}} = 0.9063$$

$$t = 5.3 \textit{ years}$$

Time to spalling for 2.5% of rebar = 3 years
Time for continued deterioration = (30 - 2.5)/2.1 = 13.1 years
Time to rehabilitation = 5.3 + 3 + 13.1 = 21 years

2.4.1.2 Diffusion-Spalling Model

This procedure also requires a definition of end of functional service life of concrete bridge components. For bare decks, 9% to 14% damage in the worst traffic lane and 20% to 40% damage on substructure components will be used as the definition of the end of functional service life. This procedure makes full use of the end of functional service life definitions through the realization that rehabilitation will take place only after spalling or delamination has occurred over the deeper rebars. Until that time, repairs will take place to maintain conditions (riding or structural). The deeper rebars are the rebars that represent the greater percentage of damage or the rebars with the greater cover depth. The following examples, 4, 5, and 6, are presented for illustrative purposes, and are the same as the previously presented examples 1, 2, and 3, respectively.

Example 4: Bridge deck in Kansas. $Co = 3.0$ lb/yd^3; $D_c = 0.12$ in^2/yr. End of functional service life = 12%. Average concrete cover depth d = 1.98 in. Cover depth standard deviation $\sigma_{n-1} = 0.32$ in.

$\alpha = 1.20$ for 11.5% damage (from table 2.7).
$d = \overline{d} - \alpha\sigma_{n-1} = 1.98 - (1.20)(0.32) = 1.60$ in.

$$C_{(d,t)} = Co\left(1 - erf\frac{d}{2\sqrt{D_c t}}\right)$$

$$1.2 = 3.0\left(1 - erf\frac{1.60}{2\sqrt{0.12t}}\right)$$

$$erf\frac{0.80}{\sqrt{0.12t}} = 0.60$$

$$erf\, y = 0.60 \qquad y = 0.5961$$

$$\frac{0.80}{\sqrt{0.12t}} = 0.5961$$

$$t = 15.0\ years$$

Time for spalling ranges between 2 and 5 years (10), the average would be 3.5 years. The time to rehabilitation = 15.0 + 3.5 = 18 years.

Example 5: Bridge deck in West Virginia. Co = 9.0 lb/yd^3; D_c = 0.07 in^2/yr. End of functional service life = 12%. Average concrete cover depth $\overline{d}$ = 1.98 in. Cover depth standard deviation σ_{n-1} = 0.32 in.

α = 1.20 for 12% damage (from table 2.7).
d = $\overline{d} - \alpha\sigma_{n-1}$ = 1.98 - (1.20)(0.32) = 1.60 in.

$$C_{(d,t)} = Co\left(1 - erf\frac{d}{2\sqrt{D_c t}}\right)$$

$$1.2 = 9.0\left(1 - erf\frac{1.60}{2\sqrt{0.07t}}\right)$$

$$erf\frac{0.80}{\sqrt{0.07t}} = 0.87$$

$$erf\,y = 0.87 \qquad y = 1.0707\ (\textit{table}\ 2.8)$$

$$\frac{0.80}{\sqrt{0.07t}} = 1.070$$

$$\sqrt{0.07t} = 0.7472$$

$$t = 10.0\ \textit{years}$$

Time to spalling = 3.5 years

Time to rehabilitation = 10.0 + 3.5 = 13 years

Example 6: Concrete pile in seawater with chloride content of 750 ppm in Florida. Co = 6.0 lb/yd^3; D_c = 0.33 in^2/yr. End of functional service life = 30%. Average concrete cover depth $\overline{d}$ = 2.8 in. Cover depth standard deviation σ_{n-1} = 0.21 in.

α = 0.52 for 30% damage (from table 2.7).
$d = \overline{d} - \alpha\sigma_{n-1}$ = 2.8 - (0.52)(0.21) = 2.69 in.

$$C_{(d,t)} = Co\left(1 - erf\frac{d}{2\sqrt{D_c t}}\right)$$

$$1.2 = 6.0\left(1 - erf\frac{2.69}{2\sqrt{0.33t}}\right)$$

$$erf\frac{1.34}{\sqrt{0.33t}} = 0.80$$

$$erf\, y = 0.80 \qquad y = 0.9063$$

$$\frac{1.34}{\sqrt{0.33t}} = 0.9063$$

$$t = 6.9\ years$$

Time to spalling = 3.5 years

Time to rehabilitation = 6.9 + 3.5 = 10 years

As shown in table 2.9, for bridge decks the diffusion-cracking-deterioration model (DCDM) agrees with the diffusion-spalling model (DSM). However, for piles the DCDM results do not agree with the DSM results. This is as expected, because the deterioration rate of 2.1% per year was developed from bridge deck data. Note that for the DCDM Florida pile example, 13 of 21 years of the estimated service life was due to the deterioration phase of the model; see example 3. Thus, the rate of 2.1% per year is too low, and one should not use the DCDM for estimating the life of substructure components. The DSM is based less on empirical results and more on theoretical processes than the DCDM and thus is applicable to substructure components.

Table 2.9 Estimated Time to Rehabilitation of Concrete Bridge Decks and Substructure Elements Using the Diffusion-Cracking-Deterioration Model (DCDM) and the Diffusion-Spalling Model (DSM)

State	Component	DCDM Years	DSM Years
Kansas	Deck	18	18
West Virginia	Deck	13	13
Florida	Pile	21	10

Example 7: Concrete bridge deck in New York. Two cases will be examined to demonstrate the utility of the DSM, one deck constructed with normal concrete and another constructed with the corrosion inhibitor DCI.

D_c = 0.13 in²/yr (from table 2.2).
Co = 12.4 lb/yd³ (from table 2.3).
End of functional service life = 12% damage.
Average cover depth: $\overline{d}$ = 2.3 in. (from pachometer measurements).
Cover depth standard deviation σ_{n-1} = 0.25 in. (from pachometer measurements).

α = 1.20 for 12% damage (from table 2.7).
$d = \overline{d} - \alpha\sigma_{n-1} = 2.3 - (1.20)(0.25) = 2.0$

Case I. Normal concrete; corrosion threshold level = 1.2 lb/yd³.

$$C_{(d,t)} = Co\left(1 - erf\frac{d}{2\sqrt{D_c t}}\right)$$

$$1.2 = 12.4\left(1 - erf\frac{2.0}{2\sqrt{0.13t}}\right)$$

$$erf\frac{1.0}{\sqrt{0.13t}} = 0.90$$

$$erf\ y = 0.90 \qquad y = 1.1631$$

$$\frac{1.0}{\sqrt{0.13t}} = 1.1631$$

$$\sqrt{0.13t} = 0.8598$$

$$t = 5.7\ years$$

Time to spalling = 3.5 years

Time to rehabilitation = 5.7 + 3.5 = 9 years

Case II. Normal Concrete with DCI Admixture

Note: If the chloride/nitrite ratio is greater than or equal to 2, then corrosion will initiate; that is, the chloride corrosion threshold level is increased in the presence of DCI (calcium nitrite) (5).

For this case, 2 gallons of 30% calcium nitrite solution (DCI) is added to a cubic yard of concrete. The ratio of nitrite to calcium nitrite is 0.692. Therefore, the amount of nitrite in a cubic yard of concrete is equal to (2 gal/yd^3)(10.7 lb/gal)(30%)(0.692) = 4.44 lb/yd^3.

Thus the chloride corrosion threshold level is equal to:

chloride/nitrite =2
chloride/4.44 lb/yd^3 = 2
chloride = 8.88 lb/yd^3

$$C_{(d,t)} = Co\left(1 - erf\frac{d}{2\sqrt{D_c t}}\right)$$

$$8.88 = 12.4\left(1 - erf\frac{2.0}{2\sqrt{0.13t}}\right)$$

$$erf\frac{1.0}{\sqrt{0.13t}} = 0.28$$

$$erf\, y = 0.28 \qquad y = 0.2535$$

$$\frac{1.0}{\sqrt{0.13t}} = 0.2535$$

$$\sqrt{0.13t} = 3.9448$$

$$t = 120\ years$$

Time to spalling = 3.5 years

Time to rehabilitation = 120 + 3.5 = 123 years

Example 8: Remaining Life and Effects of Delaying Rehabilitation

The DSM can also be used to determine the remaining life, or time to the point when an existing concrete bridge component must be rehabilitated. If for example 7, case I (a bridge deck in New York), the bridge deck is currently 6 years old, then the remaining life would be 3 years (total time to rehabilitation, 9 years minus the present age of 6 years).

To estimate the effects of delaying the time to rehabilitation, levels of damage greater than the rehabilitation level of 12% and the number of years it would take to reach that level of damage would be calculated using the DSM model. For this example we will use the conditions presented in example 7, case I where 12% damage would occur in 9 years. Now the time for 15%, 20%, 30% and 40% damage to occur would be

α = 1.04 for 15% damage, d = 2.04
α = 0.85 for 20% damage, d = 2.09
α = 0.52 for 30% damage, d = 2.17
α = 0.26 for 40% damage, d = 2.24

$$C_{(d,t)} = Co\left(1 - erf\frac{d}{2\sqrt{D_c t}}\right)$$

$$1.2 = 12.4\left(1 - erf\frac{2.0}{2\sqrt{0.13t}}\right) \textit{ for } 15\%$$

$$erf\frac{1.02}{\sqrt{0.13t}} = 0.90$$

$$erf\, y = 0.903 \qquad y = 1.1631$$

$$\frac{1.02}{\sqrt{0.13t}} = 1.1631$$

$$t = 5.9 \textit{ years}$$

Time to 15% damage = 5.9 + 3.5 = 9.4 years

$$1.2 = 12.4\left(1 - erf\frac{2.09}{2\sqrt{0.13t}}\right) \; for \; 20\%$$

$$erf\frac{1.05}{\sqrt{0.13t}} = 0.90$$

$$\frac{1.05}{\sqrt{0.13t}} = 1.1631$$

$$t = 6.3 \; years$$

Time to 20% damage = 6.3 + 3.5 = 9.8 years

$$1.2 = 12.4\left(1 - erf\frac{2.17}{2\sqrt{0.13t}}\right) \; for \; 30\%$$

$$\frac{1.09}{\sqrt{0.13t}} = 1.1631$$

$$t = 6.8 \; years$$

Time to 30% damage = 6.8 + 3.5 = 10.3 years

$$1.2 = 12.4\left(1 - erf\frac{2.24}{2\sqrt{0.13t}}\right) \; for \; 40\%$$

$$\frac{1.12}{\sqrt{0.13t}} = 1.1631$$

$$t = 7.1 \; years$$

Time to 40% damage = 7.1 + 3.5 = 10.6 years

2.4.2 Repaired Elements

A repaired element is one in which the spalled and delaminated areas have been patched but sound chloride-contaminated concrete surrounds the new patch concrete. In this case, the surrounding concrete deteriorates in 1 to 5 years. The service life of the patch concrete can be determined by the DSM presented in section 2.4.1.

An element is also considered to be a repaired element if the spalled and delaminated areas have been patched and overlaid (decks) or encased (abutments, piers, pier caps, piles) and the sound chloride-contaminated concrete is left in place. For these cases, neither the DCDM nor the DSM presented in section 2.4.1 is applicable because the left-in-place concrete is chloride-contaminated. The left-in-place sound chloride- contaminated concrete is the cause of a limited service life. The steel still continues to corrode under the overlay (LMC, LSDC, MSC, and bituminous concrete with a membrane) but at a reduced rate. If the membrane is not watertight, the corrosion rate will accelerate.

Because the corrosion process continues under the overlay material, it is reasonably predictable. A relationship has been developed for bridge decks that are actively corroding (2). The model can be used to predict the remaining life of an unprotected or overlaid deck (2). It is based on the percent of worst-lane damage (spalled + delaminated + asphalt patches) and can be used within the limits of 3% to 30% present damage. However, the upper limits of 20% to 30% are the least reliable. Best results are achieved between 3% and 20% damage. The remaining service life prediction model for the worst traffic lane of repaired decks is as follows (2):

$$y = 11.2 - 5.34x + 3.41x^{1.1}$$

where

y = Time to rehabilitate
x = Total percent of spalled, delaminated, asphalt patched area in the worst traffic lane

Example 9: A bridge deck was overlaid 10 years ago with LMC. The delaminated areas have been identified using the chain drag method, and spalled and asphalt areas have been measured. The following presents the deck condition survey.

Damage Type	Traffic Lane Damage (ft^2)		
	Deceleration	Right	Left
Asphalt patch	0	0	0
Spalls	0	10	0
Delamination	8	76	24
Total damage	8	86	24

The deceleration lane is 960 ft^2, the right and left lanes are each 1,440 ft^2. The percent total damage is 0.8% [(8/960)100], 6.0% [(86/1,440)100], and 1.7% [(24/1,440)100], for the deceleration lane, right lane, and left lane, respectively. Thus, the worst traffic lane is the right lane, with 6.0% damage.

The end of functional service life is 9% to 14% damage, with a midpoint of 12% damage. Thus, the remaining service life is equal to:

$$y = 11.2 - 5.34x + 3.41x^{1.1}$$

$y = 11.2 - 5.31(12 - 6.0) + 3.41(12 - 6.0)^{1.1}$
$y = 11.2 - 28.7 + 21.8$
$y = 4.3$ years

Users of the above model must remember that the present damage must be greater than 3.0%. If the present damage in the worst lane is greater than 12%, the remaining life y will be negative. The negative value of y indicates that the deck should have been rehabilitated y years ago.

2.4.3 Rehabilitated Elements

A rehabilitated element is one in which the damaged and chloride-contaminated concrete (all concrete with a corrosion potential more negative than 250 mV CSE) is removed. For these elements the DCM may be used to estimate the service life. If a low-permeability concrete such as LMC, LSDC, or MSC is used, the diffusion constant for these materials must be used in the diffusion equation. If a low-permeability concrete is used to overlay or encase the existing concrete, the diffusion constant will be a composite of the two materials based on their thickness relative to the total concrete cover thickness. Table 2.10 presents diffusion constants for LMC, LSDC, and MSC that can be used if actual values are not available (2). However, because these values are based on limited observations (2), it is recommended that the diffusion constant for these materials be determined from field surveys.

Table 2.10 Diffusion Constants for LMC, LSDC, and MSC

Diffusion Constant	LMC	LSDC	MSC
D_c (in²/yr)	0.045	0.020	0.020
D_c (cm²/yr)	0.290	0.129	0.129

The following examples are presented to illustrate how the service life and remaining service life of rehabilitated concrete bridge elements may be determined.

Example 10: Bridge deck in Minnesota. The rehabilitated concrete deck received an LSDC overlay 10 years ago. The average cover depth and standard deviation of the concrete left in place (after milling) are 0.8 in. and 0.20 in., respectively. The average overlaid cover depth is 3.0 in. and the standard deviation is 0.38 in. A 2 in. LSDC overlay was placed on the deck.

End of functional service life of the overlaid decks is 12% total damage (asphalt patches, spalls, delaminations), thus $\alpha = 1.20$ (table 2.7).

Then $d = \overline{d} - \alpha\sigma_{n-1} = 3.0 - (1.20)(0.38) = 2.54$ in.

Composite Diffusion Constant

Existing concrete = 0.05 in^2/yr (table 2.2)
LSDC = 0.02 in^2/yr (table 2.10)

$$\text{Composite } D_c = \left(\frac{0.8}{3.0}\cdot 0.05\right) + \left(\frac{3.0 - 0.8}{3.0}\cdot 0.02\right)$$

$$D_c = 0.02 + 0.012 = 0.028$$

$$Co = 6.0\ lb/yd^3\ (table\ 2.3)$$

$$C_{(d,t)} = Co\left(1 - erf\frac{d}{2\sqrt{D_c t}}\right)$$

$$erf\left(\frac{1.27}{\sqrt{0.028t}}\right) = 0.80$$

$$erf\ y = 0.80 \qquad y = 0.9063\ (table\ 2.8)$$

$$\frac{1.27}{\sqrt{0.028t}} = 0.9063$$

$$t = 70\ years$$

Estimated service life = Corrosion Initiation + Spalling = 70 + 3.5 = 73 years
Remaining Life = Estimated Life - Present Age = 73 - 10 = 63 years

Example 11: Bridge deck in West Virginia. the rehabilitated concrete deck received a LMC overlay 10 years ago. Average cover depth of left-in-place concrete = 0.8 in. Overlaid cover depth, average = 2.5 in. Standard deviation = 0.38 in. End of functional service life = 12% damage, α = 1.20 (table 2.7).

$d = \overline{d} - \alpha\sigma_{n-1} = 2.5 - (1.20)(0.38) = 2.04$.
$Co = 9.0$ lb/yd^3 (table 2.3).
Existing concrete, $D_c = 0.07$ in^2/yr (table 2.2).
LMC, $D_c = 0.045$ (table 2.10)

$$\text{Composite } D_c = \left[\left(\frac{0.8}{2.5}\right)(0.07)\right] + \left(\frac{2.5 - 0.8}{2.5}\right)(0.045) = 0.053 \ in^2/yr$$

$$C_{(d,t)} = Co\left(1 - erf\frac{d}{2\sqrt{D_c t}}\right)$$

$$1.2 = 9.0\left(1 - erf\frac{2.04}{2\sqrt{0.053t}}\right)$$

$$erf\frac{1.02}{\sqrt{0.053t}} = 0.87$$

$$erf\, y = 0.87 \qquad y = 1.0669 \ (table\ 2.8)$$

$$\frac{1.02}{\sqrt{0.053t}} = 1.0669$$

$$t = 17.2 \ years$$

Estimated service life = Corrosion Initiation + Spalling = 17.2 + 3.5 = 21 years
Remaining service life = Estimated - Present = 21 - 10 = 11 years

2.5 Methodology for the Selection of Cost-Effective Methods

This section presents a standardized methodology for cost-effectiveness comparisons in order to generate least-cost solutions to bridge work. The least-cost solution must be based on the costs incurred over the service life of the bridge, taking into account the time value of money. This is the true meaning of cost-effectiveness. Decisions based on a comparison of the initial cost of protecting, repairing, or rehabilitating the structure versus replacement costs or the initial costs of individual protection, repair, or rehabilitation items will not result in a cost-effective solution.

The cost-effectiveness methodology is based on comparisons of alternatives using accepted engineering economic analysis procedures. Generalized models and illustrative examples are presented in Appendix A to demonstrate how the conversion of cash flows to equivalent values can be compared to provide decisions about cost-effectiveness (11). The available alternatives are

1. force account protection, repair, or rehabilitation of the existing structure followed by eventual replacement;

2. contracted protection, repair, or rehabilitation of the existing structure followed by eventual replacement;

3. immediate replacement of the structure.

Since the first two alternatives, both include eventual replacement, the third replacement alternative becomes an input for the protection, repair, and rehabilitation alternatives and therefore may be evaluated first. The protection, repair, and rehabilitation alternatives are to be evaluated first using the rehabilitation model, and the one that displays the lower cost will be compared with the replacement alternative in the calculation of a parameter called the "value management" (VM) term, where

VM = Equivalent structure replacement cost minus equivalent structure, protection, repair, or rehabilitation cost

A *positive* value of VM indicates that the structure should be *protected*, *repaired*, or *rehabilitated*; a *negative* value indicates that it should be *replaced*. The magnitude of VM indicates the amount of savings involved if the decision is made as indicated by the sign.

In the models presented in Appendix A, the equivalent values are determined as "equivalent uniform annual costs" (EUAC) for perpetual service. The choice of perpetual service is based on the fact that bridge sites are normally used for long periods (50 years or more), and the differences between equivalent values for 50 years or more and infinity (perpetual service) are very small in comparison to the uncertainties in predicting future cash flows.

Notice that the equivalent replacement and protection, repair, or rehabilitation costs and the VM can be expressed in terms of "capitalized cost" (present worth of perpetual service) simply by multiplying by the reciprocal of the interest rate, which is the value for the uniform series present worth factor for infinite time.

2.6 References

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3

Standard Methods

3.1 Introduction

This chapter presents standard protection, repair, and rehabilitation methods currently used by state highway agencies (SHAs). Methods applied to concrete bridge elements (decks, substructures, or superstructures) are described with respect to limitations, estimated service life, estimated price or cost, and construction procedures and specifications.

Estimated service life is presented as a range for a given set of environmental exposure conditions. The service life range represents about 70% of the protected, repaired, and rehabilitated cases. Thus 15% of the treated elements will have a shorter service life, and 15% will have a longer service life. The midpoint of the range represents the average service life and can be used for planning purposes. Although individual bridge elements are equally likely to fall below or above the average service life, the least-cost solutions can be selected because equal numbers of treated elements will have higher and lower life-cycle costs. For individual bridge elements, use the methodologies presented in chapter 2 for estimating service lives.

In the calculations below for the estimated price and cost, benchmark year and location are mid-1991 and the national average, respectively. Prices and costs can be adjusted for year and location (1). If comparisons between methods are made to select the lowest life-cycle cost or price, and if all costs or prices are adjusted by the same time factor, only the cost or price relative to a comparison year and location will be different. The life-cycle cost or price solution will not change. However, to estimate individual project costs, adjustments must be made for time and location.

The protection methods presented in section 3.2 include concrete sealers and coatings; the repair methods (section 3.3) and rehabilitation methods (section 3.4) include patching, overlays, and encasement.

3.2 Protection Methods

Protection methods are used to prevent chloride ions from diffusing into concrete. Chloride diffuses through concrete by being ionized in water; hence, methods that prevent water from entering the concrete will also exclude the chloride ions.

It is desirable that the protection material be breathable, allowing water vapor to pass through but not liquid water. A breathable material will reduce the degree of saturation of the concrete and thus reduce the effects of concrete-deteriorating mechanisms that require water, such as freezing and thawing and the alkali-silica reaction. A breathable material will also reduce the stripping action of water by allowing the concrete to dry out somewhat, and it will adhere to the concrete longer.

Protection methods presented in this section are penetrating sealers for decks (sections 3.2.1 through 3.2.4) and sealers and coatings for substructure and superstructure elements (sections 3.2.5 through 3.2.8).

3.2.1 Deck Sealers

- Description

A sealer is a solvent- or water-based liquid applied to a deck surface. Only penetrating sealers, silanes and siloxanes (or combinations), are recommended for deck surfaces. Other sealer types have an inadequate depth of penetration and quickly wear when exposed to traffic abrasion.

Silanes and siloxanes are normally applied to a prepared concrete surface at a rate of about 150 ft^2/gal (3.7 m^2/L). When silanes and siloxanes penetrate the prepared concrete surface, they react with the pore walls of the hardened, moist cement paste to create a nonwettable surface. This seal prevents liquid water from entering the concrete but allows water vapor to enter and leave the concrete.

- Limitations

The effectiveness of penetrating sealers in excluding chloride is related to the permeability of the original unsealed concrete. The permeability of hardened concrete depends on the degree of consolidation of the fresh concrete, the curing conditions, and the water/cement ratio. The chloride exclusion efficiency of a sealer would be greater for a consolidated, well-cured concrete with a high water/cement ratio than one with a low water/cement ratio because an unsealed low-permeability concrete has a lower rate of chloride ingress than a high-permeability concrete. However, the application of a sealer is not a license to use a concrete with a higher water/cement ratio concrete nor to discontinue or take liberties with sound construction consolidation and curing practices.

Sealer treatments are not to be incorporated as part of the original construction. Sealer treatments are to be left as separate maintenance contracts and limited to warm-weather months.

Sealers are *not* to be applied to concrete elements with active corrosion sites or to critically chloride-contaminated concrete. Active corrosion sites are defined as locations with a corrosion potential more negative than 250 mV CSE. Concrete is considered to be critically contaminated when the chloride contamination level for 1% of reinforcing steel is greater than 1.0 lb/yd^3 (0.59 kg/m^3) or will not remain below this concentration even if further ingress of chlorides is prevented through the application of a sealer.

The following examples illustrate how to determine whether a deck is critically contaminated.

Example 1. Average cover depth $\overline{d}$ = 2.0 in. Cover depth standard deviation σ_{n-1} = 0.20 in. For 1% of the rebar, α = 2.33 (from normal distribution table 2).

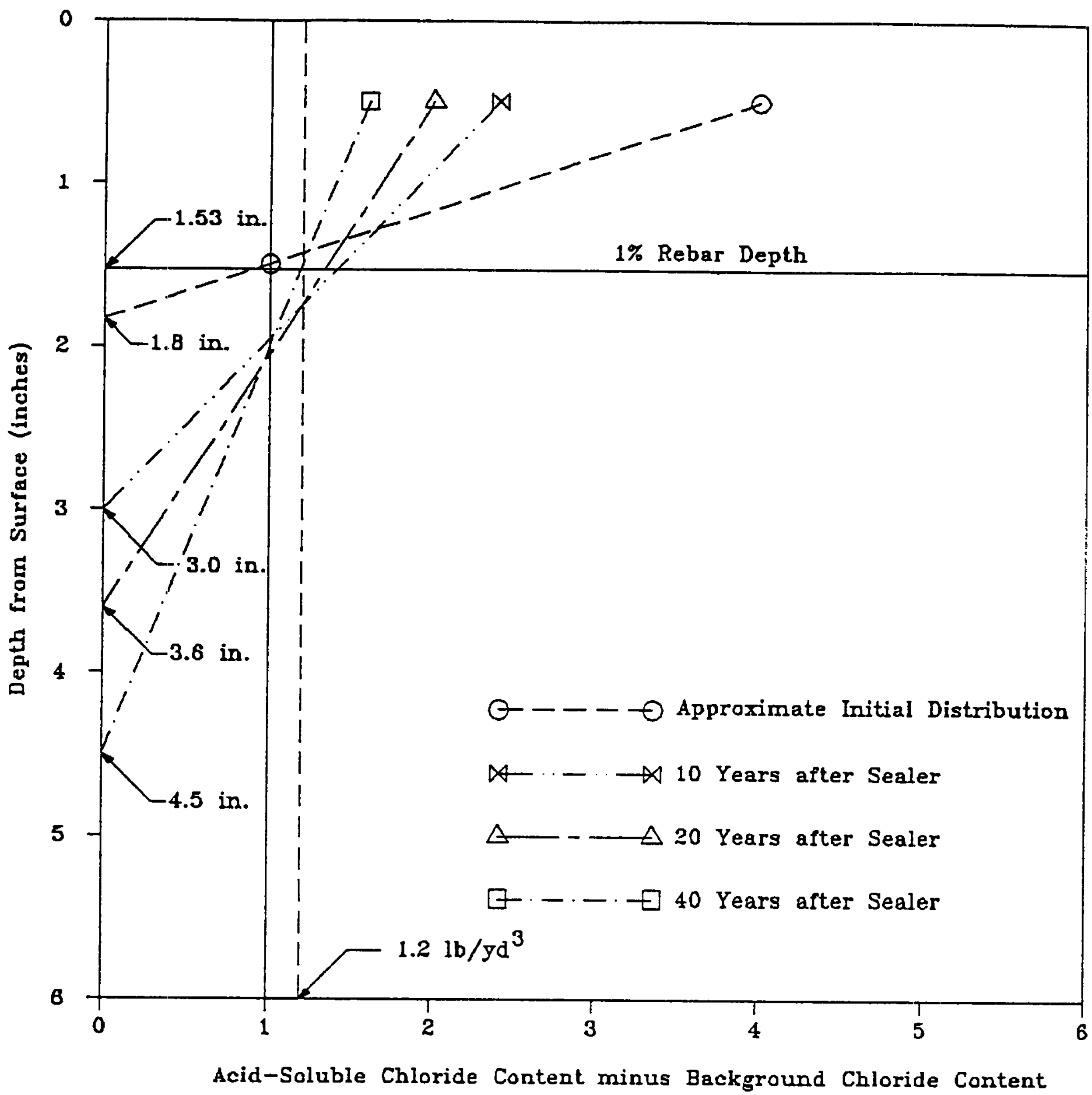
Cover depth of 1% of rebar, $d_I = \overline{d} - \alpha\sigma_{n-1} = 2.0 - (2.33)(0.20) = 1.53$ in.

Present average chloride contamination at depth d = 0.5 in is 4.0 lb/yd^3; at depth d = 1.5 in. = 1.0 lb/yd^3 (determined from chloride contamination survey of deck; see section 2.3.1 for recommended chloride and cover depth survey plans). For the sealer case, the chloride distribution can be approximated by a straight line (figure 3.1) (3).

The approximate chloride distribution is presented in figure 3.1 for a bridge deck after 40 years of a protection program of periodic sealer applications where further chloride ingress is prevented. The chloride distribution after 40 years of sealer applications was determined as follows (3).

Chloride content at the 0.5 in. depth is equal to 40% of the initial 4.0 lb/yd^3 or 1.6 lb/yd^3 [(0.40)(4.0)]. The depth of zero chloride content is equal to initial zero depth of 1.80 in. (see figure 3.1) divided by 0.4, or 4.5 in. (1.80/0.40). These two points, 1.6 lb/yd^3 @ 0.5 in. and 0.0 lb/yd^3 @ 4.5 in. form the straight line presented in figure 3.1. As shown in figure 3.1, at the depth of 1% of the steel (1.53 in.), the chloride content would exceed 1.0 lb/yd^3 and thus periodic sealer applications will *not* provide 40 years of protection. In 40 years, this deck will experience about 1% damage. Similar analysis can be made for the chloride distribution for 10 and 20 years after periodic sealer applications (3).

For 20 years, the chloride content at the 0.5 in. depth is 50% of the initial content and the zero chloride concentration is at a depth of the initial zero chloride concentration depth divided by 0.5, 2.0 lb/yd^3 [(4.0)(0.50)] and @ 3.6 in. [(1.8)/(0.5)], respectively. The straight line formed by these two points, @ 2.0 lb/yd^3 0.5 in. and 0.0 lb/yd^3 3.6 in., is shown in figure 3.1. As shown in figure 3.1, periodic sealer applications would not provide 20 years of protection. This would also be true of 10 years; the two points that would form the 10-year straight line are @ 2.4 lb/yd^3 [(4.0)(0.60)] 0.5 in. and @ 0.0 lbs/yd^3 [(1.8)/(0.6)] 3.0 in. (3).



(1 in. = 2.54 cm; 1 lb/yd^3 = 0.59 kg/m^3)

Figure 3.1 **Approximate Change in Chloride Distribution after Periodic Applications of a Sealer, Example 1**

Thus, for this example, sealer use is not recommended.

Example 2. Average cover depth $\overline{d}$ = 2.6 in. Cover depth standard deviation σ_{n-1} = 0.22 in. Thus the cover depth of 1% of the reinforcing steel d_l = 2.6 - (2.33)(0.22) = 2.09 in.

Chloride contamination @ 4.0 lb/yd^3 is 0.5 in. and 0.6 lb/yd^3 @ 1.5 in. As shown in figure 3.2, the zero chloride content level is at 1.7 in.

Forty years after periodic sealer applications, the chloride content at 0.5 in. is calculated as (4.0)(0.40) = 1.6 lb/yd^3. The depth of zero chloride content is calculated as (1.7)/(0.40) = 4.25 in. (see figure 3.2 for 40 year straight line). As shown, at 40 years the chloride at the 1% rebar cover depth is slightly less (0.95 lb/yd^3) than the 1.0 lb/yd^3 criterion. Thus periodic sealer applications will provide 40 years of chloride protection. At 20 years, the chloride content is 0.85 lb/yd^3 at the 1% rebar cover depth level (see figure 3.2).

3.2.1.1 Estimated Service Life

Field environmental exposure conditions that may influence the service life of penetrating sealers applied to decks may include degradation caused by ultraviolet light, moisture, and surface wear (traffic conditions). Considering typical rates of traffic wear and depths of penetration, wear has little to no influence on the length of service life of sealers. Thus the service life of a penetrating sealer is controlled by environmental degradation mechanisms that manifest themselves as increased water permeability of the sealed surface.

The service lives of penetrating sealers (silanes and siloxanes) applied to decks range from 5 to 7 years, and they should be reapplied every 6 years (3).

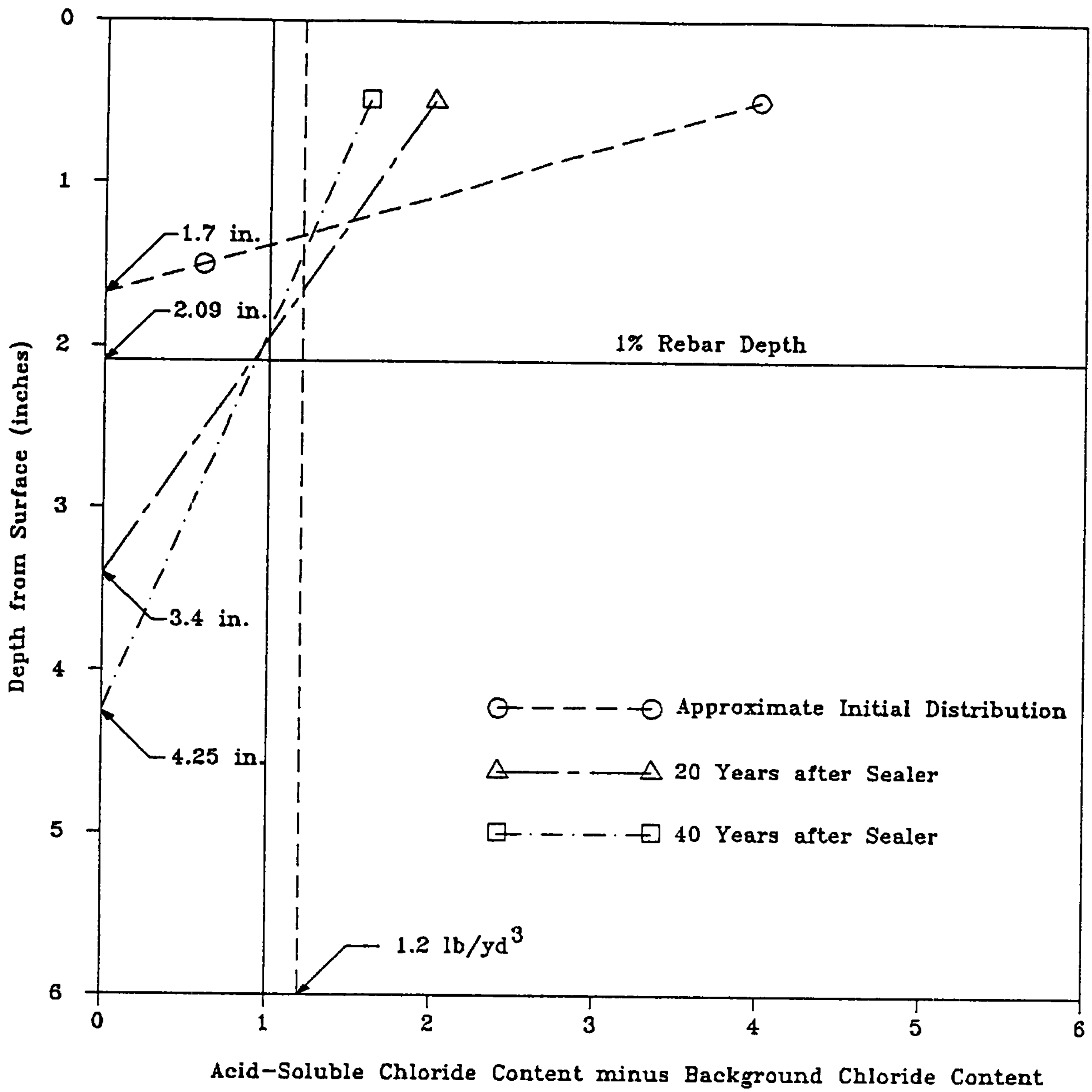
3.2.1.2 Estimated Price

Price was determined from an analysis of bids for silane and siloxane sealers and includes surface preparation and all labor, equipment, and materials. The price is influenced by quantity and is \$8 to \$10/yd^2 (\$9.50 to \$12/m^2). The following relationship can be used to adjust the price for quantity:

$$y = 8.652 + (7.04 \times 10^{-5})X + [(56.077)/(X^{1.2394})]$$

where y = predicted national adjusted price, (\$/yd^2)

X = job quantity, (yd^2)



(1 in. = 2.54 cm; 1 lb/yd^3= 0.59 kg/m^3)

Figure 3.2 Approximate Change in Chloride Distribution after Periodic Applications of a Sealer, Example 2

3.2.2 Construction Procedure: Deck Sealers

- Surface Preparation

A sealer will not penetrate properly unless it is applied to a clean, dry surface. Not only should the surface be dry for maximum penetration, but the subsurface voids should be dry as well. Before application, any surface laitances, residual curing compounds, asphalt, grease, and oil should be removed from the concrete surface. The following cleaning methods may be employed:

> *Sandblasting*. The oilers used for the operation of compressed-air tools should be removed, and oil and water traps should be installed on the compressor before use. Failure to do so may lead to surface contamination.
>
> *Shotblasting*. Since both the steel shot (metallic abrasive) and the dust resulting from abrasion are collected by the unit, it is environmentally contained. Care must be taken in choosing the speed of the machine and the size and application rate of shot that will provide the desired depth of removal (4).

Following cleaning, any dust or other loose matter should be removed with compressed air or vacuum. The same provisions outlined under sandblasting apply to the compressed air source.

Epoxy injection of visible cracks should be considered before application of the sealer. An alternative is to first seal the cracks and then reseal the entire surface, including the cracks (5). If the observed cracking is greater than 6 lineal ft (1.8 m) per 100 ft^2 (9.3 m^2), alternative forms of protection should be considered. The decision whether to use sealers on decks with more than 6 ft/100 ft^2 (1.8 m/9.3 m^2) of cracking should be based on an economic analysis comparing the cost of sealing cracks and the deck with alternative forms of protection, such as a polymer concrete overlay.

- Moisture Content

To provide for proper penetration, the subsurface pores must be dry to the desired depth of penetration before sealing. The drying requires a sufficient period of dry, warm weather before application as outlined in table 3.1 (5).

Table 3.1 Required Drying Time Before Sealing (Days)

Ambient Temperature	Type of Last Rain		
	Light	Moderate	Heavy
70-85°F (21-29°C)	0.5	1	2
50-70°F (10-21°C)	1	2	3
40-50°F (4-10°C)	2	3	5

Recently patched and overlaid areas should be allowed to cure a minimum of 28 days after placing before sealing, or longer if recommended by the manufacturer. In any case, penetrating sealers shall not be applied if the moisture content of the concrete is greater than 2.5% when tested in accordance with AASHTO T239 standard test procedure.

- Sealer Application

Sealers may be applied by low-pressure pump (with either nozzle or spray bar combinations) and by flood and brush techniques. The sealer should be applied at the rate determined during approval testing (see section 3.2.4).

To ensure proper coverage, the area that can be sealed should be delineated on the deck surface. This area can be calculated from the capacity of the sprayer or container used for flooding the sealer. For example, if an application rate of 120 ft^2/gal (2.9 m^2/L) is desired, and the low-pressure spray container used for application has a capacity of 2 gal (7.6 L), then a 240 ft^2 (22.2 m^2) area should be delineated on the deck as a guide.

In addition, the sealer shall contain a fugitive dye to enable the solution to be visible on the treated surface for at least 4 hours after application. The fugitive dye shall not be conspicuous more than 7 days after application when exposed to direct sunlight. The sealer should be applied from the low area toward the crown (high) to provide for proper saturation. All safety precautions recommended by the manufacturer should be strictly followed. The following additional provisions are recommended (5):

1. Sealers should not be applied when the temperature of the concrete surface is below 40°F (4°C).

2. Before sealing, exposed joint sealants and painted steel joints adjacent to areas to be sealed should be masked off.

3. The sealed areas should be protected from rain and traffic spray for six hours after application.

4. Unused product in open containers shall be properly disposed of if not used within 48 hours.

Note that the application of a sealer may cause bond problems for future overlays if the depth of scarification before the application of the overlay is not greater than the depth of sealer penetration.

3.2.3 Quality Assurance/Construction Inspection: Deck Sealers

The site inspector should ensure that the sealer is applied in accordance with the project specifications and/or the manufacturer's recommendations. The following are critical to a successful application (5):

1. Inspect the deck to ensure that it is free from all laitances and contaminants; that the concrete surface is dry, having been allowed to dry for the specified time since the last rainfall; and that it meets moisture content requirements.

2. Determine that the weather conditions are suitable for the sealer application and that they will remain so for the specified curing time after application. Surface temperature of the concrete shall not be less than 40°F (4°C) during the application and curing period.

3. Determine the application rate from the product acceptance specifications (see section 3.2.4) and ensure that the applicator is prepared to apply the product at the correct rate. Fugitive dye is to be used to gauge the uniformity of the sealer application. Retreatment is to be required in areas where coverage is inadequate. Note that the application rate must be adjusted for the quality and moisture content of the substrate. The inspector should also determine whether sufficient material is present at the site to seal the specified area.

To assess the field performance of a given sealer, it is recommended that the following information be recorded at the time of application: the sealer name and batch number; surface preparation; substrate condition including concrete quality (good-quality low permeability or poor-quality high permeability); moisture content, previous coatings, and residual curing compounds, weather conditions affecting moisture content; and the coverage of the product (5).

3.2.4 Material Performance Specifications: Deck Sealers

The following performance qualities should be used to identify suitable products for sealing concrete decks:

- Water repellency
- Chloride screening

- Penetration depth
- Moisture vapor permeability
- Alkaline resistance
- Resistance to ultraviolet (UV) exposure

Skid resistance and resistance to freeze-thaw scaling may also be considered. Silanes, siloxanes, and blends of the two are examples of products that currently meet these criteria.

- Water Repellency and Chloride Screening

Water repellency and chloride screening can be tested jointly, since the intruding chloride ion is in its hydrated form (6). Currently, there is no accepted standard test method for determining the chloride screening abilities of a sealer, though a variety of acceptance tests are used. Alberta tests the water absorption of sealed cubes over a 120-hour period (7). Florida and Wisconsin apply impressed current to sealed cylinders in a 15% NaCl bath (8, 9). NCHRP 244, *Concrete Sealers for Protection of Bridge Structures*, uses indoor accelerated saltwater ponding cycles alternating with periods of drying and ultraviolet exposure; this is referred to as the Southern Climate Exposure Test (10).

The best procedure may be the one used by Drumm (11). The procedure employs alternating cycles of saltwater ponding and air drying over a 30-week period in an outdoor exposure area. The test method offers several advantages; it simulates conditions similar to those experienced in the field, with the exception of wear; the extended test period allows the in situ testing of the sealers' alkaline resistance because the sealers are applied to new concrete with the highest pH; and the outdoor exposure tests the sealers' resistance to deterioration from ultraviolet exposure. The test procedure is outlined below (11):

1. For each sealer to be tested, one slab 36 x 36 x 4 in. (91 x 91 x 10 cm) shall be cast from the state's standard air-entrained concrete bridge deck mixture. An additional slab shall be cast as a control. The slabs shall contain two ASTM #4 bars 12 in. (30 cm) on center in each direction for temperature and shrinkage control, placed with a minimum cover depth of 2.5 in. (6.4 cm). The minimum cover is specified to prevent the initiation of corrosion during the test period and to prevent the bars from interfering with sampling. The top surface of the slab shall be broom-finished or tined in accordance with state procedure. The slabs shall be cured under moist burlap for 28 days followed by a sufficient period of air drying (see section 3.2.2, moisture content) before the application of the sealer.

2. One powdered chloride sample should be taken from the side of each new slab to a depth of 2.0 in. (5 cm) to determine background chloride contents.

3. Before sealing, the slabs should be lightly sandblasted to remove laitances.

4. The sealer to be tested should be applied at a rate in the middle to upper range of the manufacturer's recommendations.

5. Once the sealer has dried, ponding dikes should be secured to the top surface of the slab. The dikes are fashioned from thick, insulating Styrofoam 0.5 in. (13 cm) cut to a height of 1.25 in. (3.2 cm). The strips are fastened around the top perimeter of the slabs with silicone rubber.

6. The slabs are ponded weekly to a depth of 0.5 in. (1.3 cm) with a 3% NaCl solution. The solution is allowed to stand on the slabs for three days, and is then removed with a wet/dry vacuum. The slabs are then allowed to air dry for four days before the reapplication of the ponding solution. This cycle is continued for 30 weeks. The slabs are covered with translucent (white) plexiglas during the ponding phase to prevent evaporation and dilution by rainfall. Clear plexiglas may cause a "greenhouse" effect which will cause excessive heating of the ponding solution.

7. Two powdered concrete samples are collected from each slab for chloride analysis after 10, 20, and 30 weeks of ponding. The samples are collected as a function of depth in 0.5 in. (1.3 cm) increments for mean depths of 0.5, 1.0, 1.5, and 2.0 in. (1.3, 2.5, 3.8, and 5 cm). Samples from the top 0.25 in. (6 cm) are collected separately and discarded. The samples are collected and analyzed using the procedures outlined by Herald *et al*, (12) in ASTM C 114, or in AASHTO T260.

8. The sample holes are filled with an epoxy mortar and topped with a coating of neat epoxy. Subsequent samples are taken at least 6 in. (15 cm) from the previous sample hole.

- Criteria for Acceptance

Graphs of the percent reduction of chloride intrusion over time should be prepared. Since the background chlorides measured in step 2 above would typically be locked up in the cement matrix or the aggregate and therefore would not contribute to the initiation of corrosion, they should be subtracted as a base level from the readings taken at 10, 20, and 30 weeks. The percent reduction can be calculated as follows:

$$\%Reduction = \frac{CON_t - S_t}{CON_t} \times 100$$

where CON_t = Chloride content of the control at time t (weeks) adjusted for the background chlorides at $t = 0$.

S_t = Chloride content of a sealer specimen at time t (weeks) adjusted for the background chlorides at $t = 0$.

The graphs of percent reduction over time can be used as indicators of early sealer failure due to alkaline or UV deterioration. The performance acceptance for chloride screening should be based on the readings taken after 30 weeks of ponding. The desired level of reduction can be estimated as follows (11):

$$\%Reduction_{min} = \left(1 - \frac{T_{SL}}{50}\right) \times 100$$

where T_{SL} = Service life of the sealer (years)
50 = Assumed desired service life of the structure (years)

Using a typical T_{SL} of 5 years, the value of $\%\mathrm{Reduction}_{min}$ would be 90%. Note that this performance level must be maintained or exceeded throughout the life of the sealer to achieve the desired service level. That is, no more than 10% of the total chlorides that would initiate corrosion in 50 years can accumulate in the concrete over the 5-year service life.

- Penetration Depth

The penetration depth of a sealer is normally measured by splitting a core taken from a sealed substrate and measuring the nonwettable band. An initial penetration depth of at least 0.15 in. (0.38 cm) and ideally 0.25 in. (0.6 cm) is recommended to provide for wear and protection from UV degradation. The actual depth of penetration for a given sealer will vary with the quality and moisture content of the substrate.

Note that the visual indication provided by the nonwettable band can be misleading. The water-repellent effect of a true penetrant must be uniform for the various depths of penetration. This is termed uniform gradient permeation (UGP) (6). The Alberta Transportation and Utilities test procedure for measuring the waterproofing performance of sealers measures this by comparing the absorption of a concrete cube subject after mechanical abrasion (7).

- Moisture Vapor Permeability

Moisture vapor permeability prevents moisture from being trapped inside when the concrete is sealed. As the concrete dries out, the electrical resistance of the concrete increases significantly, further inhibiting corrosion. This is especially important when sealing new concrete. Therefore, a minimum vapor transmission of 80% is recommended (13).

Currently there is no accepted test method for measuring the vapor transmission of sealers. Vapor transmission can be determined by the Alberta procedure (7). The moisture content of concrete cubes at 100% relative humidity is determined by comparing their oven-dry and saturated surface dry (SSD) weights. The cubes are allowed to dry to 70% of their SSD moisture content. Then they are sealed, and their weight is measured. The percent vapor transmission is determined by comparing the weight loss of the sealed cube to that of a control cube over a 14-day period. However, during this same period some of the volatiles in the sealer will be lost, further increasing weight loss. This loss can be estimated by using ASTM D 2369, *Standard Test Method for Volatile Content of Coatings.*

- Additional Criteria

The reduction in skid resistance caused by the sealer may also be considered. A reduction of 5 points (wet) using the British pendulum test ASTM E 303 should be considered acceptable, provided the skid resistance is not reduced below an acceptable minimum value. Northern states and provinces may also be concerned with freeze-thaw scaling. The resistance of a sealer to freeze-thaw scaling can be determined using the procedures outlined in ASTM C 672 *Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals.*

3.2.5 Superstructure and Substructure Sealers/Coatings

3.2.5.1 Sealers

- Description

A sealer is a solvent- or water-based liquid applied to a prepared concrete surface. Both penetrating sealers and surface sealers are recommended for substructure and superstructure elements. Penetrating sealers are silanes and siloxanes (or combinations) that react with the pore walls of the hardened cement paste to create a nonwettable surface (see section 3.2.1.1). Surface sealers are pore-blocking materials such as linseed oil or epoxy.

- Limitations

Sealers are not to be applied to concrete elements with active corrosion sites or to critically chloride-contaminated concrete. (See section 3.2.1.1 for methods and techniques for identifying active corrosion sites and critically chloride-contaminated areas.) Individual environmental site exposure conditions must be considered in selecting a sealer to be used on substructure and superstructure elements. Epoxies are susceptible to degradation by UV light. The service life of surface sealers may be limited by abrasive wave or ice actions.

- Estimated Service Life

The service life of penetrating sealers (silanes and siloxanes) applied to substructure and superstructure components is 5 to 7 years with a recommendation of reapplications every 6 years.

The service life of surface sealers (epoxies and linseed oil) applied to substructure and superstructure components *not* exposed to abrasive wave or ice actions is 1 to 3 years with a recommendation of reapplication every 2 years. If the component is subjected to abrasive wave or ice actions, the service life of surface sealers may be less than 1 year.

- Estimated Price

Price was determined from an analysis of bids for silane, siloxane, and linseed oil. Prices for other sealer types such as epoxies were not determined. Price includes surface preparation and all labor, equipment, and materials, and is influenced by quantity.

For silanes and siloxanes, the unit price is \$8 to \$10/yd^2 (\$9.50 to \$12/m^2). The following relationship can be used to adjust the price for quantity:

$$y = 8.652 + (7.04 \times 10^{-5})X + [(56.077)/(X^{1.2394})]$$

where y = Predicted national adjusted price, \$/yd^2

X = Job quantity, yd^2

For boiled linseed oil, the unit price is \$3 to \$5/yd^2 (\$3.50 to \$6/m^2). The following relationship can be used to adjust the price for quantity:

$$y = 1.375 + (-3.0 \times 10^{-5})X + [(10.895)/(X^{1.0123})]$$

where y = predicted national adjusted price \$/yd^2

X = job quantity, yd^2

3.2.5.2 Coatings

- Description

A coating is a one- or two-component organic liquid that is applied in one or more coats to a prepared concrete surface. Coating materials have a high solid content, usually 100%, have a surface film thickness of 1 to 3 mil (25 to 76 μm), and usually do not contain aggregate. Organic coating materials are normally epoxies, acrylics, or urethanes.

Coatings usually do not permit water vapor transmission, and thus permeability to both water vapor and liquid water is very low.

- Limitations

Coatings are not to be applied to concrete substructure or superstructure components with active corrosion sites or to critically chloride-contaminated concrete (see section 3.2.1).

Coatings are not to be applied to surface-dry but saturated concrete.

Individual site conditions must be considered in the selection of a coating material. Epoxies are abrasive resistant and have a high adhesive strength, but are susceptible to degradation by UV light. Acrylics are brittle and normally have a low impact strength. Urethanes have a high impact strength but have a low resistance to abrasive forces.

- Estimated Service Life

Field exposure conditions may influence the service life of coatings. Bridge components subjected to sea spray may have a shorter service life than those exposed to deicer salt runoff water. Coating materials also appear to affect service life. Table 3.2 gives estimated service lives for epoxies, urethanes, and methacrylates for sea spray/splash and deicer salt runoff water environments.

Table 3.2 Estimated Service Lives for Non-traffic Surfaces

Material	Service Life (years)	
	Sea Spray/Splash	Deicer Salt Runoff Water
Epoxy	6 - 10	10 - 14
Methacrylate	9 - 13	13 - 17
Urethane	10 - 14	14 - 18

- Estimated Price

Price was determined from an analysis of bids for epoxy. Price was not determined for methacrylate and urethane. Price includes surface preparation and all materials, labor, and equipment to furnish and apply an epoxy coating to substructure and superstructure elements. Quantity and quantity times the number of bidders are the best predictors of price; because the latter is only a slightly better predictor. The quantity relationship is used here. The quantity-price relationship for epoxy coating applied to substructure and superstructure components is as follows:

$$y = -68.285 + (0.0008766)X + [(157.53)/(X^{0.09199})]$$

where y = predicted national adjusted price, \$/yd^2

X = job quantity, yd^2

Thus for a typical three-span bridge, the predicted national adjusted price to apply an epoxy coating to the piers and caps, a total surface area of 300 yd^2, of surface area is \$25/yd^2 (\$30/m^2).

3.2.6 Construction Procedure: Superstructure and Substructure Sealers/Coatings

- Surface Preparation

The performance of any sealer or coating system is contingent upon proper surface preparation. The surface should be free from any latencies and contaminants such as grease, oil, wax, or residual curing compounds. Most coating failures occur because of debonding of a weak surface layer of the substrate concrete. Such debonding is caused by the high internal stresses in the coating (14). Additionally, the surface and subsurface voids should be dry to prevent moisture from collecting at the coating-substrate interface before the coating can cure properly.

The following should not be used in new construction or rehabilitation for areas to be coated: form release agents such as oil, grease, wax, or silicone; membrane curing compounds; and rubbed mortar finishes.

- Cleaning Methods

Sand- or gritblasting is the preferred method. To prevent surface contamination, oilers should be removed and oil and water traps installed on the compressed-air source before blasting.

Waterblasting is not recommended, since an adequate drying time must be provided before the application of the sealer/coating.

Acid etching is strongly discouraged. The results are not as satisfactory as gritblasting, and etching requires water flushing to remove residual acid. Also, water flushing may cause corrosion of metallic items, and spillage or disposal may cause environmental problems.

Before sealing or coating, the surface should be thoroughly air-blasted to remove dust and debris. The compressed-air source shall be modified as described above (sand or gritblasting.)

- Patching

Before the application of the sealer/coating, all irregular surface protrusions shall be removed and all surface voids filled. Areas damaged by reinforcement corrosion shall be patched in accordance with the guidelines in section 3.3.13 through 3.3.15 (cast-in-place portland cement concrete) and 3.3.16 through 3.3.19 (shotcrete). Care must be taken when patching shallow or surface voids, such as honeycombed areas. The high internal stresses in the surface coating may cause normal portland cement-based patches to debond. Therefore, these areas should be patched with polymer-based materials compatible with the surface coating as recommended by the manufacturer. Visible dormant cracks should be sealed by epoxy injection before sealing/coating.

- Moisture Content

The provisions outlined for deck sealer in section 3.2.2 shall be followed.

- Sealer/Coating Application

Sealers should be applied as outlined in section 3.2.2. Coatings should be mixed to produce a uniform and homogeneous mix. The manufacturer's safety precautions and environmental protection measures should be strictly followed during mixing. Once the mixed coating has been allowed to stand for the recommended induction period (the time it takes for the coating components to react), it may be applied by brush, roller, or spray as necessary to ensure even coverage. For roller application, long-nap rollers should be used for rough surfaces; short-nap rollers can be used for smoother surfaces. For spray application, a low-pressure, externally atomized spray gun should be used. A thinner recommended by the manufacturer should be added to the coating mixture before spraying (15). The coating should be applied when the temperature is between 50 and 90°F (10 and 32°C).

Two applications of the coating should be applied to ensure even coverage and minimize the likelihood of pinholes. Each application should produce a dry film thickness of 2 to 3 mil (51 to 76 μm). The second coat is normally applied 24 hours after the application of the first coat, but this can vary with environmental conditions and material type. The coating should be applied at the rate of coverage approved during the material acceptance tests.

The following additional provisions are recommended (5):

1. Before coating, all bearings, painted steel surfaces, exposed bituminous materials, and joint sealers should be masked off.

2. The coated surface shall be protected from rain and traffic spray for at least six hours after application. Use a longer protection period if warranted environmental conditions and material type.

3. Unused product in open containers should be properly disposed of if not used within 48 hours.

3.2.7 Quality Assurance/Construction Inspection: Superstructure and Substructure Sealers/Coatings

The site inspector should ensure that the sealer/coating is applied in accordance with the project specifications and/or the manufacturer's recommendations. The following steps are critical to a successful application (5):

1. Inspect the concrete substrate to ensure that it is free from all latencies and contaminants and that the concrete surface is dry, having been allowed to dry for the specified time since the last rainfall.

2. Determine that the weather conditions are suitable for the sealer application and that they will remain so for the specified curing time after application.

3. Ensure that multicomponent sealers/coatings are adequately mixed and that they are allowed the specified induction period before application.

4. Determine the application rate from the product acceptance specifications (see section 3.2.8), and ensure that the applicator is prepared to apply the product at the correct rate. Note that the application rate may need to be adjusted for the quality, surface roughness, and moisture content of the substrate. The inspector should also determine whether sufficient material is present at the site to seal/coat the specified area.

5. Ensure that all necessary safety precautions and environmental protection measures are strictly followed during mixing and application.

To assess the field performance of a given protective agent, it is recommended that the following be recorded at the time of application: the sealer/coating name and batch number; surface preparation; substrate condition including quality (good-quality low permeability or poor-quality high permeability), moisture content, texture, previous coatings, and residual curing compounds; weather conditions affecting moisture content; and the coverage of the product (5).

3.2.8 Material Performance Specifications: Superstructure and Substructure Sealers/Coatings

- Penetrating Sealers

See section 3.2.4.

Epoxy compounds. A wide variety of chemical compositions are grouped as epoxies. The only requirement for such classification is that the monomer molecule contain two carbon atoms and one oxygen atom in a triangular configuration. Epoxies may be one- or two-component products. The most common composition used for sealers/coatings is a reaction between bisphenol A and epichlorohydrin (16). The actual chemical composition and especially the choice of the curing agent will affect the chemical and mechanical properties of the sealer. Epoxy compounds used as sealers are either water- or solvent-based, typically with a solids content less than 50%.

Acceptance Criteria. The acceptance of the sealer for use should be based on chloride screening and moisture vapor permeability as outlined in section 3.2.4. The acceptance criteria should be as follows:

1. $\%Reduction_{min}$ should be greater than 90% after 30 weeks of testing.

2. A minimum vapor transmission of 35% is recommended (13).

- Surface Coatings

Epoxy compounds. Epoxy compounds used as coatings are similar to the compounds described above, except that the solids content should be greater than 50% (generally 100%).

Acrylic compounds. Acrylic compounds can be polymers or copolymers of acrylic acid, methacrylic acid, esters of these acids, or acrylonitrile (17). Examples are acrylic resins produced from the vinyl polymerization of acrylic monomers and methyl methacrylate (14).

Urethane compounds. Urethane compounds are based on the reaction of two chemicals: a resin component (polyol) and an isocyanate curing agent (14). The chemical and mechanical properties of the urethane coating depend on the choice of polyol.

Acceptance citeria. The sealer/coating materials should be evaluated for chloride screening, vapor permeability, and shrinkage. Chloride screening and vapor permeability are presented in section 3.2.4. The acceptance criteria should be as follows:

1. The acceptable values for $\%Reduction_{min}$ for each compound group are presented in Table 3.3.

Table 3.3 Acceptable Values for $\%Reduction_{min}$

Compound	$\%Reduction_{min}$
Epoxy	90
Acrylic	95
Urethane	95

2. A minimum vapor transmission of 35% is recommended (13).

3. The compounds should pass ASTM C 883, *Standard Test Method for Effective Shrinkage of Epoxy-Resin Systems Used with Concrete.*

3.3 Repair Methods

Repair methods are used to restore a bridge component to an acceptable level of service. A deck can be repaired to restore the ride quality to near original condition. A column can be repaired to restore the structural capacity to original design conditions.

Repair methods do little to address the cause of the deterioration. For the subject case, repair methods do little to stop or significantly retard corrosion of the reinforcing steel.

The repair methods presented in this section include partial- and full-depth patching of decks (sections 3.3.1 through 3.3.4) and of substructure and superstructure components (sections 3.3.12 through 3.3.18); deck overlays (sections 3.3.5 through 3.3.11); and encasement and jacketing methods for substructure and superstructure components (sections 3.3.19 through 3.3.22).

3.3.1 Deck Patching

- Description

Patching methods are used to replace localized areas of deteriorated concrete (spalls and delaminations).

By its nature, the deterioration of concrete caused by the corrosion of the reinforcing steel ultimately results in the exposure of the reinforcing steel. For decks, the depth of deterioration may include the top layer of reinforcing steel or both the top and bottom layers of reinforcing steel. If only the top reinforcing mat is corroding, a partial-depth repair would be used. For partial-depth deck repairs, the deteriorated concrete is removed to the depth required to provide a minimum of 0.75 in (1.9 cm) clearance below the top layer of reinforcing steel. Maximum depth of removal for a partial-depth repair should not exceed half the deck thickness.

Corrosion of both the top and bottom layers of reinforcing steel requires full-depth repairs. The concrete within the delineated area for the entire deck thickness, normally 8 in. (23 cm), is removed.

Partial-depth deck patching materials include portland cement concrete, quick-set hydraulic mortar and concrete, and polymer mortar and concrete. Portland cement concretes are used for full-depth deck patches.

Bituminous concretes are also used for partial-depth deck repairs. However, because of their short service life (1 year or less), bituminous concrete patches are considered temporary patches and are thus outside the scope of this manual.

- Limitations

Deck patches have a relatively short service life because they do not address the cause of the problem, corrosion of the reinforcing steel, but address only the symptoms; spalling and delaminations.

When concrete contaminated with chloride beyond the threshold level is left in place in the area surrounding the patches, patches often accelerate the rate of deterioration of the surrounding concrete. The patch concrete area acts as a large noncorroding site (cathodic area) adjacent to corroding sites and increases the rate of corrosion.

- Estimated Service Life

The service life of deck patches is somewhat dependent on type (partial- or full-depth) and material (portland cement concrete, quick-set hydraulic mortar or concrete, or polymer mortar or concrete). The best estimate of the service life of deck patches is 4-10 years.

- Estimated Price

Partial-depth portland cement concrete (PCC) prices were determined from analysis of bid prices. These prices include all labor, material, and equipment costs associated with the removal of unsound concrete, preparation of concrete surfaces, repair or replacement of damaged reinforcing steel, and the furnishing, placing, finishing, and curing of the PCC patch. Minimum removal depth is 0.75 in. (19 cm) below the top layer of the reinforcing steel, with a maximum depth of half the slab thickness.

The price depends on job quantity and number of bidders, and can be predicted from the following relationships.

$$y = 132.89 + [(1{,}382{,}600)/(X^{2.381})]$$

where y = Predicted national adjusted price, ($/yd^2$)

X = Job quantity, (yd^2) times number of bidders

If information on the expected number of bidders is not available, use the job quantity to determine price:

$$y = 134.4 + (0.00460)X + [(316{,}200)/(X^{3.345})]$$

where y = Predicted national adjusted price, ($/yd^2$)

X = Job quantity, yd^2

For a bridge deck requiring 50 yd^2 of partial depth patching with PCC and on which three firms are expected to bid, the predicted national adjusted price of PCC partial depth patch is \$142/yd^2 (\$170/m^2).

If the number of expected bidders is unknown, the predicted national adjusted price for the same 50 yd^2 is \$135./yd^2 (\$161/m^2).

Partial-depth quick-set hydraulic mortar/concrete deck patch price was determined from analysis of bid prices and includes the same items listed for partial-depth PCC deck patches except for the material change. The price can be predicted from the following relationship:

$$y = 235.16 + (-0.03712)X + [(1408.71)/(X^{0.5014})]$$

where y = Predicted national adjusted price, (\$/yd^2)

X = Job quantity, (yd^2)

For a job quantity of 50 yd^2, the partial-depth quick set hydraulic mortar/concrete deck patch price is \$431/yd^2 (516/m^3).

Partial-depth polymer mortar/concrete deck patch price was determined from analysis of bid prices and includes the same items listed for partial-depth deck PCC patches, except that epoxy rather than portland cement is used as the binder. Other polymer mortar/concrete patching methods were not available for this study (1). The price can be predicted from the following relationship:

$$y = 266.79 + (0.02933)X + [(83{,}400)/(X^{1.2214})]$$

where y = Predicted national adjusted price, (\$/yd^2)

X = Job quantity (yd^2) * contract amount (\$) divided by maintenance and protection of traffic, (MPT) cost (\$)

For a job in which the partial-depth patch quantity is 50 yd^2, the contract amount is \$80,000, and the MPT cost is \$20,000, the price for partial-depth deck patching using polymer mortar/concrete is \$402 \$/yd^2/(\$480/m^2).

Full-depth PCC patch price includes all labor, material, and equipment costs; surface preparation; restoration of damaged reinforcing steel; and furnishing, placing, finishing, and curing the patch concrete. Average deck slab thickness is 9 in. (23 cm). Price was determined from an analysis of bid prices and is influenced by job quantity:

$$y = 221.11 + (0.0610)X + [(3794.1)/(X^{1.8688})]$$

where y = Predicted national adjusted price, ($/yd^2)

X = Job quantity, (yd^2)

For a job of 25 yd^2 of full-depth PCC patching, the price is $232/yd^2 ($277/m^2).

3.3.2 Construction Procedure: Deck Patching

- Delineation of Areas to Be Removed

Areas of unsound concrete should be located using drag chains and hammer. The unsound concrete is evidenced by a hollow sound. The area to be removed should be delineated by the site engineer as the unsound or delaminated area plus a periphery of 3 to 6 in. (8 to 15 cm). Delineations should be made in a manner that minimizes cuts, avoids small "islands" of sound concrete, and avoids acute angles.

The delineated area should be outlined with a saw cut 0.75 in. (19 cm) deep. Care should be taken to avoid cutting existing reinforcing steel. In no case are feathered edges acceptable.

- Removal of Unsound Concrete

The unsound concrete should be removed with pneumatic breakers of less than 35 nominal pounds (15.9 kg). Pneumatic breakers should be fitted with bull-point chisels (pointed). Fifteen-pound (6.8 kg) chipping hammers should be used to provide clearance around reinforcing bars. The pneumatic breakers should be operated at an angle no greater than 45° from the horizontal.

The minimum depth of removal is 1.0 in. (2.5 cm) around the periphery of the exposed reinforcing steel. However, concrete should be removed to the depth necessary to expose sound concrete. Patches between 0.75 in. (1.9 cm) below the depth of the reinforcing steel and half the deck thickness are hereinafter referred to as partial-depth patches. Patches requiring removal greater than half the deck thickness should be removed to full depth and are hereinafter referred to as full-depth patches.

- Preparation of Repair Cavity

Once all the unsound concrete has been removed, the cavity should be blasted clean to remove all loose material and provide a dust-free surface. All exposed reinforcing steel should be blasted to near white metal (all scale and rust removed). Reinforcing bars with greater than 25% sectional loss as determined by the engineer should be lapped with reinforcing bar of equal diameter for 30 bar diameters on either side of the deteriorated area. Following the completion of blasting, the cavity should be airblasted to remove all dust and debris. The compressed-air source used for blasting should have any oilers removed and oil traps installed.

Full depth patches will require formwork constructed to prevent leakage of mortar. Formwork for small areas can be suspended from existing reinforcing bars with wire ties. Formwork for larger areas should be supported by blocking erected within the stringers or box girders.

- Application of Bonding Agents

Table 3.4 describes further repair cavity preparations to ensure adequate bond.

Table 3.4 Bonding Agents and Cavity Preparation

Patch Type	Patch Material	Saturate to SSD	Bond Compound
Partial depth	PCC	Yes	Neat grout
	Rapid-set hydraulic cement	Yes	Neat grout
	Polymer concrete	No	Prime coat
Full depth	PCC	Yes	Neat grout

The neat grout shall be prepared by sieving or brooming the coarse aggregate out of a portion of the patch mortar or concrete. Additional water may be added to the grout to produce a creamy consistency. The grout shall be applied with a stiff brush or broom.

Patches to be placed using a polymer mortar/concrete should be primed with a neat polymer resin as recommended by the manufacturer.

- Placement and Consolidation

The patch material may be batched and mixed at the site or supplied ready-mixed. The patch material should be placed in a manner that will prevent segregation. The patch material should be consolidated with an internal vibrator and screeded. The surface of the patch should be floated and textured to match the surrounding concrete surface.

- Curing

PCC and hydraulic mortar/concrete must be moist cured for a minimum of 72 hours. Moist curing should be provided by clean, wet burlap covered with plastic sheeting anchored around the patch perimeter with sand. Shorter moist curing times may be approved by the site engineer in rapid patching conditions. For rapid repair materials, the patches should be cured until the structure is to be opened to traffic or for the minimum time recommended by the manufacturer. Note that shorter curing times will produce an inferior repair.

3.3.3 Quality Assurance/Construction Inspection: Deck Patching

The engineer will be responsible to perform the following:

1. Delineate the areas of concrete to be removed and the depth of removal. The engineer should ensure that all of the provisions for concrete removal are strictly enforced.

2. Determine that the patch cavity is properly prepared.

   - Ensure that the exposed reinforcing steel is free from all rust and scale.

   - Determine the sectional loss and specify lapping of reinforcement bars where required.

   - Ensure that the patch cavity is free from blast material and is clean, dry, and dust-free.

   - Ensure that the substrate is thoroughly coated with the bonding compound as specified in table 3.3. If the bonding compound is allowed to dry before placing the patch material, it shall be removed by gritblasting and reapplied.

3. Ensure that the patching materials as supplied meet the requirements outlined in section 3.3.4.

   The following are recommended for PCC and rapid set hydraulic cement concrete:

   - Inspect the concrete mixer, water-measuring device, and aggregate-weighing scale before the first concrete is batched and mixed. Aggregate proportioning by volume or shovel is not acceptable. Retempering through the addition of water to the original mixture shall not be permitted.

   - Prepare compressive-strength cylinders and perform slump and air-content tests. The minimum requirements for testing frequency should be the initial batch on each day and a second randomly chosen batch.

   - Atmospheric conditions should be monitored and recorded.

4. Following placement, the patches shall be checked for levelness. Patches deviating more than 0.125 in. (0.3 cm) from the surrounding concrete shall be replaced. After the patches have cured, they shall be sounded with a hammer to check for voids.

3.3.4 Material Performance Specifications: Deck Patching

- Portland Cement Concrete

PCC used for patching should meet the following requirements:

1. PCC (ASTM C 150), type I or II, 611 lb/yd^3 (362 kg/m^3)
2. Maximum water/cement ratio 0.42
3. Maximum aggregate size 0.5 in. (1.3 cm)
4. Total air content 6% to 8%
5. Minimum compressive strength 3,500 psi (24 MPa) at 7 days

To prevent shrinkage, the total water content should be kept to a minimum. If an increased slump is desired to improve workability, a high-range water reducer (superplasticizer) should be added. High-range water reducers should not be used without first testing trial batches.

- Rapid Set Hydraulic Cement Mortar/Concrete

All materials should be prequalified with laboratory testing and a minimum of 1 year field testing. Rapid set hydraulic cement/mortar should meet the following requirements:

1. Minimum of 30 minutes time to initial set as tested by ASTM C 403, *Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance*. The mixture shall have sufficient workability to allow placement and consolidation before initial set.

2. Field-cured cylinders should have a minimum compressive strength of 1,200 psi (13.8 MPa) at 2 hours.

3. Expansion/shrinkage should be tested in accordance with ASTM C 157, *Standard Test Method for Length Change of Hardened Hydraulic Cement Mortar and Concrete*. The following allowances are recommended (18):

Cured in water	+0.20%
Cured in air	-0.20%
Difference between curing in air and water	0.30%

4. Freeze-thaw durability should be tested in accordance with ASTM C 666, *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing*. The specimens should maintain at least 80% of their relative

dynamic modulus of elasticity after 300 cycles of testing.

5. The material shall not contain any other form of chloride.

6. The product shall be used before its recommended shelf life expires.

Note: No on-site testing is required for prepackaged, approved rapid-set materials. Most products require *EXACT* measurement of liquid components; the manufacturer's directions should be strictly followed. If aggregate is to be added to prepackaged mortars it should be preweighed.

- Polymer Concrete

Two systems of polymer concrete are generally employed; epoxy resin with curing agents or high-molecular-weight methyl methacrylate with promoters and initiators.

Epoxy Resin Systems. The epoxy system should conform to ASTM C 881, *Standard Specification for Epoxy-Resin-Base Bonding Systems for Concrete*. A type III system should be used with a grade and class that will meet the job requirements.

Epoxy mortar/concrete generally consists of four to seven parts dry silaceous aggregate to one part resin by weight. The maximum aggregate size should be less than 0.50 in. (1.3 cm). Aggregate passing the #100 sieve is generally excluded. The aggregate should preferably be oven dry, and in no case should it have a moisture content greater than 0.5% (19).

The two epoxy resin components should be thoroughly mixed to a uniform color before the addition of the aggregate. The quantity of each component specified by the manufacturer should be strictly followed. Epoxy mortar/concrete should be mixed in shallow pans or conventional concrete mixers. The epoxy mortar/concrete should be placed into a dry repair cavity primed with a coat of neat resin.

The chemical reaction between the resin components is exothermic (produces heat). The time of set of the epoxy concrete will be affected by air temperature, deck temperature, and the mass of the patch material. The rate of hardening will be significantly reduced at temperatures below 70°F (21°C). The reaction may be accelerated by heating the repair cavity before placing the concrete. Care must be taken because too much heat causes flashsetting.

Tools used for mixing and placing should be disposable if possible. If not, they should be cleaned with an organic solvent recommended by the manufacturer.

Caution should be exercised when handling epoxy compounds. Epoxies can cause sensitization, which may lead to dermatitis. Care should be used to avoid contact with skin and eyes. Goggles, gloves, and protective clothing should be worn. Epoxies should only be used in well-ventilated areas.

More information on the use of epoxy mortar/concrete can be obtained from ACI 546.1R, *Guide for Repair of Concrete Bridge Superstructures*, chapter 6 (19), and ACI 503.4, *Standard Specification for Repairing Concrete with Epoxy Mortars* (20).

Methyl Methacrylate Concrete. The binder consists of high-molecular-weight methyl methacrylate (HMMA) monomer that is polymerized in place. Initiators and promoters are added to start polymerization to accelerate polymerization, respectively. HMMA is preferred, since it is less flammable than other methyl methacrylate monomers. Other advantages of HMMA are low viscosity, high bond strength to concrete, and relatively low cost (19). Both user-formulated and prepackaged systems are available.

The quantity of HMMA monomer to be mixed should be divided into two equal portions. The entire required quantity of initiator should be thoroughly mixed with half the monomer, and the entire required quantity of promoter mixed with the other half of the monomer. Then the two portions of monomer should be blended. The monomer should be mixed in clean, electrically grounded containers. Mixing should be conducted with nonsparking paddles powered by explosion-proof motors. Mixing should be conducted in a well-ventilated area out of direct sunlight. A class B fire extinguisher should be present during mixing and placing (20).

Once the HMMA components are mixed, the mixture can be poured over preplaced aggregate or mixed with aggregate and placed as a concrete. The aggregate should conform to ASTM C 33. The maximum aggregate size should be less than 0.50 in. (1.3 cm). The aggregate should preferably be oven dry, but in no case should the moisture content exceed 0.50% (19).

More information on the use of HMMA mortar/concrete can be obtained from ACI 546.1R, *Guide for Repair of Concrete Bridge Superstructures*, chapter 6 (19).

3.3.5 Deck Overlays

- Description

Overlays are used to restore the deck riding surface to as-built quality and to increase effective cover over the reinforcing steel. Overlays discussed in this section include latex-modified concrete (LMC), low-slump dense concrete (LSDC), and hot-mix asphaltic concrete with a preformed membrane (HMAM). These overlays are considered repair methods because the sound, chloride-contaminated concrete is left in place. The overlay has some influence on the service life of the repair, but the amount and degree of the chloride-contaminated concrete left in place remains the most important factor.

- Limitations

LMC, LSDC, and HMAM overlays may increase the dead load and thus decrease the live load capacity of the bridge. The influence of the overlay on the live load capacity of the bridge must be evaluated before one of these overlay systems is specified.

Also, LMC, LSDC, and HMAM overlays should not be placed on decks where the existing concrete may be susceptible to alkali aggregate reactions (silica or carbonate) unless a low-alkali cement is used or other preventive measures have been taken.

- Estimated Service Life

The estimated service life of repair overlays is governed by the area and chloride content of the chloride-contaminated concrete left in place. Since the primary factor that influences the decision to overlay a bridge deck is the extent of surface damage, the chloride-contaminated concrete left in place is similar for the various environmental exposure conditions. It just takes longer for a bridge deck in a mild environment to reach the end of its functional service life than for a deck in a severe environment. Thus, the service life for repair overlays is similar for all environmental exposure conditions. The exceptions are the HMAM repair overlays. The HMAM overlays tend to increase the moisture content of the concrete and thereby increase the average annual corrosion rate. Table 3.5 summarizes data on the service life of these overlays.

Table 3.5 Service Life of Repair Overlays

Overlay Type	Service Life, Years
LSDC	22 - 26
LMC	22 - 26
HMAM	10 - 15

- Estimated Price

The price of LSDC overlays was determined from an analysis of bid prices. The price includes all labor, materials, and equipment necessary to place, finish, and cure a 2 in. (5 cm) LSDC overlay. Note that the price is for an LSDC overlay placed, finished, and cured on a prepared concrete surface. The price does not include the other work normally associated with overlays, such as replacing expansion joints, patching deteriorated (spalled and delaminated) areas, milling the top 0.25 to 0.50 in. (0.63 to 1.27 cm), wetting the surface to be overlaid, and ramping up to and down from the new elevated riding surface.

The price of LSDC overlays is influenced by job quantity and the number of bidders, and can be determined from the following relationship:

$$y = 31.90 + (8.82 \times 10^{-5})X + [(22{,}200)/(X^{0.9791})]$$

where y = Predicted national adjusted price (\$/yd^2)

X = Job quantity (yd^2) times number of bidders

For two bridges with a total of 1,800 yd^2 and four expected bidders, the price is \$36/yd^2 (\$43/m^2) for an LSDC overlay.

The price of LMC overlays was determined from an analysis of bid prices. The price includes all labor, materials, and equipment necessary to place, finish, and cure an overlay 1.25 - 1.5 in. (18 to 3.81 cm) thick on a prepared concrete surface. It does not include other work items normally associated with overlays.

The price of LMC overlays is influenced by job quantity and the number of bidders, and can be determined from the following relationship:

$$y = 32.79 + (2.6 \times 10^{-5})X + [(2246.05)/(X^{0.6961})]$$

where y = Predicted national adjusted price (\$/yd^2)

X = Job quantity (yd^2) times number of bidders

For two bridges with a total of 1,800 SY and four expected bidders, the price is \$38/yd^2 (\$45/m^2) for an LMC overlay.

The price of HMAM overlays was determined from an analysis of bid prices. The price includes all labor, equipment, and materials necessary to furnish and place sheet membranes. It does not include the price of hot-mix asphalt (HMA) because of the large fluctuations in the price of HMA. Nor does the price include the other costs of work items normally associated with a repair HMAM overlay.

The price of HMAM overlays is influenced by job quantity and can be determined by the following relationship:

$$y = 9.781 + (-0.0001692)X + [(3{,}978{,}400)/(X^{3.100})]$$

where y = Predicted national adjusted price (\$/yd^2)

X = Job quantity (yd^2)

For two bridges with a total of 1,800 yd^2, the price is \$9.50/yd^2 (\$11/m^2) for a sheet membrane.

3.3.6 Construction Procedure: Portland Cement-Based Overlays

The following describes the construction procedures, quality assurance, construction inspection, and material performance specifications for the application of LMC, and LSDC.

- Scarification and Removal of Unsound Concrete

The entire deck surface to be overlayed should be scarified to a depth of 0.5 in. (1.3 cm) see chapter 6 for more information. Following scarification, the site engineer should sound the deck using a drag chain and hammer to delineate the areas of unsound concrete. Bituminous patches and areas of unsound concrete should be removed and patched with PCC as outlined in section 3.3.2. The patched areas should be textured to improve the overlay bond. The repaired areas should be allowed to cure for a minimum of 48 hours before placing the overlay. Dormant, visible cracks should be sealed using epoxy injection.

- Substrate Preparation

The deck surface to be overlaid should be gritblasted or shotblasted to remove laitances and any residues, then air blasted to remove dust and debris. The air compressor used for grit and air blasting should have oil and water traps installed.

At least 12 hours (preferably 24 hours) before overlay, the deck should be saturated with water and covered with 4 mil (100 μm) polyethylene sheeting. The deck substrate should be saturated surface dry at the time of overlay; all standing water should be removed.

Blocking should be placed to provide for expansion joints at the same location as existing expansion joints (21).

Screed rails should be installed so that the finishing machine will provide the minimum overlay thickness as specified in table 3.6. The supports for the rails should be adjustable; shims should not be used to adjust rail heights. A dry run should be performed to ensure that the finishing machine provides the minimum overlay thickness.

Table 3.6 Minimum Overlay Thickness

Overlay Type	Minimum Thickness (in.)
LMC	1.5
LSDC	2.0

Note: 1 in. = 2.54 cm

- Placement and Consolidation

The overlay concrete should be mixed at the site using a mobile mixer. A pan is to be placed under the truck to catch leaking oil. Before placement, a trial batch should be prepared in the presence of the site engineer to test the calibration of the mixer. Sufficient mobile mixers should be present on the site to allow the continuous placement of the overlay without delay.

A bonding grout should be thoroughly brushed into the substrate immediately in front of the finishing machine. The grout can be obtained by brushing the coarse aggregate out of a portion of the overlay concrete. The excess coarse aggregate should be discarded. If sufficient grout cannot be obtained, an acceptable grout can be produced by blending equal parts cement and concrete sand with enough water to produce a stiff grout. Care must be taken to prevent the bond grout from drying before the placement of the overlay concrete. If drying is allowed to occur, a temporary construction dam should be placed and the dried bonding grout removed by grit blasting.

The overlay concrete should be placed approximately 0.25 in. (0.6 cm) above final grade.

The overlay concrete should be consolidated and finished using a finishing machine. The self-propelled finishing machine should be equipped with screw augers, vibratory plates, and one or more rotating steel cylinders. The machine should be capable of spanning the entire placement transversely (22). Spud vibrators and hand tools should be used as necessary along the screed rails and for any deep pockets. A burlap or carpet drag can be used to texture the surface.

A lightweight catwalk should follow the finishing machine to allow hand finishing, inspection for levelness, and inspection for density of LSDC overlays and to allow placement of curing materials.

- Curing

Within 30 minutes of placement, the fresh concrete should be covered with a single layer of clean, damp burlap. The edges of the burlap should be overlapped 4 in. (10 cm). LSDC overlays should be continually misted for the first 24 hours of curing and then covered with white polyethylene sheeting. Curing shall be maintained for the minimum times shown in table 3.7. Curing hours should be defined as hours for which the temperature is greater than 45°F (7°C). The overlay concrete should be protected with insulated blankets if the temperature is predicted to drop below 45°F (7°C) (21, 22).

Table 3.7 Minimum Curing Times

Overlay Type	Minimum Curing Time (hours)
LMC	48
LSDC	168[a]

[a]72 hours may be acceptable if traffic conditions warrant

- Skid Resistance

Following 14 days of curing, grooves shall be sawed transverse to the center line. The grooves should be approximately 0.2 in. (0.55 cm) deep and 0.13 in. (0.3 cm) wide, spaced 0.75 in. (1.9 cm) on center. Grooves should terminate within 12 in. (30 cm) of the parapet or curb (36).

More information can be obtained from ACI 546.1R, *Guide for Repair of Concrete Bridge Superstructures* (19) and ACI 548.4, *Standard Specification for Latex-Modified Concrete (LMC) Overlays* (21).

3.3.7 Quality Assurance/Construction Inspection: Portland Cement-Based Overlays

The following should be considered the minimum inspection requirements to ensure a quality overlay. More than one inspector should be present when the overlay is being placed.

- Scarification and Removal of Unsound Concrete

Following the completion of the milling, inspection for delineation and repair of unsound areas should be conducted as described in section 3.3.3.

- Substrate Preparation

In assessing deck preparation, the inspector should determine the following:

1. Ensure that the deck is free from all latencies, organic residues, and debris.
2. Ensure that the substrate is in a saturated surface dry condition and free from standing water.
3. Ensure that the screed rails are properly adjusted to provide the minimum overlay thickness over the entire deck area to be overlaid, and that the

finishing machine is clean and functioning properly.

- Placement and Consolidation

The inspector should account for the following before and during each placement as applicable:

1. Before placement, a trial batch should be prepared to ensure that the specified slump and air content can be obtained with the job materials and mixture. For LMC overlays, the inspector should obtain certification of compliance with FHWA Notice 5140.15, July 1978, for each batch of latex delivered to the job site. The solids content of each batch of latex is determined using ASTM D 1417.

2. Ensure proper proportioning of ingredients by calibrating the mobile mixer. Yield should be checked once a day.

3. Ensure that sufficient mobile mixers are present to allow continuous placement. Slump tests (ASTM C 143), total air-content measurements (ASTM C 231), and a minimum of six compressive-strength cylinders should be prepared from each batch. Slump tests should be performed five minutes after mixing.

4. Monitor the air temperature, concrete temperature (ASTM C 1064), relative humidity, and wind velocity. Overlays shall not be placed under the following conditions:

    - Air temperature is below 50°F (10°C)

    - Evaporation rate exceeds 0.10 $lb/ft^2/hr$ (0.49 $kg/m^3/hr$) as determined by ACI 305R (23)

    Overlays shall be placed at night when the daytime air temperature is above 85°F (29°C).

5. Monitor the application of the bond grout and ensure that the overlay concrete is placed before the grout is allowed to dry.

6. Check the levelness requirements of the finished overlay using a 10 ft (3 m) straightedge.

7. Ensure that LSDC overlays achieve 98% of the theoretical maximum density (ASTM C 138) as tested by a nuclear density gauge in direct-transmission mode (24). Concrete failing to meet the required density should be revibrated.

If the overlay concrete fails to meet the required density after revibration, it should be removed and replaced.

- Curing

The site inspector should monitor the overlay during the curing period to ensure that it is maintained at a temperature in excess of 45°F (7°C) and uniformly saturated; membrane-forming curing components are not acceptable.

Following the curing period, the entire overlay should be sounded with a drag chain and hammer to ensure bond. The surface of the overlay should be inspected for cracking. All unsound areas should be repaired by the contractor.

3.3.8 Material Performance Specifications: Portland Cement-Based Overlays

- Latex-Modified Concrete

Latex admixture conforming to the prequalification standards of FHWA Notice 5140.15, July 1978, should be added at a minimum rate of 18.9 gal/yd^3 (94 L/m^3).

The following materials and proportions are recommended (25):

1. Cement type and content, ASTM C 150 type I or II, 658 lb/yd^3 (390 kg/m^3)
2. Maximum water/cement ratio 0.40
3. Aggregates, ASTM C 33, with a maximum aggregate size of 0.5 in. (1.3 cm)
4. Total air content, maximum 6.5%
5. Slump, range 4 to 6 in. (10 to 15 cm)
6. Minimum compressive strength 4,500 psi (31 MPa) at 28 days

Air-entrainment agents should not be used.

- Low-Slump Dense Concrete

The following materials and proportions are recommended (26):

1. Cement type and content, ASTM C 150 type I or II, 823 lb/yd^3 (488 kg/m^3)
2. Maximum water/cement ratio 0.33

3. Aggregates, ASTM C 33, with a maximum aggregate size of 0.50 in. (1.3 cm)

4. Plastic air content 6% to 7%

5. Slump, maximum 1 in. (2.5 cm)

A high-range water-reducing admixture (superplasticizer) should be added to improve workability.

3.3.9 Construction Procedure: Hot-Mix Asphalt Preformed Membrane Overlays

Deck surfaces should be prepared as described in section 3.3.6 with the following changes:

1. The deck surface shall not be wet to a saturated surface dry condition before overlay.

2. PCC repair patches shall be allowed to cure for a minimum of 14 days before the application of the primer.

- Primer Application

The primer should be thoroughly mixed according to the manufacturer's directions. It should be applied to the deck surface using the method and rate specified by the manufacturer. The primer should be allowed to dry to a tack-free condition before the application of the membrane.

- Membrane Application

The following practices are recommended for membrane installation (27, 28):

1. The membrane should be installed in a shingled pattern to allow water to drain to low areas of the deck and prevent it from accumulating at the seams.

2. The edges of the membrane should be overlapped a minimum of 2 in. (5 cm) and the ends overlapped a minimum of 6 in. (15 cm). Joints should be staggered.

3. The membrane should be rolled into place using a linoleum roller. Care should be taken to eliminate air pockets. Any air blisters greater than 3 in. (8 cm) in any dimension remaining after completion of installation should be punctured with a sharp pointed object like an ice pick and the air forced out. The area surrounding the puncture should be coated with mastic and a membrane patch applied.

4. The edges of the membrane should be sealed with a compatible mastic or polyurethane sealer.

- Hot-Mix Asphalt Overlay

The specified HMA overlay should be applied within 2 days of the application of the membrane. The overlay shall be applied at the temperature specified by the membrane manufacturer. The bottom course of the overlay shall be compacted using a rubber-tired or rubber-tracked paver to prevent damage to the membrane (28). If additional large air blisters develop during the overlay application, they should be punctured as described above.

3.3.10 Quality Assurance/Construction Inspection: Hot-Mix Asphalt Preformed Membrane Overlays

- Surface Preparation

The preparation of the substrate shall be inspected as described in section 3.3.7.

- Membrane Application

The site inspector should monitor the following:

1. Ensure that the deck is properly prepared to receive the membrane.
2. Ensure that the deck is properly primed.
3. Inspect seams for minimum overlap and proper sealing.
4. Ensure that all bubbles and blisters are properly punctured and sealed.

- Hot-Mix Asphalt Overlay

Inspection specifications for the application of HMA are beyond the scope of this report. However, the site inspector should ensure that the asphalt is placed and compacted so as to prevent damage to the membrane and that the laydown temperature is consistent with the manufacturer's recommendations.

3.3.11 Material Performance Specifications: Hot-Mix Asphalt Preformed Membrane Overlays

Currently, there are no standard specifications for preformed membranes. The products used are generally composite laminates of bitumens and synthetic fabric or mesh. Both Maine (27) and Vermont (28) specify products manufactured by W.R. Grace, Protecto Wrap Co., and Royston Laboratories, Inc. The primer recommended by the manufacturer should be used in the placement of the membrane.

Specifications for the HMA wearing surface are beyond the scope of this report. Consideration should be given to the stability of the mix design so that deformation will neither damage the membrane nor become the limiting factor in the life of the HMA preformed membrane overlay system.

3.3.12 Superstructure and Substructure Patching

Patching involves methods used to restore the structural integrity and appearance of deteriorating concrete bridge substructure and superstructure elements such as piers, pier caps, diaphragms, beams, and abutments. The depth of the patch may be shallow (to the level of the reinforcing steel) or deep (a minimum of 0.75 in. (19 mm) below the first layer of reinforcing steel. Patching materials may be PCC, quick-set hydraulic mortar/concrete, or polymer mortar/concrete. In the case of deterioration caused by corrosion of reinforcing steel, patching material is normally limited to PCC. Thus only cast-in-place PCC and shotcrete will be discussed.

3.3.12.1 Cast-in-Place Portland Cement Concrete

- Description

PCC is used to backfill the prepared cavities of corrosion damaged concrete members. The damaged concrete is normally removed to a depth of 0.75 in. (1.9 cm) below the first layer of reinforcing steel.

- Limitations

Substructure and superstructure patches have a relatively short service life because they do not address the cause of the deterioration mechanism, corrosion of the reinforcing steel, but merely the symptom.

In patching these structural elements, consideration must be given to the influence of the amount of concrete removal on the reduction of the structural capacity of the structure remaining in place after removal.

- Estimated Service Life

The service life of substructure and superstructure PCC patches is somewhat dependent on the type of patch (shallow or deep) and whether the leaking deck joints are successfully repaired. The best estimate of service life of substructure and superstructure PCC patches is 5 to 10 years for elements exposed to either splash and spray or melt water runoff.

- Estimated Price

The price was determined from an analysis of bids and includes all labor, materials, and equipment required to remove the unsound concrete, prepare the concrete surface, repair or replace damaged reinforcing steel, and place, finish, and cure the concrete.

Substructure and superstructure patching vary a great deal in complexity, location, and accessibility. The price is influenced mostly by depth and quantity. The following relationships may be used to determine the price of shallow and deep structural patches:

$$y = 487.17 + (-0.36689)X + [(291.233)/(X^{0.8114})]$$

where y = Predicted national adjusted price ($/yd²) for shallow structural patch work

X = Job quantity (yd²)

$$y = 613.13 + (-0.02657)X + \{(3327.83)/(X^{2.9158})]$$

where y = Predicted national adjusted price ($/sy²) for deep structural patch work

X = Job quantity (yd²)

For a 50 yd² job quantity, the prices for shallow and deep structural patches are $481/yd² ($575/m²) and $612/yd² ($732/m²), respectively.

3.3.12.2 Shotcrete

- Description

Shotcrete (also called Gunite) is a pneumatically applied portland cement mortar/concrete used to patch substructure and superstructure elements. The damaged concrete is normally removed to a depth of 0.75 in. (1.9 cm) below the first layer of reinforcing steel.

- Limitations

Shotcrete repair methods do not address the cause of the corrosion deterioration but merely the symptoms. Thus the service life of the shotcrete repair is limited.

The removal of large quantities of damaged concrete at one time may cause a reduction of structural capacity that needs to be considered.

Quality is highly variable and is generally operator dependent. Test panels of application should precede work on the structure.

- Estimated Service Life

The best service life estimate for shotcrete repairs of substructure and superstructure elements exposed to either spray and splash or melt water runoff is 10 to 15 years.

- Estimated Price

The following price relationship for shotcrete structural repairs was determined from an analysis of bid prices. The price includes all materials, labor, and equipment required to prepare the surface to receive the shotcrete, repair or replace any damaged steel reinforcing, attach wire fabric to the structural element, and apply the shotcrete. The price is influenced greatly by job quantity and can be determined from the following relationship:

$$y = 3{,}610.06 + (-6.118)X + [(18{,}820)/(X^{1.851})]$$

where y = Predicted adjusted national price for shotcrete structural repairs ($/yd^3)

X = Job quantity (yd^3)

For 15 yd^3 of shotcrete structural repair, the price is $3,640/yd^3 ($4,770/m^3).

3.3.13 Construction Procedure: Superstructure and Substructure Patching with Cast-in-Place PCC

Patching of superstructure and substructure components should be conducted as described in section 3.3.2 with the following modifications:

- Delineation of Areas to Be Removed

Unsound concrete should be located by sounding with a hammer. Delineation should be conducted as described in deck patching.

- Removal of Unsound Concrete

Unsound concrete should be removed to a depth of 0.75 in. (1.9 cm) below the level of the first layer of reinforcement. Care should be taken to maintain the structural capacity during concrete removal.

- Preparation of Repair Cavity

The cavity should be prepared as outlined in section 3.3.2. Formwork specifications are beyond the scope of this report. However, the following should be considered to ensure a good bond:

1. Formwork shall be sufficiently anchored and braced to prevent bulging. Concrete will act as a fluid and exert hydrostatic pressure on formwork before curing.

2. Formwork should be designed so that the fluid concrete has sufficient head to fill all voids and bond to the substrate, especially when repairing overhead surfaces such as the bottom of pier caps.

More information on formwork can be obtained from ACI 347R, *Guide to Formwork for Concrete* (21).

- Application of Bonding Agents

Formwork requirements preclude the use of bonding compounds for superstructure and substructure repairs.

- Curing

Positive steps should be taken to provide moist curing to superstructure and substructure repairs. Minimum requirements include keeping forms cool and moist and draping exposed areas with wet burlap. Formwork should be left in place for a minimum of seven days for repairs that are not moist-cured. Immediately after the removal of such formwork, a curing membrane should be applied to maintain the internal humidity of the repair concrete.

3.3.14 Quality Assurance/Construction Inspection: Superstructure and Substructure Patching with Cast-in-Place PCC

All the items outlined in section 3.3.3 will be used, with the following additions:

1. Equipment used for transporting concrete at the job site should be inspected before the first placement to ensure that appropriate measures have been taken to prevent segregation as outlined in ACI 304R, *Guide for Measuring, Mixing, Transporting, and Placing Concrete* (22).

2. *All* patches will be sounded after the removal of forms to check for adequate bond and voids. Unsound, voided, or honeycombed areas should be removed and replaced.

3.3.15 Material Performance Specifications: Superstructure and Substructure Patching with Cast-in-Place PCC

Repair concrete for superstructure and substructure components should be designed as outlined in section 3.3.4 using PCC. The following additional criteria shall be considered:

1. Concrete mixtures should be proportioned with a slump of 3 to 4 in. (8 to 10 cm) as tested by ASTM C 143 and may require a high-range water reducer.

2. The lower water/cement ratio used for the repair concrete will typically produce a patch darker than the surrounding concrete. White cement can be blended into the repair concrete to produce a color that matches the existing concrete.

3. The mix design should provide a minimum compressive strength of 4,500 psi (31 MPa) at 28 days.

4. Exceeding the maximum water/cement ratio (0.42) is strictly forbidden. Concrete not meeting consistency requirements should be refused.

3.3.16 Construction Procedure: Superstructure and Substructure Patching with Shotcrete

Shotcrete can be applied as a wet or dry mix. Superstructure and substructure repairs are most commonly carried out using dry-mix mortar. Patching of superstructure and substructure components should be conducted as described in section 3.3.13, with the following modifications:

- Removal of Unsound Concrete

Unsound concrete should be removed to a depth of 0.75 in. (1.9 cm) below the level of the first layer of reinforcement. After removal, the remaining concrete should be sounded with a hammer to ensure that all unsound concrete was removed. Care should be taken to maintain structural integrity during concrete removal. The saw-cut edges of the repair cavity should be tapered to a 45 degree angle to reduce rebound and improve bond.

- Preparation of Repair Cavity

The cavity should be prepared as outlined in Section 3.3.2. Formwork is generally not required for shotcrete repairs. To prevent debonding and limit development and depth of cracking, the use of welded wire fabric is recommended (15). Fabric reinforcement should conform to ASTM A 185, galvanized, welded straight-line 12 gauge wire, spread 2 in. (5 cm) in each direction (2 x 2/W0.9 x W0.9). The wire fabric should be provided with a minimum clear cover of 1 in. (2.5 cm). The wire fabric should be anchored with 0.25 in. (0.6 cm) expansion bolts placed 20 in. (51 cm) on center (15). Finally, an 18 to 20 gauge wire should be stretched across the cavity to act as a grade line. The grade line aids the nozzle operator in determining when the repair is flush with the surrounding concrete.

- Application of Bonding Agents

Bonding agents are generally not recommended for shotcrete applications. However, the repair cavity and surrounding substrate surface should be wetted to a saturated surface dry condition before placement of the shotcrete, to prevent the substrate from drawing moisture out of the repair patch.

- Placement

The following should be considered for the successful application of shotcrete:

1. An experienced crew is required for the successful application of shotcrete. ACI 506.3R provides specifications for the certification of the nozzle operator (29).

2. Equipment should meet specifications outlined in ACI 506R (30). Compressed-air sources should be fitted with oil and water traps to prevent contamination.

3. The nozzle should be kept nearly perpendicular and approximately 2 to 4 ft (0.6 to 1.2 m) from the repair surface. When encasing reinforcing steel, the nozzle should be held closer and at a slight angle to minimize the accumulation of rebound. Exceptional care must be taken when encasing reinforcing steel bars larger than #5 with shotcrete (25).

4. The application of a single layer of shotcrete is preferable to prevent cold layers, each joints. However, overhead surfaces may require the application of multiple layers, each 1 to 2 in. (2.5 to 5 cm) thick, to prevent sagging. Vertical cavities should be filled from the bottom up.

5. Material that ricochets off the repair surface is termed rebound. An airblast should be used to remove this material, primarily aggregate, from adjacent cavities and sound concrete. Rebound material should not be reused.

6. More information on the application of shotcrete can be obtained from ACI 506R, *Guide to Shotcrete* (30).

- Curing

Because of to the high cement contents typically used in shotcrete, curing is exceptionally important, especially for dry-mix shotcrete. Seven days of moist curing should be provided using wet burlap covered with polyethylene sheeting or sprinkling.

3.3.17 Quality Assurance/Construction Inspection: Superstructure and Substructure Patching with Shotcrete

All the items outlined in section 3.3.3 should be followed, with the following additions:

1. Shotcrete crews should be prequalified for the repair work. Test panels a minimum of 4 in. (10 cm) deep should be shot for each position (vertical, overhead, etc.) to be encountered. The test panels should be prepared according to ASTM C 1140, *Standard Practice for Preparing and Testing Specimens from Shotcrete Test Panels*. The test panels should contain reinforcement steel of the size and spacing to be encountered during the repair. Cores 4 in. (10 cm) in diameter should be cut from the cured test panels and tested for compressive strength. Compressive strength measurements should be corrected for their length-to-diameter ratio as outlined in ASTM C 42, *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*. A minimum compressive strength of 5,000 psi (34.5 MPa) is recommended at 28 days (31). Additionally, tests for splitting tensile strength, bond strength, chloride permeability, and flexural strength may be performed (31).

2. The site inspector should ensure that the curing requirements are strictly enforced. Drying shrinkage cracking is typically higher for shotcrete (0.06% to 0.10%) than for conventional concrete (30). Failure to provide proper curing will result in an inferior repair.

3. All patches should be sounded after curing to check for adequate bond and voids. Unsound, voided, or honeycombed areas should be removed and replaced.

4. Shotcrete should not be placed under the following conditions:

   - High winds
   - Temperatures below 40°F (4°C)
   - Rain or threat of rain

3.3.18 Material Performance Specifications: Superstructure and Substructure Patching with Shotcrete

Shotcrete can be applied as either a wet-mix or a dry-mix process. Since it is not feasible to air-entrain the dry-mix process, this method is not recommended for elements exposed to moist freezing and thawing conditions.

- Materials

Cement should be ASTM C 150 type I or II; white cement can be blended in for aesthetic reasons.

Aggregate should conform to ASTM C 33, with a 0.5 in. (1.3 cm) maximum aggregate size.

Calcium chloride should not be used as an accelerating admixture.

Air-entraining admixtures should be used with wet-mix process exposed to freeze-thaw cycles to produce a total air content of 5% to 7%.

Information on the prequalification of materials can be obtained from ACI 506.2R, *Specification for Materials, Proportioning, and Application of Shotcrete* (32).

- Proportioning Wet-Mix Process Shotcrete

Wet-mix shotcrete can be proportioned using the procedures outlined in ACI 211.1, *Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete* (33). The shotcrete can be batched at a batch plant and transported to the site by a transit mixer or batched and mixed at the site. The following are recommended:

1. Slump 1.5 to 3.0 in. (3.8 to 7.6 cm)

2. Cement content 658 lb/yd^3 (390 kg/m^3)

3. Maximum water/cement ratio 0.40

- Proportioning Dry-Mix Process Shotcrete

There is currently no rational procedure for designing dry-mix shotcrete. However, the following may be used as guidelines:

1. The cement/aggregate ratio should be in the range of 3:1 to 4:1 (29).

2. The aggregate gradation shown in table 3.8 is recommended for most typical pressure mortar repair applications (30). However, for very large repair areas a slightly coarser gradation can be used to help control drying shrinkage.

Table 3.8 Recommended Aggregate Gradation for Mortar

Sieve Size (U.S. standard square mesh)	Percent by Weight Passing Individual Sieves
3/8 in. (10 mm)	100
No. 4 (4.75 mm)	95-100
No. 8 (2.4 mm)	80-100
No. 16 (1.2 mm)	50-85
No. 30 (600 μm)	25-60
No. 50 (300 μm)	10-30
No. 100 (150 μm)	2-10

3. The aggregate moisture content should be between 6% to 8% to prevent dusting (30).

4. The mix water is added at the nozzle and gaged by the nozzle operator. The amount of water added should not exceed a water/cement ratio of 0.40.

3.3.19 Superstructure and Substructure Encasement/Jacketing

- Description

Encasement is used on badly deteriorated columns or piers to restore their structural capacity. The damaged concrete is removed, and concrete is placed to fill the cavities and provide a layer of concrete over the as-built structural concrete element, completely encasing the element.

For badly deteriorated abutments and wing walls, a concrete jacket is provided to restore the structural capacity of these elements. The damaged concrete on the exposed face is removed. Concrete is placed to fill the removed concrete cavities and cover the as-built concrete face.

- Limitations

Repair encasements and jackets do not address the cause of the corrosion deterioration and thus have a reduced service life. How removing a large portion of concrete affects the structural integrity of the element during construction must also be considered.

- Estimated Service Life

The best estimated service life for the encasement and jacket repair methods is 10 to 20 years, regardless of the exposure condition (splash and spray or melt water runoff).

- Estimated Cost

Cost was determined by standard engineering estimating procedures. Cost was influenced by the scale of the work and thus is not influenced by quantity for the quantity ranges expected in practice. The estimated national adjusted unit cost for encasement and jacketing work is \$354/yd^2 (\$423/m^2) and \$716/yd^2 (\$856/m^2), respectively.

The encasement cost includes removing damaged concrete, cleaning concrete surfaces, sandblasting reinforcing steel, installing an epoxy-coated rebar cage, setting forms, placing and curing concrete, and stripping forms. The thickness of the concrete encasement layer is 6 in. (15.2 cm).

The jacket cost includes excavating as necessary to expose the deteriorated face concrete, removing damaged concrete, drilling and setting lag studs for form and reinforcement support, installing epoxy-coated welded wire fabric, setting forms, applying epoxy bonding compound, placing and curing concrete, removing forms, and replacing excavated material. The thickness of the concrete jacket is 8 in. (20 cm).

3.3.20 Construction Procedure: Superstructure and Substructure Encasement/Jacketing

Procedures used to encase badly deteriorated columns, piers, or piles in a manner that restores their load-carrying capacity will be termed *encasement*. Procedures used to face badly deteriorated abutments or wing walls in a manner that restores their structural capacity will be termed *jacketing*.

- Delineation of Areas to Be Removed

Unsound concrete should be delineated by sounding with a hammer. Areas to be removed need not be outlined with a saw cut, since the entire periphery of the component will be encased or jacketed.

- Removal of Unsound Concrete

Unsound concrete should be removed using pneumatic breakers of less than 35 nominal pounds (15.9 kg). Pneumatic breakers should be fitted with bull-point chisels (pointed). Small hand chisels, 15 pounds (6.8 kg), should be used to provide clearance around reinforcing bars. The pneumatic breakers should be operated at an angle no greater than 45 degrees to the substrate.

Though the removal of sound concrete to a minimum depth of 0.75 in. (1.9 cm) around the periphery of the bar is not necessary to provide an adequate bond, it is highly recommended for corrosion abatement. Special care should be taken to ensure that the structural integrity of the member is maintained and that the superstructure components supported by the member are suitably jacketed during the repair process.

Note that jacketing will require the excavation of the slope wall adjacent to the repair area. Appropriate shoring should be provided.

- Preparation of Repair Cavity

Once all the unsound concrete has been removed, the cavity should be blasted clean. All exposed reinforcing steel should be blasted to near white metal (all scale and rust removed). Reinforcing bars with greater than 25% sectional loss as determined by the engineer should be lapped with reinforcing bar of equal diameter for 30 bar diameters on either side of the deteriorated area. Following the completion of blasting, the cavity should be airblasted to remove all dust and debris. The compressed-air source used for blasting should have any oilers removed and oil traps installed.

- Placement of Reinforcing Steel

Encasement. Minimum reinforcing steel requirements are typically met by constructing a cage around the column, consisting of #5 (1.6 cm) bars on 6 in. (15 cm) centers vertically and #4 (1.3 cm) tie bars on 9 in. (23 cm) centers horizontally (30). A minimum clearance of 1 in. (2.5 cm) should be provided between the reinforcing cage and the substrate. Standoffs should be placed to provide a minimum 3 in. (7.6 cm) cover over the reinforcing cage.

Jacketing. Minimum reinforcement should consist of 6 in. x 6 in. (15 cm x 15 cm) 6-gauge galvanized steel wire mesh (6 x 6/W2.9 x W2.9). The wire mesh should be anchored with 0.5 in. (1.3 cm) galvanized expansion bolts placed 20 in. (51 cm) on center, horizontally and vertically (31). Formwork should be placed to provide a minimum 3 in. (7.6 cm) cover from the wire mesh.

- Formwork

The substrate should be moistened to a saturated condition before placing the formwork.

Encasement. Formwork may consist of metal, wood, fabric, or stay-in-place fiberglass forms. The bottom of the formwork should be fitted with a neoprene or other compressible seal. Formwork should be properly sealed to prevent mortar loss.

Jacketing. Formwork should be designed in accordance with ACI 347R, *Guide to Formwork for Concrete* (21). The bottom of the formwork should be fitted with a neoprene or other compressible seal.

- Curing

Positive steps should be taken to provide moist curing. Minimum requirements include keeping forms cool and moist and draping exposed areas with wet burlap. Formwork should be left in place for a minimum of seven days for repairs that are not moist-cured. Immediately after the removal of such formwork, a membrane-forming curing compound should be applied to maintain the internal humidity of the repair.

3.3.21 Quality Assurance/Construction Inspection: Superstructure and Substructure Encasement/Jacketing

All the items outlined under section 3.3.14 will be used, with the following additions:

1. The site inspector shall ensure that structural integrity is maintained throughout the repair.

2. The site inspector shall ensure that all formwork provides the specified minimum cover.

3.3.22 Material Performance Specifications: Superstructure and Substructure Encasement/Jacketing

Repair concrete for encasement and jacketing should be designed as outlined in section 3.3.15. Concrete to be placed under water should have a minimum cement content of 650 lb/yd^3 (385 kg/m^3) (30).

3.4 Rehabilitation Methods

Rehabilitation methods are used to restore a bridge component to an acceptable level of service and to correct the deficiency that caused the deterioration. For the case of corrosion, the mechanisms that resulted in the corrosion damage are the diffusion of the chloride ion to the reinforcing steel and the initiation of the corrosion process. Thus, rehabilitation methods that address the cause of the corrosion deterioration of existing concrete bridge components first require that all the sound concrete areas that are actively corroding or critically chloride-contaminated must be removed.

Actively corroding areas are those areas where the corrosion potential is more negative than 250 mV CSE. Critically contaminated areas are those areas where the chloride content at a reinforcing steel cover depth equal to the depth of 1% of the reinforcing steel is currently less than 1.0 lb/yd^3 (0.59 kg/m^3) and will remain below this concentration if further chloride ingress is prevented. Corrosion potentials are to be taken on no greater than 2 ft (61 cm) grid centers. Section 2.4.1.1 presents sample plan criteria for chloride content and cover depth. Section 3.2.1.1 presents the method for determining critically chloride-contaminated areas.

A physical rehabilitation method, such as low-permeability concrete, addresses the rate of chloride ion diffusion after the chloride-contaminated concrete is removed. A chemical rehabilitation method, such as the use of corrosion inhibitors, influences the chloride corrosion threshold contamination level after the chloride-contaminated concrete is removed.

The rehabilitation methods discussed below include deck overlays (LSDC, LMC, and HMAM) and substructure and superstructure patching and encasement/jacketing methods.

3.4.1 Deck Overlays

- Description

The rehabilitation of an existing corrosion-damaged deck consists of removing and patching all damaged (spalled, delaminated), sound but corroding, and sound but critically chloride-contaminated areas and overlaying the patched deck with LSDC, LMC, or HMAM.

- Limitations

Reduction in live load capacity due to the increase in dead load from the overlay needs to be considered. Also, the concrete removal economics must be considered. At some percentage of surface area removal, the price for removing all the concrete to a depth at least 0.75 in. (1.9 cm) below the top reinforcing steel layer will be less than that for leaving a portion of the concrete in place.

Chloride content of the concrete below the removal depth must also be determined. The chloride content of the concrete left in place must be low enough so as not to diffuse to and initiate corrosion of the lower reinforcing steel layer. Methodology presented in section 3.2.1 is to be used to assess this condition.

- Estimated Service Life

The service life of rehabilitation overlays is limited by the rate of diffusion of chloride ion through the LSDC and LMC and the leakage of chloride ions through the membrane of an HMAM. Thus the overlays are significantly influenced by environmental exposure conditions (chloride concentration and temperature). Table 3.9 presents the estimated service life for LSDC, LMC, and HMAM overlays in low, moderate, high, and severe chloride environments. The driving chloride diffusion concentration Co, is 2.0, 6.0, 9.0, and 12.5 lb/yd^3 (1.2, 3.6, 5.3, and 7.4 kg/m^3) for the low, moderate, high, and severe chloride exposure conditions, respectively (3).

- Estimated Price

The estimated price for constructing a rehabilitation LSDC, LMC, or HMAM overlay on a prepared surface would be the same as the price for constructing a repair LSDC, LMC, or HMAM overlay on a prepared surface (see section 3.3.5) plus the price for the additional concrete removal and replacement required for rehabilitation. Concrete removal methods are presented in chapter 5. However, for a full discussion on the economics of removing concrete from concrete bridge components, the reader is directed elsewhere (4). Patching prices are presented in section 3.3.1.

Table 3.9 Estimated Service Life, Years Environmental Exposure

Overlay Type	Low	Moderate	High	Severe
LSDC	> 100	35 - 70	30 - 60	25 - 50
LMC	> 90	20 - 35	15 - 30	15 - 25
HMAM	30	30	25	20

3.4.2 Construction Procedure: Deck Overlays

The construction procedures, quality assurance/construction inspection measures, and material performance specifications are identical to those presented in sections 3.3.5 through 3.3.11, for repair overlays. An economic analysis should be performed to determine the most effective removal technique as directed in SHRP-S-336, *Techniques for Concrete Removal and Bar Cleaning on Bridge Rehabilitation Projects* (4).

3.4.3 Superstructure and Substructure Patching/Encasement/Jacketing

- Description

These rehabilitation methods are the same as the patching and encasement/jacketing repair methods (see sections 3.3.12 and 3.3.19), except that, as with deck rehabilitation, the sound actively corroding and critically chloride-contaminated concrete areas are removed. In addition, the patched, encased, or jacketed concrete bridge element must be sealed or coated periodically.

- Limitations

The limitations of the patching and encasement/jacketing rehabilitation methods would be the same as for the repair techniques; see sections 3.3.12 and 3.3.19.

- Estimated Service Life

With periodic sealer or coating applications, the estimated service life of the patching, encasement, and jacketing rehabilitation methods is 50 years.

- Estimated Price

The price for the patching rehabilitation methods would be the same as for the repair methods, with the addition of the sealer or coating price. The price for the encasement and jacketing rehabilitation methods would be the same as for the repair methods except for the additional concrete removal and replacement price and the price of the sealer or coating.

3.4.4 Construction Procedure: Superstructure and Substructure Patching/Encasement/Jacketing

The construction procedures, quality assurance/construction inspection measures, and material performance specifications are identical to those presented in sections 3.3.12 through 3.3.22, for repair patching, encasement, and jacketing.

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4

Experimental Methods

4.1 Introduction

This chapter presents experimental repair and rehabilitation methods. The methods are considered experimental because they are currently being used as a standard method by only a few SHAs or on a limited basis by other SHA's (1) or they were developed (2) and field-validated (3) under "Concrete Bridge Protection and Rehabilitation: Chemical and Physical Techniques," the research project that generated this report.

Experimental methods being used by SHAs presented in this chapter include microsilica overlays and polymer impregnation. Experimental methods developed and field-validated under the research project and presented in this chapter include corrosion inhibitor overlays, corrosion inhibitor impregnation, and patching with corrosion inhibitors. For each method applied to decks or substructure and superstructure elements, a description, limitations, estimated service life, estimated price or cost, and construction procedures and specifications are presented.

Estimated service life is presented as a range for a given set of environmental exposure conditions. The service life range represents about 70% of the to-be-repaired and rehabilitation cases. Thus, 15% of the repair-rehabilitation cases will have a shorter service life and 15% will have a longer service life. The midpoint of the service life range represents the average service life and can be used for planning purposes. Because equal numbers of individual bridges will be above and below the average service life and thus will average out, the least-cost solutions will be selected. For individual bridge elements, use the methodologies presented in chapter 2 for estimating service lives.

Except for microsilica overlays, the methods presented in this chapter are used by SHAs on a very limited basis or were used only during the field validation trial (3). Thus an analysis of price data for the experimental methods could not be performed, because of the extremely limited availability of data. Therefore, method costs are presented. The costs were determined using standard engineering estimating methods. The presented costs are based on mid-1991 and national average costs. Complete details on price and cost estimates and service lives are presented elsewhere (4, 1).

4.2 Repair Methods

Repair methods do little to stop or significantly retard the corrosion of the reinforcing steel. Repair methods are used to restore a bridge component to an acceptable level of service. Only one repair method is presented in this chapter: microsilica concrete overlays for decks. Silica fume and condensed silca fume are terms also used for microsilica.

4.2.1 Decks: Microsilica Concrete Overlays

- Description

Used as a deck repair method, microsilica concrete (MSC) overlays restore the deteriorated riding surface to its original service condition. MSC is a low-permeability concrete typically containing 7 to 12% microsilica by weight of the cement. Because of the extreme fineness of microsilica, a high-range water reducer must be used to reduce the water and improve workability. The most common specified overlay thickness is 2 in. (5 cm).

The overlay has some influence on the service life of the repair, but service life is largely controlled by the amount and contamination level of the chloride-contaminated concrete left in place.

- Limitations

Overlays may increase the dead load and thus decrease the live load capacity of a bridge. Thus, before an MSC overlay is specified, the reduced live load capacity of the bridge must be compared with present and future needs. However, the presence of the microsilica in the overlay concrete may prevent the alkali-silica reaction (ASR) from occurring between the overlay concrete and an existing deck concrete that contains an ASR-susceptible aggregate. If the existing deck concrete contains an aggregate susceptible to the alkali-carbonate reaction, the reaction is not negated, and a low-alkali cement must be used in the MSC overlay.

- Estimated Service Life

Since the primary factor that influences the service life of a repair MSC overlay is the area of sound but actively corroding and critically chloride-contaminated concrete left in place, environmental chloride exposure conditions have little influence on the service life of the repair MSC overlay.

The service life of a repair MSC overlay is estimated to be 22 to 26 years.

- Estimated Price

The price includes all material, equipment, and labor associated with constructing an MSC overlay on a prepared surface. The price does not include patching and milling prices. Patching prices can be found in section 3.3.1. The economics of concrete removal are presented elsewhere (5).

The price of MSC overlays is influenced by job quantity and the expected number of bidders. The price may be determined from the following relationship:

$$y = 34.02 + (-8.43 \times 10^{-5})X + [(6253.16)/(X^{0.7567})]$$

where y = Predicted national adjusted price ($\$/yd^2$)

X = Job quantity (yd^2) times number of bidders

For a total job quantity of 1,800 yd^2 with 4 expected bidders, the price of an MSC overlay is $\$41/yd^2$.

4.2.1.1 Construction Procedure: Microsilica Concrete Overlays

Except as noted below, the construction procedures for the placement of microsilica concrete overlays are identical to those described in section 3.3.6, for portland cement-based overlays.

- Substrate Preparation

The screed rails shall be adjusted to provide a minimum overlay thickness of 2 in. (5.1 cm).

- Placement and Consolidation

The overlay concrete can be produced using a mobile mixer or ready-mixed concrete.

The bond grout should be prepared and applied as described for LSDC overlays (section 3.3.6).

The overlay surface should be smoothed with a fresno trowel following consolidation by the finishing machine.

Fog spray should be used during finishing to prevent plastic shrinkage cracking.

- Curing

Within 30 minutes of placement, the fresh concrete should be covered with a single layer of clean, damp burlap. The edges of the burlap should be overlapped 4 in. (10 cm). The burlap should be covered with a layer of white polyethylene sheeting. The burlap should be kept constantly moist for 7 days; however, 72 hours of moist curing may be acceptable if traffic conditions warrant a more rapid repair. A high-pressure misting system can be substituted for the moist burlap.

4.2.1.2 Quality Assurance/Construction Inspection: Microsilica Concrete Overlays

Except as noted below, all the procedures used in section 3.3.7 for portland cement-based overlays shall apply.

- Quality Assurance

The inspector should obtain a certificate of compliance with AASHTO M 307 for each batch of microsilica used.

4.2.1.3 Material Performance Specifications: Microsilica Concrete Overlays

The following materials and proportions are recommended (6):

1. Microsilica admixture in densified powder or slurry form conforming to AASHTO M 307, 7.5% to 9.0% by weight of cement
2. Cement type and content, ASTM C 150 type I or II, 611 lb/yd^3 (362 kg/m^3)
3. Maximum water/ccement ratio 0.40 (Note that if microsilica is used in a slurry form, the water in the slurry must be included in the water/cement ratio calculation.)
4. Aggregates, ASTM C 33, with a maximum aggregate size of 0.50 in. (1.3 cm).
5. Total air content 6% to 7% (Note that the use of microsilica may require an increase in the dosage of air-entrainment agent to produce the desired air content.)
6. Slump, 5 to 6 in. (13 to 15 cm) when tested 5 minutes after discharge

A high-range water-reducing admixture (superplasticizer) should be added to improve workability.

4.3 Rehabilitation Methods

4.3.1 Decks: Microsilica Concrete Overlays

- Description

A microsilica rehabilitation overlay involves not only removing the damaged concrete, but also removing the active corrosion (more negative than 250 mV CSE) and the critically chloride- contaminated concrete. Corrosion potential and critically chloride-contaminated concrete sampling plan criteria and application methodologies are presented in sections 2.4.1.1 and 3.2.1.1.

All spalled, delaminated, actively corroding, and critically chloride-contaminated concrete is removed. The exposed rebar is cleaned to near white metal and patched with a concrete with properties similar to those of the original concrete. The entire deck is then overlaid with a 2 in. (5 cm) MSC overlay (7% to 12% microsilica by weight of the cement).

- Limitations

Reduction in live load capacity, chloride content of the concrete left in place below the top reinforcing steel layer, and the amount of concrete that has to be removed must be considered. The concrete left in place must not be critically contaminated, or the chlorides will diffuse to the top or bottom reinforcing steel layer and initiate the corrosion of the bottom reinforcing steel layer.

The amount of surface concrete to be left in place above the top reinforcing steel layer versus the entire deck surface area must be considered. The price to remove all the concrete to a depth of at least 0.75 in. (1.9 cm) below the top layer of reinforcing steel may be less than that of leaving small amounts of the original cover concrete in place.

Chapter 6 of this manual is a concrete removal primer; the economic considerations of concrete removal are presented elsewhere (5).

- Estimated Service Life

The estimated service life of an MSC rehabilitation overlay depends on the thickness of the overlay, the rate of chloride diffusion through the MSC and any original concrete left in place, the cover depth of the original left-in-place concrete, and the environmental exposure conditions (chloride concentration and temperature).

The estimated service life of a 2 in. (5 cm) MSC concrete overlay in a low (Co = 2.0 lb/yd^3 [1.19 kg/m^3]), moderate (Co = 6.0 lb/yd^3 [3.56 kg/m^3]), high (Co = 9.0 lb/yd^3 [5.34 kg/m^3]), and severe (Co = 12.5 lb/yd^3 [7.42 kg/m^3]) chloride environment is greater than

100 yeras, 35 to 70 years, 30 to 60 years, and 25 to 50 years, respectively.

- Estimated Price

The estimated price of a rehabilitation MSC overlay includes all material, equipment, and labor required to place the overlay on a prepared surface. Since the price does not include the work required to prepare the surface, such as concrete removal, bar cleaning, and patching, the price for a rehabilitation MSC overlay is the same as for a repair MSC overlay (see section 4.2.1).

4.3.1.1 Construction Procedure: Microsilica Concrete Overlays

The construction procedures, quality assurance/construction inspection measures, and material performance specifications for microsilica concrete overlays used in deck rehabilitation are identical to those described in sections 4.2.1.1 through 4.2.1.3 for microsilica concrete repair overlays. An economic analysis of the concrete removal phase should be performed by the procedures outlined in SHRP-S-336, *Techniques for Concrete Removal and Bar Cleaning on Bridge Rehabilitation Projects* (5).

4.3.2 Decks: Corrosion Inhibitor Overlays

- Description

Corrosion inhibitor overlay methods were developed to limit the amount of sound chloride-contaminated concrete to be removed. In particular, all the sound chloride-contaminated concrete surrounding the top layer of reinforcing steel is to remain in place.

First, the cover concrete is dry milled. To limit the probability of the milling machine hitting a rebar to less than 1 chance in 100, the milling depth is to be determined from the following relationship:

$$d_m = d - \alpha\sigma_{n-1}$$

where d_m = maximum milling depth
d = average cover depth
σ_{n-1} = cover depth standard deviation
α = 2.58

For methods and procedures for determining the average cover depth d and cover depth standard deviation σ_{n-1}, see section 2.4.1.1. If the average cover depth is 2.32 in. (5.89 cm) and cover depth standard deviation is 0.32 in. (0.81 cm), then dry-mill to a depth of 1.50 in. (3.80 cm).

Second, remove all damaged concrete, clean any exposed steel and replace any damaged reinforcing steel as necessary, patch with a corrosion-inhibitor-modified concrete, and shotblast the entire deck surface.

Third, apply three spray applications of the corrosion inhibitor, and overlay with a corrosion-inhibitor-modified MSC, LSDC, or LMC overlay.

The spray-on corrosion inhibitor may be Postrite, Cortec MCI 2020, or Alox 901 (see section 4.3.2.3 for details). If the spray-on corrosion inhibitor is Postrite, the patch and overlay concrete is to be a DCI corrosion-inhibitor-modified concrete.

If the spray-on corrosion inhibitor is Cortec MCI 2020, the patch and overlay concrete is to be a Cortec MCI 2000 corrosion-inhibitor-modified concrete. If the spray-on corrosion inhibitor is Alox 901, the patch and overlay concrete *shall not* to be modified with a corrosion inhibitor.

An alternative method dries the concrete with propane-fired infrared heaters after all the damaged concrete is removed but before the corrosion inhibitor patch concrete is placed (see section 4.3.2.1). The drying phase is used to improve the spray-on corrosion inhibitor uptake characteristic of the milled concrete surface.

- Limitations

In most cases, the thickness of the 2 in. (5 cm) overlay will be greater than the milling depth. Thus, the reduction in live load capacity must be checked against present and future requirements.

The spray-on corrosion inhibitors Cortec MCI 2020 and Alox 901 leave a surface residue that significantly reduces the bond between the old concrete and the overlay concrete. Thus, when Cortec MCI 2020 and Alox 901 spray-on inhibitors are used, the entire deck surface must be lightly shotblasted to remove the bond-reducing residue on the spray-treated areas and the patched areas because the residue will be tracked onto these areas.

- Estimated Service Life

During the process, all the damaged (previously patched, spalled, and delaminated) and thus highly corroded areas are patched with a corrosion inhibiting concrete, except in the Alox 901 treatment. Also, the highly chloride-contaminated concrete is removed by dry milling (see section 4.3.2), and this area is treated with a spray-on corrosion inhibitor. The spray-on inhibitor will diffuse to the reinforcing steel and retard the corrosion of the steel in a short time. The corrosion inhibitor in the overlay concrete will diffuse to the reinforcing steel and provide a maintenance level of corrosion inhibitor throughout the life of the rehabilitated deck.

Additional chlorides from future winter maintenance or exposure to seawater must diffuse through the low-permeability overlay. Thus the service life of the corrosion inhibitor deck rehabilitation methods is influenced by the amount of inhibitor at the reinforcing steel, the environmental exposure conditions (chloride concentration and temperature), and the type of overlay. Table 4.1 presents the estimated service life of the three corrosion inhibitor rehabilitation methods. Little service life is to be gained by drying the concrete before the inhibitor is sprayed on. However, drying reduces the time required to retard the corrosion (see section 4.3.2.1) and thus should be used on decks with sound but corroding areas.

Table 4.1 Corrosion Inhibitor Estimated Service Life (years)

Treatment/Overlay	Environmental Exposure			
Type	Low	Moderate	High	Severe
Postrite, DCI				
LSDC	>100	70 - 90	65 - 85	60 - 80
LMC	>100	45 - 65	40 - 50	35 - 45
MSC	>100	70 - 90	65 - 85	60 - 80
Cortec MCI 2020/2000				
LSDC	>100	70 - 90	65 - 85	60 - 80
LMC	>100	45 - 65	40 - 50	35 - 45
MSC	>100	70 - 90	65 - 85	60 - 80
Alox 901				
LSDC	>100	60 - 80	55 - 65	50 - 60
LMC	80 - 90	40 - 60	35 - 45	30 - 40
MSC	>100	60 - 80	55 - 65	50 - 60

- Estimated Cost

The cost of the spray-on corrosion inhibitor deck rehabilitation methods was determined using standard engineering estimating techniques. Thus, the presented *costs* may not include some items that are included in the *price* estimate. The estimated costs are national average, mid-1991 costs.

The presented costs include removal of 1.5 in. (3.81 cm) of concrete, patching of deteriorated areas with corrosion inhibitor-modified concrete, shotblasting the entire deck surface, spraying on inhibitor applications, overlaying with a corrosion-inhibitor-modified LMC, LSDC, or MSC, maintenance and protection of traffic (MPT), and contractor overhead and profit (O&P). Note that the spray-on corrosion inhibitor rehabilitation methods costs include procedures for preparing the surface for the overlay, such as patching and shotblasting, whereas the rehabilitation and repair LMC, LSDC, and MSC overlay *prices* are for placing the overlays on a prepared surface. Thus the LMC, LSDC, and MSC overlay prices do not include surface preparation prices such as patching and shotblasting. Consequently, the presented values cannot be used to compare the economics of the two methods because price is not the same as cost, and because one includes work items not included in the other.

In addition to being influenced by material costs (inhibitor type) and percentage of damaged area to be patched, the costs of the corrosion inhibitor rehabilitation methods are influenced by the number of contracts performed by a contractor in a year. Tables 4.2 and 4.3 present the estimated corrosion inhibitor rehabilitation methods costs for a typical bridge deck of 8,800 ft^2 (817 m^2) (44 ft x 200 ft)[(13.4 m x 61.0 m)]. The cost-estimating details for the deck corrosion inhibitor rehabilitation methods are presented elsewhere (4).

Table 4.2 Bridge Deck Spray-On Corrosion Inhibitor Overlay Rehabilitation System, Non-dried ($\$/yd^2$)

Inhibitor Spray-On Overlay	Percent Deck Damage	Decks/Year/Contractor		
		1	4	10
Postrite/DCI	0	62	47	44
	5	71	55	52
	10	79	63	60
	20	96	80	77
Cortec MCI 2020/2000	0	60	44	41
	5	68	52	49
	10	76	60	57
	20	92	77	74
Alox 901	0	66	50	47
	5	74	58	55
	10	82	66	63
	20	98	83	80

$\$1/yd^2 = \$1.196/m^2$

Table 4.3 Bridge Deck Spray-On Corrosion Inhibitor Overlay Rehabilitation System, Dried ($/yd²)

Inhibitor Spray-On Overlay	Percent Deck Damage	Deck/Year/Contractor		
		1	4	10
Postrite/DCI	0	105	81	76
	5	110	86	81
	10	114	90	85
	20	119	95	90
Cortec MCI 2020/2000	0	102	78	74
	5	107	83	79
	10	111	87	82
	20	116	92	88
Alox 901	0	111	87	82
	5	116	92	87
	10	120	96	91
	20	125	101	96

$\$1/yd^2 = \$1.196/m^2$

4.3.2.1 Construction Procedure: Corrosion Inhibitor Overlays

The following describes the construction procedures, quality assurance, construction inspection, and material performance specifications for the application of inhibitor-modified concrete overlays. A corrosion inhibitor will be spray-applied to the substrate before the application of the overlay. The substrate can be dried to improve the absorption of the inhibitor.

- Scarification and Removal of Unsound Concrete

The entire deck surface to be overlaid should be scarified to the depth of the reinforcing steel as determined in section 4.3.2. If steel reinforcement is encountered, the scarification depth at such locations shall be to the top of the reinforcing bars. Hydrodemolition equipment may not be used for scarification unless the substrate is to be dried.

Following scarification, the site engineer should sound the deck using a drag chain and hammer to delineate the areas of unsound concrete. Bituminous patches and areas of unsound concrete should be removed and patched as outlined in section 3.3.2. The concrete used to backfill the repair cavities should be modified with the inhibitor to be used for the overlay as described in section 4.3.2.3. The patched areas should be textured to improve the overlay bond. The repaired areas should be allowed to cure for a minimum of 48 hours before placing the overlay. The following should be completed before the application of the surface-applied inhibitors:

1. Dormant (nonworking or moving) visible cracks should be sealed using epoxy injection.

2. Following scarification, the deck should be airblasted to remove dust and debris. The air compressor used for airblasting should have oil and water traps installed.

3. Water must not be allowed to contact the deck after the cover concrete is removed. Should water wet the scarified area before the application of the corrosion inhibitors, the concrete surface shall be dried for 30 minutes with a propane-fired infrared heater to a surface temperature of 400°F (204°C). Spray application of the inhibitors should be delayed until the concrete has returned to ambient temperature.

- Drying

To promote the absorption of the surface-applied corrosion inhibitor, the substrate can be dried to remove the absorbed water in the concrete below the level of the reinforcing steel. Note that patching of unsound concrete should not commence until the completion of drying. Exposed reinforcement should be protected with thin metal sheets to prevent damage to the concrete from excessive heating. The specifications for drying are adapted in part from FHWA-PA-85-014, *Deep Polymer Impregnation of a Bridge Deck Using the Grooving Technique* (7). The following steps should be followed in drying the substrate:

1. *Expansion*. Before beginning the drying operation, expansion joints must be cleaned and joint sealer removed. The joints must be monitored and the heating or drying rate must be reduced if complete closure of a joint becomes imminent. It shall be the responsibility of the contractor to reseal the expansion joints at the completion of the job.

2. *Drying method*. Gas-fired radiant infrared heaters shall be used to dry the concrete. The heaters shall be sufficient in number and size to cover the entire width of the deck for a distance of at least 4 ft (1.2 m), but no more than 20 ft (6.1 m), in the longitudinal direction for each heater setup. The heater units should be mounted on steel casters to permit easy movement from one setup to the next. The heating capacity of the heaters shall be sufficient to provide the temperatures specified in table 4.4.

   The surface temperature shall be monitored under each heater unit with welded, pad-type, copper-constantan, parallel, grounded thermocouple probes (quick-disconnect type) having an Inconel sheath long enough to extend from the center of the heater to outside the heated area. A heat shield of galvanized sheet metal shall be erected around the perimeter of the heaters extending from the deck surface to the height of the heaters. Glass wool insulation (R-19) shall be placed on the deck over a perimeter area 24 in. (61 cm) wide around the heater group for each heater setup to reduce thermal gradients.

Table 4.4 Schedule of Surface Temperatures

Time	Surface Temperature (°F)	Surface Temperature (°C)
Start	Ambient	Ambient
15 minutes	350 ± 25	177 ± 14
Until dry	350 ± 25	177 ± 14

3. *Condition-concrete dry*. Drying is considered complete when the temperature at a depth of 0.5 in. (1.3 cm) below the average depth of reinforcing steel reaches 180°F (82°C) as measured by unsheathed, copper-constantan thermocouples having teflon conductor insulation and glass braid overall insulation. The thermocouples are to have welded hot junctions and are to be set in holes 0.25 in. (0.6 cm) drilled in the bottom of the deck. The holes shall be backfilled with a fast-set epoxy gel suitable for work on overhead surfaces. A minimum of three embedded thermocouples will be used for each heater set up: one at the bridge centerline and one in the center of each lane. The thermocouples shall remain in place after drying is completed, to monitor cool-down.

4. *Cool-down*. Immediately upon attainment of the drying criterion, the heaters will be moved and the dried area will be covered with R-19 insulation (to minimize thermal cracking by preventing too rapid cool-down of the surface). Heating of the next adjacent area shall immediately follow the placement of the insulation. The insulation shall remain in place until the surface temperature has dropped below 100°F (38°C). The contractor shall make provision for preventing wind from blowing the insulation off the deck.

5. *Weather protection*. Dried areas shall be protected from precipitation, runoff, and other sources of moisture before the application of the corrosion inhibitor. Any dried areas subjected to moisture before treatment must be redried at the direction of the inspector. Dikes constructed of compacted asphaltic concrete cold mix sealed with asphalt emulsion and placed in the roadway upgrade from the bridge deck have been found to be effective in diverting runoff.

6. *Drying time*. The approximate heating time at each heater setup is expected to be 40 minutes according to the surface temperature schedule required in table 4.4. The drying of the entire area must be carried out as a continuous operation until completed.

- Application of Surface-Applied Corrosion Inhibitor

Application of the surface-applied inhibitor involves spraying the dry concrete with a liquid corrosion inhibitor. The inhibitor can be successfully applied with an industrial-grade garden sprayer. Three applications of the inhibitor are to be applied and allowed to soak into the base concrete before overlay. The inhibitors that can be applied, and their corresponding concentration and application rates, are as follows:

1. *Postrite* is a water-based inhibitor containing 15% by weight calcium nitrite to be used as a surface treatment. Each application of the Postrite solution is to be applied at the rate of 150 ft^2/gal (3.7 m^2/l). A total of three applications of the inhibitor are to be applied, the second at one hour and the third at eight hours after the initial application.

2. *Cortec MCI 2020* is a water-based inhibitor to be used as a surface treatment or by injection. Each application of Cortec MCI 2020 shall be spray-applied at the rate of 225 ft^2/gal (5.5 m^2/L). A total of three applications of the inhibitor are to be applied, the second at two hours and the third at 12 hours after the initial application.

3. *Alox 901* is an oxygenated hydrocarbon to be used as a surface treatment. The inhibitor is applied at a concentration of 4.7% by weight Alox 901 in denatured ethyl alcohol. The inhibitor must be field-mixed at the site. Each application of Alox 901 shall be spray-applied at the rate of 70 ft^2/gal (1.7 m^2/l). Three applications of the inhibitor are to be applied, the second at one hour, and the third at four hours after the initial application.

Safety and handling measures for these inhibitors are discussed in section 4.3.2.3.

- Substrate Preparation

Following the application of the surface-applied corrosion inhibitor, the deck surface to be overlaid should be lightly sandblasted or shotblasted to remove laitances and any residues, followed by an airblast to remove dust and debris. The air compressor used for sandblasting and airblasting should have oil and water traps installed.

Blocking should be placed to provide for expansion joints at the same locations as existing expansion joints.

Screed rails should be installed so the finishing machine will provide the minimum overlay thickness of 2 in. (5 cm). The supports for the rails should be adjustable; shims should not be used to adjust rail heights. A dry run should be performed to ensure that the finishing machine provides the minimum overlay thickness.

- Placement and Consolidation

Latex concrete with inhibitor should be mixed at the site using a mobile mixer. Before placement, a trial batch should be prepared in the presence of the site engineer to test the calibration of the mixer. Sufficient mobile mixers should be present on the site to allow the continuous placement of the overlay without delay. Other overlay concretes containing DCI or Cortec MCI 2000 can be mixed with a mobile mixer or be ready-mixed concrete. However, overlay concrete containing DCI mixed at a concrete batch plant will require the addition of a suitable retarder.

A bonding grout should be thoroughly brushed into the substrate immediately in front of the finishing machine. The grout can be obtained by brushing the coarse aggregate out of a portion of the overlay concrete. The excess coarse aggregate should be discarded. Care must be taken to prevent the bond grout from drying before the placement of the overlay concrete. If drying is allowed to occur, a temporary construction dam should be placed, and the dried bonding grout should be removed by sandblasting.

The overlay concrete should be placed approximately 0.25 in. (0.6 cm) above final grade.

The overlay concrete should be consolidated and finished using a finishing machine. The self-propelled finishing machine should be equipped with screw augers, vibratory plates, and one or more rotating steel cylinders. The machine should be cable of spanning the entire placement transversely. Spud vibrators and hand tools should be used as necessary along the screed rails and for any deep pockets. A burlap or carpet drag can be used to texture the surface.

A lightweight catwalk should follow the finishing machine to allow hand finishing, inspection for levelness, and placement of curing materials.

- Curing

Within 30 minutes of placement, the fresh concrete should be covered with a single layer of clean, damp burlap. The edges of the burlap should be overlapped 4 in. (10 cm). The burlap should be covered with a layer of white polyethylene sheeting. In addition, DCI overlays should be continually misted for the first 24 hours of curing before the application of the polyethylene sheeting. Curing shall be maintained for the minimum times shown in table 4.5. Curing hours should be defined as hours for which the temperature is greater than 45°F (7°C). The overlay concrete should be protected with insulated blankets if the temperature is predicted to drop below 45°F (7°C).

Table 4.5 Minimum Curing Times

Overlay Type	Minimum Curing Time (hours)
LMC	48
DCI	168
Cortec MCI 2000	72

Curing for 72 hours may be acceptable if the site conditions warrant.

- Skid Resistance

Following 14 days of curing, grooves shall be sawed transverse to the center line. The grooves should be approximately 0.2 in. (0.5 cm) deep and 0.13 in. (0.3 cm) wide, spaced 0.75 in. (1.9 cm) on center. Grooves should terminate within 12 in. (30 cm) of the parapet or curb (7).

4.3.2.2 Quality Assurance/Construction Inspection: Corrosion Inhibitor Overlays

The following should be considered the minimum inspection requirements to ensure a quality overlay. More than one inspector should be present when the overlay is being placed.

- Scarification and Removal of Unsound Concrete

Before scarification, the engineer should conduct a cover depth survey to determine the depth of removal. The average cover depth and standard deviation shall be determined from a minimum of 40 measurements. Inspection for delineation and repair of unsound areas should be conducted as described in section 3.3.3. The inspector should ensure that no water is allowed to contact the deck after scarification.

- Drying

If drying is specified, the site inspector must carefully monitor the deck joints during drying to prevent closure. The inspector is also responsible for monitoring thermal differentials to ensure that adequate drying occurs while preventing differential thermal cracking. A log should be kept to track the internal and surface temperatures of the deck during drying. The inspector should ensure that all safety precautions are strictly followed.

- Application of Surface-Applied Corrosion Inhibitors

The site inspector should monitor and enforce the following:

1. The application rate and concentration of the inhibitor: The inspector should ensure that the inhibitor is applied until the surface appears saturated.

2. If water is allowed to contact the treated area after the initiation of the inhibitor applications, the affected area should be dried and the application process repeated.

3. Safety and environmental contamination regulations applying to the storage and handling of the inhibitors should be carefully enforced.

- Substrate Preparation

In assessing deck preparation, the inspector should examine the following:

1. Ensure that the deck is free from all laitances, organic residues, inhibitor residues, and debris that may be detrimental to the overlay bond.

2. Ensure that the screed rails are properly adjusted to provide the minimum overlay thickness over the entire deck area to be overlaid, and that the finishing machine is clean and functioning properly.

- Placement and Consolidation

The following should be accounted for by the site inspector during each placement:

1. Before placement, a trial batch should be prepared to ensure that the specified slump and air content can be obtained using the job materials and mixture. For LMC overlays the inspector should obtain certification of compliance with FHWA Notice 5140.15, July 1978, for each batch of latex delivered to the job site. The solids content of each batch of latex should be determined by ASTM D 1417.

2. Ensure proper proportioning of ingredients by calibrating the mobile mixer. Yield should be checked once a day.

3. Ensure that sufficient mobile mixers are present to allow continuous placement. Slump test (ASTM C 143), total air-content measurements (ASTM C 231), and a minimum of six compressive-strength cylinders should be prepared from each 20 yd^3 (15.3 m^3). Note that slump tests should be performed five minutes after mixing.

4. Monitor the air temperature, concrete temperature (ASTM C 1064), relative humidity, and wind velocity. Overlays shall not be placed under the following conditions:

- Air temperature is below 50°F (10°C);

- Evaporation rate exceeds 0.10 lb/ft^2/hr (0.49 kg/m^2/hr) as determined by ACI 305R.

Overlays shall be placed at night when the air temperature is above 85°F (29°C).

5. Monitor the application of the bond grout, and ensure that the overlay concrete is placed before it is allowed to dry.

6. Check to see that the levelness of the finished overlay meets requirements using a 10 ft (3 m) straight edge.

- Curing

The site inspector should monitor the overlay during the curing period to ensure that it is maintained at a temperature above of 45°F (7°C) and uniformly saturated; curing membranes are not acceptable.

Following the curing period, the entire overlay should be sounded with a drag chain and hammer to ensure bond. The surface of the overlay should be inspected for cracking. All unsound areas should be repaired by the contractor. After 7 days of curing, bond strength tests should be performed in accordance with ACI 503R. A minimum of three tests should be run. The minimum acceptable bond strength should be 200 psi (1.4 MPa). If any of the cores fail to meet the minimum requirements, a sampling program should be developed to determine the areas that will require replacement.

4.3.2.3 Material Performance Specifications: Corrosion Inhibitor Overlays

- *Postrite/DCI.*

If Postrite is used as the spray-on inhibitor, DCI (30% calcium nitrite) should be added to the repair and overlay concrete. The rate of addition should be 6 gal/yd^3 (30 L/m^3). DCI acts as a set accelerator and requires the addition of a high-range water reducer and initial set retarder. The addition rates provided below should be used as guidelines. Trial batches should be prepared with job materials to ensure proper air content, workability, and set time.

The inhibitors must be stored, handled, and applied in such a manner as to prevent injurious exposure to personnel associated with the job and to the public. Personnel working with the inhibitor shall be provided with and use safety glasses or goggles and rubber or other impervious gloves. Additional protective clothing should be worn to minimize skin contact. Spills should be absorbed with an inert, noncombustible medium and removed for disposal in accordance with existing federal, state, and local environmental regulations. Spills should be prevented from entering drinking water supplies and streams or groundwater.

The following materials, proportions and properties are recommended for the overlay and repair concrete (3):

1. DCI should be added as an admixture at the rate of 6 gal/yd^3 (30 l/m^3). Note that the water in the admixture must be accounted for in the water/cement ratio. DCI contains approximately 7.0 lb water per gallon (0.84 kg/L).

2. Darasen 100 or other approved ASTM C 494 type G high-range water reducer should be added at a rate of 12 oz/100 lbs (750 g/100 kg) of cement.

3. Daratard-17 or other approved ASTM C 494 type B & D initial set retarder should be added at the rate of 6 oz/100 lb (375 g/100 kg) of cement.

4. Cement content, ASTM C 150 type I or II, 658 lb/yd^3 (390 kg/m^3)

5. Maximum water/cement ratio 0.42

6. Aggregates, ASTM C 33, with maximum aggregate size of 0.5 in. (1.3 cm)

7. Total air content, 7.5% ± 1%

8. Slump, range 2 to 4 in. (5 to 10 cm)

Postrite and DCI can be obtained from

Construction Products Division
W.R. Grace and Company
62 Whittemore Avenue
Cambridge, MA 02140
(617) 876-1400

- *Cortec MCI 2020/2000.*

If MCI 2020 is used as the spray inhibitor, MCI 2000 should be added to the repair and overlay concrete. The rate of addition should be 2 lb/yd^3 (1.2 kg/m^3). A high-range water reducer should be added to improve workability. Research has shown that MCI 2000 has no effect on the properties of fresh concrete (2). However, trial batches should be prepared with job materials to ensure air content, workability, and set time.

The inhibitor should be stored in original shipping containers. Containers should be kept tightly closed and kept away from heat, open flame, and spark sources. Personnel working with the inhibitor shall be provided with and use safety glasses or goggles, approved respirators, and chemical-resistant rubber or plastic gloves. The contractor shall provide a field eyewash and safety shower to be used in the event of an accidental splash of inhibitor on the workers. Additionally, workers shall be required to thoroughly wash hands with soap and water before eating, smoking, drinking, or using the lavatory. Accidental spills should be absorbed with a sweeping compound or other absorbent material. The compound shall be disposed of according to existing federal, state and local environmental regulations.

The following materials, proportions, and properties are recommended for the overlay and repair concrete:

1. Cortec MCI 2000 should be added at the rate of 2 lb/yd^3 (1.2 kg/m^3)
2. An approved ASTM C 494 type G high-range water reducer should be added at a rate of 12 oz/100 lb (750 g/100 kg) of cement to improve workability.
3. Cement content, ASTM C 150 type I or II, 658 lb/yd^3 (390 kg/m^3)
4. Maximum water/cement ratio 0.42
5. Aggregates, ASTM C 33, with maximum aggregate size of 0.5 in. (1.3 cm)
6. Plastic air content, 7.5% ± 1%
7. Slump, range 2 to 4 in. (5 to 10 cm)

Cortec MCI 2000/2020 can be obtained from

Cortec Corporation
St. Paul, MN 55107
(612) 224-5643.

- *Alox 901.*

If Alox 901 is used as the spray-on inhibitor, LMC should be used for repair and overlay. Alox 901 is an oxygenated hydrocarbon and is therefore unsuitable for use as an admixture. The LMC should be proportioned, mixed, and placed in accordance with the specifications presented in section 3.3.6.

The inhibitor must be stored, handled, and applied in such a manner as to prevent fire, explosion, and injurious exposure to personnel associated with the job and to the public. The Alox 901 and denatured alcohol shall be stored in original shipping containers before application. Maximum alcohol storage temperature shall not exceed 90°F (32°C). The storage facility shall be constructed to provide protection from direct sunlight, fire hazard, and oxidizing chemicals. Sufficient ventilation shall be maintained in the storage facility to prevent the hazardous buildup of alcohol vapor concentrations. "Warning" and "No Smoking" signs shall be placed at appropriate intervals. Dry chemical fire extinguishers meeting Underwriters Laboratories, Inc., approval for class A, B, and C fires shall be provided.

Personnel working with the inhibitor shall be provided with and use safety eyeglasses or goggles and oil-impervious gloves.

Accidental spills of the inhibitor must be prevented from entering ground streams and other waterways. Attempt to recover and recycle accidental spills; nonrecyclable spills should be absorbed with vermiculite or dry sawdust, cleaned up with nonsparking tools, and disposed of by methods complying with government regulations.

Alox 901 can be obtained from

> Alox Corporation
> 3943 Buffalo Avenue
> P.O. Box 517
> Niagara Falls, NY 14302
> (716) 282-1295

4.3.3 Decks: Polymer Impregnation

- Description

Corrosion-damaged areas (spalls and delaminations) are patched with a PCC similar to the existing concrete. Grooves are cut into the deck on lines of equal contour. The grooves are 0.75 in. (1.9 cm) wide, 1.5 in. (3.8 cm) deep, and 3.0 in. (7.6 cm) on center. The concrete is dried to a depth of 0.5 in (1.3 cm) below the top reinforcing steel layer using propane-fired infrared heaters. The concrete is allowed to cool slowly under an insulating mat to ambient temperature. The monomer (methyl methacrylate) is poured into the grooves and allowed to soak into the concrete. Heat is applied to polymerize the monomer insitu. The grooves are backfilled with a latex-modified mortar (LMM) by pouring the LMM on the deck and squeezing it into the grooves.

- Limitations

Heavily worn decks may need a thin polymer overlay in the wheel path areas to restore the riding surface to prevent hydroplaning.

- Estimated Service Life

Polymer-impregnated concrete is a very dense, hard, low-permeability concrete. The primary action of the polymer, an electrically nonconducting material, is to replace the continuous concrete pore water (which acts as the corrosion battery electrolyte) and thereby *stop* the corrosion process. Thus the service life of deep polymer impregnation as a corrosion rehabilitation method is not influenced by the chloride environmental exposure conditions. Field installations have demonstrated that the service life of deep polymer impregnation is at least 33 years (2). The estimated service life of deep polymer impregnation is at least 50 years.

- Estimated Cost

The estimated deep polymer impregnation costs were determined by standard engineering estimating procedures. Details are presented elsewhere (4). The costs are national average costs for mid-1991.

The presented costs include all costs associated with the construction of a deep polymer impregnation of a typical bridge deck, including patching, grooving, drying, impregnation, polymerizing, backfilling the grooves, MPT, and contractor O&P. Table 4.6 presents the costs for four levels of deck damage and three levels of number of jobs per year for a contractor.

Table 4.6 Cost Estimates of Bridge Deck Deep Polymer Impregnation Rehabilitation Method Using the Grooving Technique ($\$/yd^2$)

Impregnant	Percent Deck Damage	Decks/Year/Contractor		
		1	4	10
Methyl Methacrylate	0	196	157	150
	5	204	165	157
	10	211	173	165
	20	226	188	180

$\$1/yd^2 = \$1.196/m^2$

4.3.3.1 Construction Procedure: Polymer Impregnation

The following describes the construction procedures, quality assurance, construction inspection, and material performance specifications for the deep polymer impregnation of bridge decks (7).

- Preliminary Test Work

The following must be completed in order to develop a plan for the grooving process:

1. The mean and standard deviation of the reinforcing steel cover depth should be determined from a minimum of 40 pachometer measurements per 8,000 ft^2 (743 m^2) of surface area or portion thereof.

2. A precise level survey of the deck surface must be made, taking readings to the nearest 0.001 ft (0.03 cm). The level survey should be conducted on a 10 ft (3 m) grid. Readings should also be taken at breaks in slope. The level survey will be used to determine the orientation of the lines of equal elevation.

3. A minimum of three cores 4 in. (10 cm) in diameter should be taken per 8,000 ft^2 (743 m^2) of surface area or portion thereof. The cores, approximately 4 in. (10 cm) long, should be obtained from random locations on the deck. The cores will be used to determine the required impregnation time. The cores should be oven-dried at 230°F ± 5°F (110°C ± 3°C) to a constant weight, the sides sealed with epoxy, and a ponding dam attached to the top surface. The monomer mixture (see section 4.3.3.3) is ponded on the surface for 24 hours, after which time the cores are to be polymerized by immersion in 160°F (71°C) water for 4 hours. The cores are then to be cut in half and the depth of impregnation determined by etching with muriatic (hydrochloric) acid. The average depth of penetration is plotted against the square root of impregnation time ($\sqrt{24}$). By drawing a straight line from the origin to this point, the field impregnation time can be estimated by entering the depth axis at 1.1 D (depth of impregnation) and intersecting the straight line at the point representing the impregnation time.

The required depth of impregnation and the depth, width, and spacing of the grooves are all functions of the reinforcement cover, distribution, and bar size. The complete methodology for optimizing the interrelationship among these variables is presented in appendix A of NCHRP Report 257, *Long-Term Rehabilitation of Salt-Contaminated Bridge Decks* (8). In summary, they are as follows:

Depth of Impregnation:

$$D = C + R_1 + R_2 + 0.5$$

Groove Depth:

$$d = C - 0.5$$

Groove Width:

$$w = \frac{D(D-d)}{(11d-D)}$$

Groove Spacing:

$$s = w + D - d$$

where: D = depth of impregnation (in.)
R_1 = diameter of main reinforcing steel in top mat (in.)
R_2 = diameter of temperature steel in top mat (in.)
C = average reinforcing steel cover (in.)
d = groove depth (in.)
w = groove width (in.)
s = groove spacing (in.)

- Removal and Repair of Unsound Concrete

At least one month before the impregnation process commences, the site engineer should sound the deck using a drag chain and hammer to delineate the areas of unsound concrete. Bituminous patches and areas of unsound concrete should be removed and patched as outlined in section 3.3.2. The patch material should be PCC. The repaired areas should be allowed to cure for a *minimum* of 28 days before impregnation. The deck may be opened to traffic as soon as the patch portland cement has reached sufficient strength.

- Grooving

Grooves shall be cut in the surface of the concrete that is to be impregnated. The grooves significantly reduce the impregnation time and contain the monomer during the impregnation period and therefore are aligned along lines of constant elevation. The grooves also allow the monomer to bypass surface contamination, improving the rate of impregnation.

The grooves may be cut along lines of equal elevation using a water-cooled diamond concrete saw. String lines should be provided to indicate the orientation of the grooves. The grooves should be cut to the depth, width, and spacing determined in the preliminary test work. The grooves should be terminated 12 in.(30 cm) from the face of the parapet or curb.

The following tolerances should be considered acceptable:

Groove spacing:	± ½ in. (1.3 cm) between any two adjacent grooves
	± 1 whole groove width over a 10 ft (3 m) width perpendicular to the direction of grooving (the proper number can be calculated by dividing 120 by the groove spacing in inches.)
Groove width:	± 1/16 in. (0.16 cm)
Groove depth:	± 1/8 in. (0.3 cm)

Following the completion of groove cutting, the bridge deck and the grooves shall be cleaned of particulate matter and fines by water pressure. The waste water should be properly disposed of.

- Drying

To impregnate the concrete with monomer, it is necessary to remove absorbed water in the concrete to the desired depth of impregnation. The following steps should be followed in drying the substrate:

1. *Expansion.* Before the drying operation begins, all water must be removed from the grooves; expansion joints must be cleaned and joint sealer removed. The joints must be monitored and the heating or drying rate must be reduced if complete closure of a joint becomes imminent. It shall be the responsibility of the contractor to reseal the expansion joints at the completion of the job.

2. *Drying method.* Gas-fired radiant infrared heaters shall be used to dry the concrete. The heaters shall be sufficient in number and size to cover the entire width of the deck for a distance of at least 4 ft (1.2 m), but no more than 20 ft (6.1 m), in the longitudinal direction for each heater setup. The heater units should be mounted on steel casters to permit easy movement from one setup to the next. The heating capacity of the heaters shall be sufficient to provide the of surface temperatures specified in table 4.7.

   The surface temperature shall be monitored under each heater unit with welded, pad-type, copper-constantan, parallel, grounded thermocouple probes (quick-disconnect type) having an Inconel sheath long enough to extend from the center of the heater to outside the heated area. A heat shield of galvanized sheet metal shall be erected around the perimeter of the heaters extending from the deck surface to the height of the heaters. Glass wool insulation (R-19) shall be placed on the deck over a perimeter area 24 in. (61 cm) wide around the heater group for each heater setup to reduce thermal gradients.

Table 4.7 Schedule of Surface Temperatures

Time	Surface Temperature (°F)	Surface Temperature (°C)
Start	Ambient	Ambient
15 minutes	350 ± 25	177 ± 14
30 minutes	450 ± 25	232 ± 14
45 minutes	550 ± 25	288 ± 14
Until Dry	550 ± 25	288 ± 14

3. *Condition-concrete dry.* Drying is considered to be complete when the temperature at the desired depth of impregnation reaches 180°F (82°C) as measured by unsheathed, copper-constantan thermocouples having Teflon conductor insulation and glass braid overall insulation (2, 7). The thermocouples are to have welded hot junctions and are to be set in 0.25 in. (0.6 cm) diameter drilled in the bottom of the deck. The holes shall be backfilled with a fast-set epoxy gel suitable for work on overhead surfaces. A minimum of three embedded thermocouples will be used for each heater setup: one at the bridge centerline and one in the center of each lane. The thermocouples shall remain in place after drying is completed, to monitor cool-down.

4. *Cool-down.* Immediately upon attainment of the drying criterion, the heaters will be moved and the dried area will be covered with R-19 insulation (to minimize thermal cracking by preventing too rapid cool-down of the surface). Heating of the next adjacent area shall immediately follow the placement of the insulation. The insulation shall remain in place until the surface temperature has dropped below 100°F (38°C). The contractor shall make provision for preventing wind from blowing the insulation off the deck.

5. *Weather protection.* Dried areas shall be protected from precipitation, runoff, and other sources of moisture before the application of the monomer mixture. The weather shelter shall be opaque to reduce deck temperatures during the impregnation step. Any dried areas subjected to moisture before treatment must be redried at the direction of the inspector. Dikes constructed of compacted asphaltic concrete cold mix sealed with asphalt emulsion and placed in the roadway upgrade from the bridge deck have been found to be effective in diverting runoff.

6. *Drying time.* The approximate heating time at each heater setup is expected to be 2 to 4 hours according to the surface temperature schedule required in table 4.7. The time will vary with the desired depth of impregnation and ambient

temperature conditions. The drying of the entire area must be carried out as a continuous operation until completed.

- Impregnation

Impregnation involves soaking the dried concrete with a liquid monomer mixture that is contained in the grooves during the soaking process. The monomer mixture is described in section 4.3.3.3. The impregnation process should begin as soon as the deck has cooled to below 100°F (38°C) over the entire area to be impregnated. To minimize the chances of premature polymerization, the impregnation process should commence after sundown. The following describes the impregnation procedure:

1. Any open cracks that may allow the monomer to seep through the deck shall be sealed with a silicone sealant before the application of the monomer.

2. The deck shall be covered with a seamless plastic tarpaulin. The tarpaulin shall have a minimum melting temperature of 325°F (163°C) and be resistant to methyl methacrylate (MMA) vapor and liquid. The tarpaulin shall be anchored around the perimeter of the impregnation area.

3. The monomer shall be introduced to the groove by lifting the plastic sheeting to expose the groove ends. A gravity feed system with a suitable manifold system will facilitate the rapid filling of the grooves. The grooves should be filled to capacity with the catalyzed monomer mixture.

4. Immediately following the completion of the filling operation, the plastic tarpaulin shall be replaced.

5. The impregnation period shall continue until the monomer has disappeared from the grooves, approximately 16 hours. Any excess monomer should be removed using an explosion-proof wet/dry vacuum and disposed of by bulk polymerization.

- Fire Protection

The contractor shall inform the local fire company of the nature of the monomer and operations at the site and shall make arrangements for the fire company to be on site with equipment deemed necessary by the fire company during the period beginning with the introduction of the monomer mixture into the grooves on the bridge deck and ending with the completion of polymerization. Four 20 lb (9 kg) nontoxic, dry chemical fire extinguishers meeting Underwriters Laboratories, Inc., approval for class A, B, and C fires, (minimum rating of 20 A, 80 B, C) shall be kept near the monomer storage area.

- Polymerization

Immediately following the completion of impregnation, the polymerization step shall be implemented. Polymerization is accomplished by heating the impregnated concrete at the top surface until the temperature at the desired depth of impregnation reaches 150°F (66°C).

Polymerization should be carried out using electric heating blankets. Suitable blankets were built for the FHWA's work on "internally sealed concrete." Specifications for the blankets appear in an FHWA status report on internally sealed concrete, dated February 1976. The power requirements are 100 W/ft^2 (1,080W/m^2). The insulation material used for the blankets must resist attack from MMA liquid and vapors. (In the past, monomer has also been successfully polymerized in situ using hot water ponding [8, 9, 10], but this method is extremely inefficient.)

The heating blankets should be placed on top of the plastic tarpaulin. The blankets should be covered with unfaced R-19 fiberglass insulation. When the temperature of the concrete at the depth of impregnation reaches 150°F (66°C), polymerization will be complete (see section 4.3.3.2). After the heat source is removed, the polymerized deck should be covered with unfaced R-19 fiberglass insulation until it cools to ambient temperature.

- Groove Filling

Following polymerization, the grooves shall be filled flush with the pavement surface with latex-modified mortar. Groove filling may begin as soon as the deck has cooled to ambient temperature after polymerization. Grooves shall be blasted clean of any debris and standing water. Grooves shall be saturated surface dry at the time the mortar is placed. The batching and mixing of the latex mortar should be carried out in such a manner as to ensure a steady, continuous groove-filling operation. The mortar should be placed and consolidated in such a manner as to ensure complete filling of the grooves. A rubber-edge squeegee may be used to push the mortar into the grooves and strike it off flush with the deck surface.

- Curing

Within 30 minutes of placement, the fresh mortar should be covered with a single layer of clean, moist burlap. The edges of the burlap should be overlapped 4 in. (10 cm). The wet burlap should be covered with a single layer of white polyethylene sheeting. The latex-modified mortar should be moist-cured for 72 hours following placement. The structure should not be opened to traffic for seven days following placement.

Complete construction specifications are provided in FHWA-PA-85-014, *Deep Polymer Impregnation of a Bridge Deck Using the Grooving Technique* (7).

4.3.3.2 Quality Assurance/Construction Inspection: Polymer Impregnation

The following should be considered the minimum requirements for site inspection:

- Preparation

  1. All requirements outlined in section 3.3.3 shall apply to preliminary repairs.

  2. The contractor should provide the site inspector with a detailed report describing the procedures that will be used for grooving, drying, and impregnation. The site inspector should verify the orientation of the grooves using a spirit level. The site inspector should also ensure that all safety provisions are strictly followed.

The site inspector should ensure the following:

- Drying

Both surface and internal drying temperatures should be continuously logged to ensure that the entire deck is dried to 180°F (82°C) at the desired depth of impregnation. Before construction, the engineer shall determine drying deck lengths and procedure that will prevent expansion joint closure. However, the inspector must carefully monitor the deck joints during drying to prevent closure. The inspector is also responsible for monitoring thermal differentials to ensure that adequate drying occurs while preventing differential thermal cracking.

- Impregnation

The monomer mixture shall be properly mixed before its introduction into the grooves. The deck temperature should be closely monitored to prevent premature polymerization.

The grooves must be completely filled with the monomer mixture, and liquid monomer must remain in the grooves for the specified soak period.

- Polymerization

To ensure complete in situ polymerization of the bridge deck, the polymerization temperatures should be logged using the thermocouples placed for the drying operation. Mortar cubes should be dried in a lab at 230°F (110°C) to a constant weight. The cubes should then be immersed in a sample of the monomer mixture. The cubes are then polymerized in a hot water bath at the minimum temperature logged on the bridge deck and periodically tested for compressive strength. When the compressive strength reaches a maximum, about 4 times the unpolymerized strength, the polymerization heat source can be turned off. The polymerized area should then be covered with unfaced R-19 fiberglass insulation until it returns to ambient temperatures, to prevent differential thermal cracking.

- Groove Filling

The site inspector should ensure the following:

1. Grooves are clean and the surfaces are saturated surface dry (no free water or standing water).
2. The proper proportions are used for batching the latex-modified mortar.
3. The latex-modified mortar shall be pushed into the grooves with the squeegee. The mortar should flow in front of the squeegee.
4. The mortar must be *moist*-cured for 72 hours.

4.3.3.3 Material Performance Specifications: Polymer Impregnation

- Methyl Methacrylate Monomer Mixture

The monomer mixture shall consist of 100:10:0.5 parts by weight methyl methacrylate (MMA); trimethylolpropane trimethacrylate (TMPTMA); and 2,2-azobis(isobutyronitrile) (AZO), respectively. MMA is the basic monomer in the system. TMPTMA is a cross-linking agent used to enhance the polymerization process and to reduce the vapor pressure of the system. AZO is the initiator for the system.

MMA shall meet the following requirements:

Formula	$CH_2=C(CH_3)COOCH_3$
Inhibitor	25 ppm hydroquinone
Molecular weight	100
Assay (gas chromatography)	99.8% minimum
Density	7.83 lb/gal (0.938 kg/L)
Boiling point	212°F (100°C)
Flash point (Tag, ASTM D 1310)	55°F (13°C)

TMPTMA shall meet the following requirements:

Formula	$(CH_2{=}CH_3\ COOCH_2)_3CCH_2CH_3$
Inhibitor	100 ppm hydroquinone
Assay	95.0% minimum
Density	8.82 lb/gal (1.058 kg/L)
Flash point (Cleveland ASTM D 92)	> 300°F (149°C)

AZO shall meet the following requirements:

Empirical formula	$C_8H_{12}N_4$
Minimum purity	96%

It is recommended that the MMA and TMPTMA be obtained from a supplier preproportioned in the required ratio (100:10 by weight) and be shipped in 55 gal (208 L) drums with 400 lb (181 kg) of mixture (approximately 50 gal [189 L]) per drum. This eliminates the need for mixing the MMA and TMPTMA at the site and allows enough space for the addition and mixing of the catalyst (AZO) on a drum-by-drum basis at the job site. The AZO should be obtained in 1.82 lb (826 g) packages to simplify mixing (i.e., one package per drum of MMA/TMPTMA mixture).

- Safety in Handling and Storing Monomer

The monomer materials must be stored, handled, and applied in such a manner as to prevent fire, explosion, and injurious exposure to personnel associated with the job and to the public. Unsafe handling practices will be sufficient cause to discontinue work until hazardous procedures are corrected. Occupational Safety and Health Administration material storage, handling, and safety specifications shall be adhered to in the performance of this work.

Storage of monomer materials. The premixed MMA/TMPTMA monomer system shall be stored in the original shipping containers. Maximum monomer storage temperature shall not exceed 90°F (32°C). The storage facility shall be located and constructed to provide protection from direct sunlight, fire hazard, and oxidizing chemicals. Sufficient ventilation shall be maintained to prevent the hazardous buildup of monomer vapor concentrations within the storage air space. "Warning" and "No Smoking" signs shall be placed at appropriate intervals. Facilities shall be available in the monomer storage area to spray water on the drums if the drum temperatures should reach 90°F (32°C). The AZO shall be stored in accordance with the manufacturer's recommendations, but in no event shall the catalyst storage temperature exceed 35°F (2°C). The catalyst shall be stored in an area separate from the monomer.

Ignition sources. All electrical equipment, lighting, and power sources must conform to the explosion-proof requirements of the National Electrical Code. No smoking shall be permitted in and around the monomer storage facilities or on the job site when the monomer is being mixed, transferred, or applied. Drums must be electrically grounded during mixing and transfer operations.

Personnel protection. Personnel working with the monomers and catalyst shall be provided with and use safety glasses or goggles, impervious gloves, aprons, and boots. Normally in outdoor monomer applications respiratory equipment is not necessary. However, self-contained respiratory equipment should be available in case of emergency. A field eyewash and water washing facility should be available on the job site.

Accidental spills of monomer materials. Accidental spills must be absorbed with vermiculite or dry sawdust and collected with nonsparking tools, and shall be disposed of in accordance with government regulations.

- Mixing of Monomer with the Polymerization Catalyst

The AZO shall be mixed with the monomer system no more than 60 minutes before use. Monomer temperature at the time of catalyst addition shall not exceed 90°F (32°C).

Mixing and transfer equipment must not be made of copper or brass, which may cause bulk polymerization by chemical reaction. Any plastic components should be resistant to the monomer. Mixing shall be accomplished using explosion-proof propeller-type stirrers or by bubbling small volumes of air at low pressure (1 to 2 psi [6.9 to 13.7 kPa]) through the mixing vessel.

The drum containing the monomer mixture should be electrically ground. After the monomer mixture is mixed for 5 minutes, a premeasured package of AZO catalyst should be removed from its refrigerated storage location and added to the mixing container. The mixture should not be mixed during the addition. The monomer mixture should be mixed for 30 minutes following the addition of the AZO. The catalyzed monomer should be used within 4 hours of the addition of the catalyst. The unused portion should be disposed of by bulk polymerization.

Additional information and lists of possible suppliers are available in FHWA-PA-85-014, *Deep Polymer Impregnation of a Bridge Deck Using the Grooving Technique* (7).

- Latex Modified Mortar

The following materials and proportions are recommended:

1. Latex admixture conforming to the prequalification standards of FHWA Notice 5140.15, July 1978, should be added at a rate of 3.5 gal/sack (13 L/sack).

2. Cement:sand (dry basis) = 1:2.90

3. Cement type and content, ASTM C 150 type I or II, 658 lb/yd^3 (390 kg/m^3)

4. Maximum water/cement ratio 0.37 (including water in latex emulsion)

5. Fine aggregate, ASTM C 33

6. Air-entrainment agent should not be used.

7. Total air content, 7% ± 1%

8. Slump, range 4 to 6 in. (10 to 15 cm)

9. Minimum compressive strength at 28 days, 4,000 psi (31 MPa)

4.3.4 Superstructure and Substructure Elements: Patching with Corrosion Inhibitors

- **Description**

There are two types of superstructure/substructure corrosion inhibitor rehabilitation patching methods. Type I is a standard patching method using a corrosion inhibitor-modified concrete patching material. Type II uses the same patching material but includes four spray-on inhibitor applications on the exposed reinforcing steel and patch cavity before patching. For both methods, all the damaged concrete, sound but actively corroding concrete, and critically chloride-contaminated concrete is removed. Active corrosion potential and critically chloride-contaminated concrete sampling and application methodologies are presented in sections 2.4.1.1 and 3.2.1.

For type I methods, concrete removal areas are marked out and scored 0.75 in. (1.9 cm) deep along the patch perimeter with a dry concrete saw, concrete is removed at least 0.75 in. (1.9 cm) below the rebar, and the cavity is backfilled with a corrosion-inhibitor-modified concrete. A penetrating sealer is applied to the entire structural element. For type II methods, before backfilling the patch cavity, the patch cavity and rebar receive four spray-on applications of corrosion inhibitor. Because of potential bond problems, when Cortec 2020 is used as the spray-on inhibitor the cavity area is sandblasted to remove the surface residue.

- Limitations

The influence of removing a large amount of concrete at one time on the structural capacity of the member must be considered.

- Estimated Service Life

The service life of the type II method compared with the type I method is not significantly affected by the spray-on applications of inhibitor. For prepatching highly corrosive conditions, the type II method should be used in place of the type I method.

The estimated service life of both the type I and type II corrosion inhibitor patching methods is estimated to be greater than 50 years.

- Estimated Cost

The estimated costs of the type I and type II methods were determined by standard engineering estimating procedures. Details are presented elsewhere (4). The costs are national average costs for mid-1991.

The presented costs include all costs associated with the patching methods, including labor, materials, and equipment for removing the concrete, cleaning the patch cavity and rebar, applying the corrosion inhibitor, backfilling the patch cavity, and curing the concrete. The costs also include maintenance and protection from traffic and contractor operations and profit. Table 4.8 presents the costs for four levels of concrete removal, two corrosion inhibitors (type I, DCI and Cortec 2000; type II, Postrite/DCI and Cortec 2020/2000), and seven bridge members.

4.3.4.1 Construction Procedure: Patching with Corrosion Inhibitors

The following addresses the patching of superstructure and substructure elements using corrosion inhibitors. Two types of rehabilitation patch techniques are identified: type I and type II. Type I is a standard patching method for which a corrosion inhibitor is added to the patching material. Type II is identical to type I, with the addition of four spray applications of a corrosion inhibitor to the repair cavity. The repair cavity can be backfilled with cast-in-place concrete or shotcrete. The following will primarily address cast-in-place concrete; notes on the use of shotcrete are provided in section 4.3.4.3. These techniques can also be applied to encasement and jacketing. However, costs have not been developed for these procedures.

- Delineation of Areas to Be Removed

Areas of unsound concrete should be located by sounding with a hammer. The unsound concrete is evidenced by a hollow sound. The area to be removed should be delineated by the site engineer as the unsound or delaminated area plus a 2 in. (5 cm) periphery. Delineations should be made in a manner that minimizes cuts, avoids small "islands" of sound concrete, and avoids acute angles.

Table 4.8 **Superstructure/Substructure Rehabilitation Using Corrosion Inhibitor-Modified Concrete and Corrosion Inhibitor Spray-On Patch Systems ($/yd²)**

Bridge Member	Percent Concrete Removal	Type I*		Type II*	
		DCI	Cortec 2000	Postrite/DCI	Cortec 2020/2000
Beams	0.5	218	217	227	224
	1.0	124	119	132	129
	2.0	75	71	84	81
Diaphragms	1.0	1,429	1,425	1,437	1,434
	2.0	691	687	699	697
	3.0	478	473	486	483
Piers	2.0	690	686	698	695
	5.0	288	284	297	294
	10.0	219	215	166	163
Pier caps	5.0	372	368	380	377
	10.0	219	214	208	205
	40.0	219	214	131	128
Backwalls	5.0	1,386	1,381	1,394	1,391
	10.0	707	702	715	712
	40.0	253	249	205	202
Abutments	2.0	1,761	1,757	1,769	1,766
	5.0	721	716	729	726
	10.0	492	488	382	379
Wing walls	2.0	2,118	2,173	2,186	2,183
	5.0	864	859	872	869
	10.0	445	411	454	451

$1/yd²) = $1.196/m²

* Type I = Remove concrete to below rebars and patch with corrosion-inhibitor-modified concrete.

* Type II = Remove concrete to rebar depth and apply corrosion inhibitor-spray-on patch system.

The delineated area should be outlined with a saw cut 0.75 in. (1.9 cm) deep. Care should be taken to avoid cutting existing reinforcing steel. In no case will feathered edges be acceptable.

- Removal of Unsound Concrete

The unsound concrete should be removed with pneumatic breakers of less than 35 nominal pounds (15.9 kg). Pneumatic breakers should be fitted with bull-point chisels (pointed). Small hand chisels should be used to provide clearance around reinforcing bars. The pneumatic breakers should be operated at an angle no greater than 45 degrees from the plane of the removal surface.

The minimum depth of removal is 0.75 in. (1.9 cm) around the periphery of the exposed reinforcing steel. However, concrete should be removed to the depth necessary to expose sound concrete. Care should be taken to maintain structural integrity during concrete removal.

- Preparation of Repair Cavity

Once all the unsound concrete has been removed, the cavity should be gritblasted to remove laitances. All exposed reinforcing steel should be blasted to near white metal (all scale and rust removed). Reinforcing bars with greater than 25% sectional loss as determined by the engineer should be lapped with reinforcing bar of equal diameter for 30 bar diameters on either side of the deteriorated area.

Following the completion of gritblasting, the cavity should be airblasted to remove all dust and debris. The compressed-air source used for grit- and airblasting should have oil and water traps installed.

Water must not be allowed to contact the type II removal areas after the cover concrete is removed. Should water wet the type II removal areas before the application of the corrosion inhibitor, the concrete shall be dried 30 minutes with a propane-fired infrared heater to a surface temperature of 400°F (204°C). Spray application of the inhibitor shall be delayed until the concrete has returned to ambient temperature.

Formwork specifications are beyond the scope of this report. However, the following should be considered to ensure a good bond:

1. Formwork shall be sufficiently anchored and braced to prevent bulging.

2. Formwork should be designed to withstand concrete vibration forces.

3. Formwork should be designed to give the fluid concrete sufficient head to fill all voids and bond to the substrate, especially when patching overhead surfaces such as the bottom of pier caps.

More information on formwork can be obtained from ACI 347R, *Guide to Formwork for Concrete* (11).

- Application of Surface-Applied Corrosion Inhibitor: Type II Only

Application of the surface-applied inhibitor involves spraying the dry concrete cavity with a liquid corrosion inhibitor. The inhibitor can be successfully applied with an industrial-grade garden sprayer. Four applications of the inhibitor are to be applied and allowed to soak into the base concrete before overlay. The inhibitors that can be applied, and their corresponding concentrations and application rate are as follows:

1. *Postrite* is a water-based inhibitor containing 15% by weight calcium nitrite to be used as a surface treatment. Each application of the Postrite solution is to be applied at the rate of 150 ft^2/gal (3.7 m^2/L). Four applications of the inhibitor are to be applied, the second at one hour and the third at eight hours after the initial application. The last application should be applied immediately before the erection of formwork or the application of shotcrete. This application should be applied a minimum of 16 hours after the initial application.

2. *Cortec MCI 2020* is a water-based inhibitor to be used as a surface treatment or by injection. Each application of Cortec MCI 2020 shall be spray-applied at the rate of 225 ft^2/gal (5.5 m^2/L). Four applications of the inhibitor are to be applied, the second at two hours, the third at 12 hours, and the fourth at 24 hours after the initial application. Once the fourth application has been allowed to soak in (surface appears dry), the cavity should be lightly gritblasted to remove any residues left by the inhibitor. Failure to do so may lead to bond failure. Formwork may be erected following the gritblasting.

Safety and handling measures for these inhibitors are discussed in section 4.3.4.3.

- Placement and Consolidation

The patch material may be batched and mixed at the site or supplied ready mixed. The patch material should be placed in a manner that will prevent segregation. The patch material should be consolidated with an internal vibrator.

- Curing

Positive steps should be taken to provide moist curing to superstructure and substructure repairs. Minimum requirements include keeping forms cool and moist and draping exposed areas with wet burlap. Formwork should be left in place for a minimum of seven days for repairs that are not moist-cured. Within 24 hours of the removal of such formwork, a curing membrane should be applied to maintain the internal humidity of the repair concrete. Shotcrete patches should be moist-cured for a minimum of 7 days; curing membranes are not acceptable.

- Penetrating Sealer

Following 28 days of curing, a penetrating sealer should be applied to the entire substructure component. The sealer should be applied in accordance with section 3.2.6.

4.3.4.2 Quality Assurance/Construction Inspection: Patching with Corrosion Inhibitors

The site engineer will be responsible for delineation of the areas of concrete to be removed and the depth of removal. The engineer should ensure that all of the provisions for concrete removal are strictly enforced.

- Substrate Preparation

In assessing deck preparation, the site inspector should examine the following:

1. Ensure that the exposed reinforcing steel is free from all rust and scale.

2. Determine the sectional loss and specify lapping of reinforcement bars where required.

3. Ensure that the patch cavity is free from gritblast material, latencies, and contaminants.

- Application of Surface-Applied Corrosion Inhibitors

The site inspector should monitor and enforce the following:

1. The application rate and concentration of the inhibitor: The inspector should ensure that the inhibitor is applied until the surface appears saturated.

2. If water is allowed to contact the treated area after the initiation of the inhibitor applications, the affected area should be dried and the application process repeated.

3. Safety regulations applying to the storage and handling of the inhibitors must be enforced.

4. The inspector should ensure that all inhibitor residues are removed by grit-blasting before concrete placement.

The following are recommended for mixing placement and consolidation:

1. Equipment used for transporting concrete at the job site should be inspected

before placement to ensure that appropriate measures have been taken to prevent segregation as outlined in ACI 304R, *Guide for Measuring, Mixing, Transporting, and Placing Concrete* (12).

2. Ensure that the patching materials as supplied meet the requirements outlined in section 4.3.4.3.

3. Inspect the concrete mixer, water-measuring device, and aggregate-weighing scale before the first concrete is batched and mixed. Aggregate proportioning by volume or shovel is not acceptable.

4. Prepare compressive-strength cylinders and perform slump and air-content tests. The minimum requirement for testing frequency is two tests per day: the initial batch and a second, randomly chosen batch.

5. Atmospheric conditions should be monitored and recorded.

- Acceptance

All patches will be sounded after the removal of forms to check for adequate bond and voids. Unsound, voided, or honeycombed areas should be removed and replaced.

4.3.4.3 Material Performance Specifications: Patching with Corrosion Inhibitors

- *Postrite/DCI*:

If Postrite is used as the spray-on inhibitor, DCI (30% calcium nitrite) should be added to the patch concrete. The rate of addition should be 6 gal/yd^3 (30 L/m^3). DCI acts as a set accelerator and requires the addition of a high-range water reducer and initial set retarder. The addition rates provided below should be used as guidelines. Trial batches should be prepared with local materials to ensure proper air content, workability, and set time.

The inhibitors must be stored, handled, and applied in such a manner as to prevent injurious exposure to personnel associated with the job and to the public. Personnel working with the inhibitor shall be provided with and use safety glasses or goggles and rubber or other impervious gloves. Additional protective clothing should be worn to minimize skin contact. Spills should be absorbed with an inert, noncombustible medium and removed for disposal in accordance with existing federal, state, and local environmental regulations. Spills should be prevented from entering drinking water supplies and streams or groundwater.

The following materials, proportions, and properties are recommended for the overlay and repair concrete:

1. DCI should be added as an admixture at the rate of 6 gal/yd^3 (30 L/m^3). Note that the water in the admixture must be accounted for in the water/cement ratio. DCI contains approximately 7.0 lb water per gallon (0.84 kg/L).

2. High-range water reducer conforming to ASTM C 494 type G should be added at a rate recommended by the manufacturer.

3. Retarding admixture conforming to ASTM C 494 type B or D should be added at a rate recommended by the manufacturer.

4. Cement content, ASTM C 150 type I or II, 611 lb/yd^3 (362 kg/m^3)

5. Maximum water/cement ratio 0.43

6. Aggregates, ASTM C 33, with maximum aggregate size of 0.5 in. (1.3 cm)

7. Total air content, 7.5% $\pm$ 1%

8. Slump, range 3 to 4 in. (8 to 10 cm)

9. To control or reduce shrinkage, the total water content should be kept to a minimum. Concrete not meeting consistency requirements should be rejected.

10. *Shotcrete*: DCI should be added directly to the mixer for dry-mix process; no adjustments are necessary for wet mix. The rate of addition should be 0.5 gal (1.9 L) per hundred weight of cement. The high-range water reducer and retarder should not be included.

Postrite and DCI can be obtained from

Construction Products Division
W.R. Grace and Company
62 Whittemore Avenue
Cambridge, MA 02140
(617) 876-1400

- *Cortec MCI 2020/2000*:

If MCI 2020 is used as the spray inhibitor, MCI 2000 should be added to the repair and overlay concrete. The rate of addition should be 2 lb/yd^3 (1.2 kg/m^3). A high-range water reducer should be added to improve workability. Research has shown that MCI 2000 has no effect on the properties of fresh concrete (2). However, trial batches should be prepared with local materials to ensure air content, workability, and set time.

The inhibitor should be stored in original shipping containers. Containers should be kept tightly closed and kept away from heat, open flame, and spark sources. Personnel working with the inhibitor shall be provided with and use safety glasses or goggles, NIOSH-approved respirators, and chemical-resistant rubber or plastic gloves. The contractor shall provide a field eyewash and safety shower to be used in the event of an accidental splash of inhibitor on the workers. Additionally, workers shall be required to thoroughly wash hands with soap and water before eating, smoking, drinking, or using the lavatory. Accidental spills should be absorbed with a sweeping compound or other absorbent material. The compound shall be or disposed of according to existing federal, state and local environmental regulations.

The following materials, proportions, and properties are recommended for the overlay and repair concrete:

1. Cortec MCI 2000 should be added at the rate of 2 lb/yd^3 (1.2 kg/m^3).

2. A high-range water reducer meeting ASTM C 494 type G should be added at a rate recommended by the manufacturer.

3. Cement content, ASTM C 150 type I or II, 611 lb/yd^3 (362 kg/m^3).

4. Maximum water/cement ratio 0.43

5. Aggregates, ASTM C 33, with maximum aggregate size of 0.5 in. (1.3 cm)

6. Total air content, 7.5% $\pm$ 1%

7. Slump, range 3 to 4 in. (8 to 10 cm)

8. To reduce shrinkage, the total water content should be kept to a minimum. Concrete not meeting consistency requirements should be rejected.

9. *Shotcrete*: The rate of addition is 6 fl oz (177 mL) per hundred weight of cement. The MCI 2000 should be dispersed in 32 fl oz (946 mL) water before addition to dry-mix process. No changes in addition are necessary for wet-mix applications. The high-range water reducer should not be included in the wet-mix process.

Cortec MCI 2000/2020 can be obtained from

Cortec Corporation
St. Paul, MN 55107
(612) 224-5643

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5

Rapid Deck Treatment Methods

5.1 Introduction

This chapter presents standard "rapid" protection, repair, and rehabilitation methods used by SHAs. As in chapter 3, methods applied to concrete bridge decks are described with respect to limitations, estimated service life, estimated price or cost, and construction procedures and specifications. However, this chapter emphasizes those aspects of the deck treatments that make them suitable for rapid installation, and minimizes the duplication of information contained in previous chapters.

5.2 Criteria for Rapid Bridge Deck Treatment Methods

Rapid bridge deck treatment methods are those that are suitable for installation during off-peak traffic periods and open to traffic during peak traffic periods. A flow diagram for rapid bridge deck treatment methods is shown in figure 5.1. Although deck replacement is an option in a rapid treatment situation, replacement is outside the scope of this manual (1).

Lane closure, concrete removal, and surface preparation are necessary first steps for any rapid deck treatment. Lane closure can be accomplished with cones (or other temporary barriers) or a concrete barrier system that facilitates rapid placement and removal; (see figure 5.2) (2). All unsound concrete must be removed in preparation for new treatment materials. Necessary forms must be placed for full-depth patches, and surfaces to which concrete should bond must be blasted clean, in accordance with specifications.

If there is insufficient time to install and cure a patching material or protection system, temporary materials (steel plates, asphalt concrete, etc.) should be placed as needed to maintain a traffic-bearing surface. Otherwise, the deck treatment should continue with the installation of rapid-curing concrete treatment material and the curing of the materials to the required strength to receive traffic. Necessary temporary materials are installed, and the

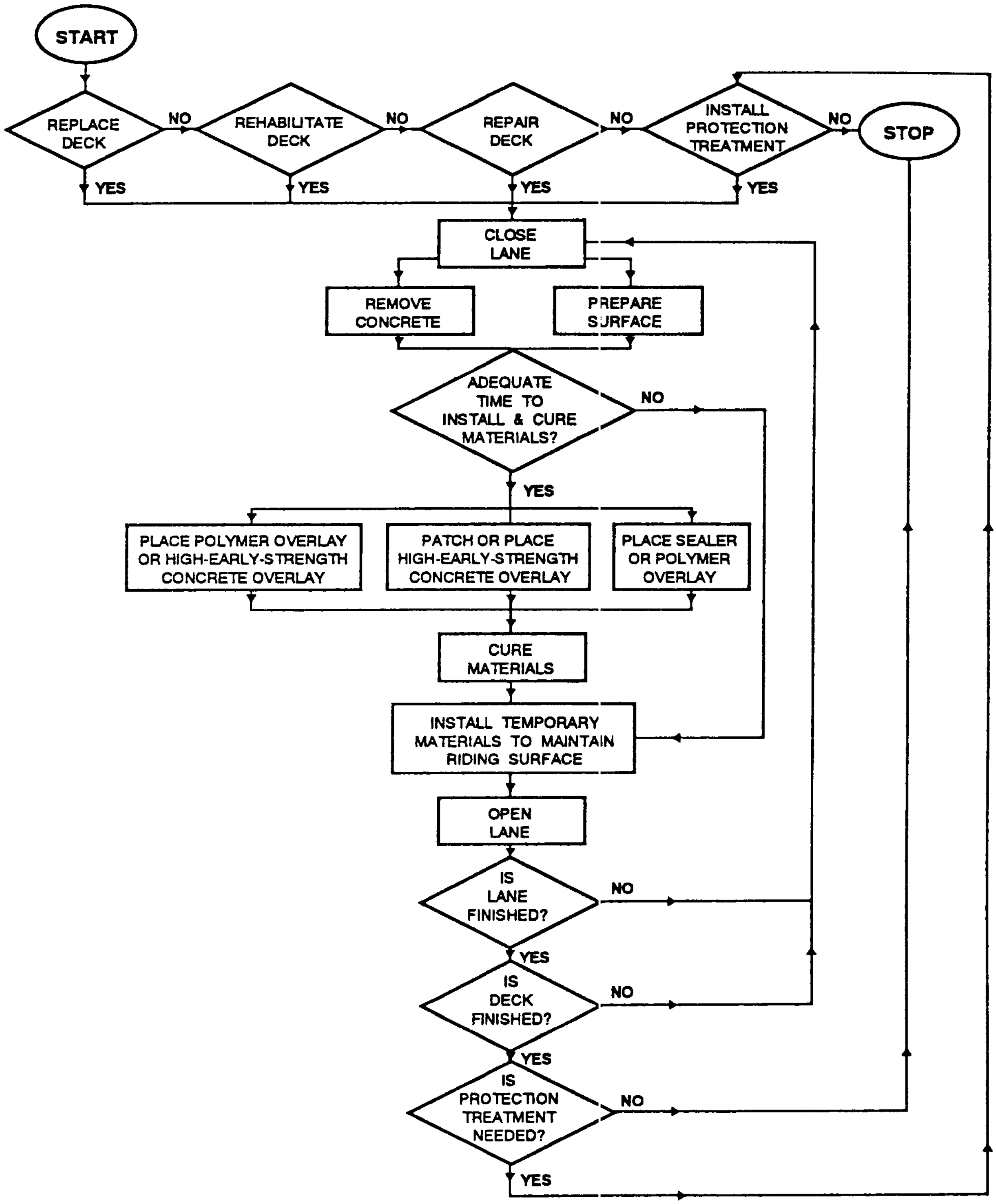


Figure 5.1. Flow Diagram for Rapid Bridge Deck Treatment Methods

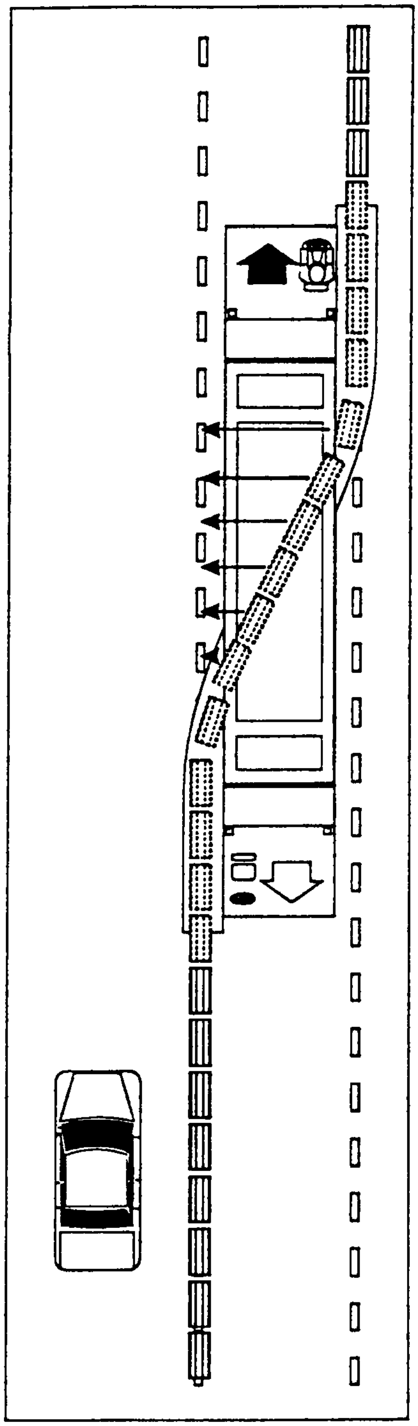


Figure 5.2 Rapid Concrete Barrier Placement and Removal System

lane is opened to traffic.

A bridge deck that must receive a rapid treatment method can usually be classified into four maximum lane closure time conditions that require the use of one of four rapid treatment methods as follows:

21 to 56 hours: semirapid (e.g., 9 p.m. Friday to 5 a.m. Monday)

12 to 21 hours: rapid (e.g., 6:30 p.m. to 3:30 p.m.)

8 to 12 hours: very rapid (e.g., 6 p.m. to 6 a.m.)

less than 8 hours: most rapid (e.g., 9 p.m. to 5 a.m.)

A deck treatment must follow the flow diagram (figure 5.1) within the lane closure constraints of 21 to 56, 12 to 21, 8 to 12, or less than 8 hours to qualify as part of a rapid deck treatment method.

5.2.1 Methods

As discussed in chapters 2, 3, and 4, deck treatment methods include protection, repair, and rehabilitation. Protection methods restrict the infiltration of chloride ions into concrete that is not critically contaminated with chloride (chloride ion content exclusive of background chloride is less than 1.0 lb/yd^3 [0.6 kg/m^3] and half-cell potentials are less negative than -250 mV, CSE). Rapid protection methods used by SHAs include asphalt overlays on preformed membranes, polymer overlays, and sealers.

Repair methods do not deal with the cause of deterioration, but rather emphasize the rapid replacement of deteriorated, delaminated, and spalled concrete. Rapid repair methods used by SHAs include asphalt overlays with and without preformed membranes, high-early-strength hydraulic cement concrete overlays, patching with high-early-strength hydraulic cement concrete and asphalt concrete, and polymer overlays.

Rehabilitation methods include the removal of all deteriorated, delaminated, and critically chloride-contaminated concrete, followed by patching and placement of a protective system. Rapid rehabilitation methods include asphalt overlays on membranes, high-early-strength hydraulic cement concrete overlays, and polymer overlays. Rehabilitation methods have a much longer service life than repair methods because critically chloride-contaminated concrete is removed when rehabilitation methods are used. Asphalt concrete overlays on membranes and sealers are covered in more detail in chapter 3. Almost all methods can be used with lane closures of less than 8 hours, but whenever feasible, longer lane closures should be used to enhance quality.

5.2.2 Minimum Curing Time

One of the most important properties of a rapid protection, repair, or rehabilitation method is the strength of the materials at the time they are first subjected to traffic. Materials that do not have adequate strength can be damaged by traffic and fail prematurely as a result of a failure of the matrix or bond. Materials must be relatively free of cracks and must be adequately bonded to the substrate to protect the deck and provide skid resistance.

The most convenient indicators of the strength of hydraulic cement concretes and mortars are the compressive strengths of 4 x 8 in. (10 x 20 cm) cylinders for concrete and 2 in. (5 cm) cubes for mortar. Hydraulic cement concretes and polymer concretes are usually required to have a compressive strength of 2,500 to 4,000 psi (17.2 to 27.6 MPa) before being subjected to traffic (3). Guillotine shear bond strengths of at least 200 to 400 psi (1.4 to 2.8 MPa) are usually obtained at these compressive strengths when concrete substrates are properly prepared (4, 5). Tensile adhesion strengths greater than 100 psi (0.7 MPa) are also indicative of satisfactory performance (6, 7, 8).

Sealers must be tack-free at the time they are subjected to traffic. Membranes must be tack-free before being overlaid with asphalt concrete, which is then allowed to cool to 150°F (66°C) before it is opened to traffic (3). Patches that can be protected with a steel plate can be opened to traffic once the plate is in place.

Table 5.1 shows estimates of the minimum curing times needed to subject protection methods to traffic without causing major damage to them. The estimates are based on compressive and bond strength data, tack free times, and asphalt concrete cooling rate data obtained from the literature and the responses to the questionnaire sent to the materials suppliers (3, 7 through 22). Curing time is a function of the curing temperature of the material, which is a function of the mixture proportions, the mass, the air and substrate temperatures, and the degree to which the material is insulated. The values in table 5.1 are reported as a function of air temperature for typical installations. Research is needed to provide additional values and to refine the estimates shown in the table.

The minimum curing times in table 5.1 are for an asphalt concrete overlay placed on a prefabricated, rubberized asphalt membrane and prime coat. Approximately 1 hour is required for the prime coat to cure at 75°F (24°C). At 90°F (32°C), the prime coat usually cures faster; however, a minimum of approximately 1 hour curing time is still required for the asphalt concrete to cool to 150°F (66°C) (3, 9). At temperatures of 55°F (13°C) and below, the curing time is controlled by the curing rate of the prime coat.

Minimum curing times for hydraulic cement concrete can be reduced by increasing the rate of reactions by adjusting the mixture proportions, applying insulation, and increasing the mass of the application. Asphalt concrete cools more rapidly when placed in thin lifts, and sealers become tack-free sooner when the application rate is reduced. Patches constructed with materials similar to those used in overlays should have minimum curing times similar to

Table 5.1 Minimum Curing Times of Rapid Protection Methods (Hours)

References	Method	Installation Temperature, o F (o C)			
		40 (4)	55 (13)	75 (24)	90 (32)
Dickson, Corlew (9) VA DOT (3)	Asphalt concrete overlay on membrane	NA	2	2	2
Carrasquillo, Farbiarz (12) Popovics, Rejenderan (17)	Hydraulic cement concrete overlay (special blended cement)	5	4	3	3
Sprinkel (19) Streb (21) Temple, Balluo, Fowler, Meyer (22)	(magnesium phosphate)	1	1	1**	1**
Kubacka, Fontana (15) Sprinkel (7)	Polymer overlay (epoxy)	2*	6	3	2
Sprinkel (7)	Sealer (silane)	4	3	2	1
Sprinkel (18)	(high-molecular weight-methacrylate)	N/A	9	3	1

NA: Not applicable since materials are not usually placed at indicated temperature.

* Special cold weather formulation of methacrylate.

** Special hot weather formulation of magnesium phosphate.

those shown in table 5.1, with the exception of asphalt concrete patches. These patches are suitable for traffic in one hour or less.

Judging by the data in table 5.1, all the rapid deck treatment methods can satisfy the requirements for a most rapid treatment. However, the high-early-strength hydraulic cement concrete overlays would have to be constructed with special blended cements and admixtures or magnesium phosphate (12, 17, 19, 21, 22). The more conventional high-early-strength PCC overlays, such as those constructed with 15% latex and type III cement or 7% silica fume and high-range water-reducing admixtures, would only satisfy the requirements for a semirapid treatment (5, 16). Although both high-early-strength latex-modified and silica fume-modified PCCs can achieve 3,000 psi (20.7 MPa) compressive strength in less than 21 hours, additional lane closure time is required to prepare the deck, set up the screed, and moist-cure the concrete. To obtain optimum properties, latex-modified PCC should be moist-cured for two days and silica fume concrete for three days. Although liquid membrane curing materials can be applied to silica fume concrete once the wet burlap is removed, some loss in durability should be expected when less than optimum curing time is provided.

5.3 Protection Methods

5.3.1 Polymer Overlays

5.3.1.1 Description

Polymer concrete overlays similar to those currently in use have been installed on PCC bridge decks in many states during the past 15 years. The overlays are usually placed on decks to reduce the infiltration of water and chloride ions into the concrete and improve skid resistance, ride quality, and surface appearance (23, 24). Three types of overlays are typically used:

Multiple-layer: two or more layers of unfilled polymer binder and broadcast gap-graded, clean, dry angular-grained aggregate.

Slurry: a polymer aggregate slurry struck off with gauge rakes and covered with broadcast aggregate.

Premixed: a polymer concrete mixture consolidated and struck off with a vibratory screed.

The most frequently used binders for polymer concretes and mortars are epoxy, polyester styrene, and methacrylate (25, 26, 27, 28, 29). The binders are usually two-component systems, one component containing the resin and the other containing curing agent or initiator. The aggregates are usually silica and basalt. Fillers such as coke breeze are used to impart conductivity when the overlays are used as part of a cathodic protection system (30). Uniformly graded aggregates are used with premixed overlays and most slurry overlays, and gap-graded aggregates are used with multiple-layer overlays and are broadcast on the top of slurry and some premixed overlays.

Decks that are likely candidates for polymer overlays have the following characteristics:

1. Cover concrete that is not critically chloride-contaminated (see section 3.2.1)
2. Cover concrete over the reinforcement bars with a permeability > 2,000 coulombs (AASHTO T277)
3. Clear cover over the reinforcement bar of < 2 in. (5 cm)
4. Extensively cracked cover concrete (particularly decks that were constructed without epoxy-coated reinforcement bars)
5. Bald tire skid number (ASTM E 524) < 20 at 40 mph (64 kph)

5.3.1.2 Limitations

Decks that are not good candidates for polymer overlays have the following characteristics:

1. Corrosion-induced delaminations and spalls
2. Cover concrete that is critically chloride-contaminated (see section 3.2.1)
3. Half-cell potentials more negative than -250 mV CSE
4. Unsound concrete (tensile rupture strength less than 150 psi [1.0 MPa])
5. Poor drainage
6. Poor ride quality (although slurry and premixed overlays can correct minor surface irregularities)

Surfaces must be shotblasted, sound, clean, and dry to obtain a high bond strength. Large cracks should be filled ahead of time. The successful application of a polymer concrete overlay includes use of acceptable materials, adequate surface preparation, proper batching and placement of materials, and adequate curing before subjecting the overlay to traffic.

Some very flexible binders show a rapid loss in skid number when subjected to high volumes of traffic and high temperatures (31).

5.3.1.3 Estimated Service Life

The literature review revealed an average service life estimate of 10 years for polymer overlays (8, 32, 33, 34, 35). The questionnaire responses produced estimates from 6 to 25 years, with an average of 13 years (36).

Service life estimates for the rapid deck protection and rehabilitation treatments based on the SHRP C 103 field evaluations are summarized in table 5.2 (31). The life of the rehabilitation is controlled by permeability to chloride ion, skid resistance, adhesion, or

Table 5.2 Service Life of Rapid Protection Treatments Based on Field Evaluations (Years)

Treatments	Age of Oldest Instal. Eval. (years)	Projected Minimum Service Life (years) (ADT=5000)	Property Controlling Service Life	Adjustments for Traffic: Service Life Adjustment (years) for ADT[a]				Adjustments for Traffic: Average Permeability (coulombs) for ADT[a]			
				L	M	H	VH	L	M	H	VH
Overlays											
Multiple-Layer Epoxy	15	25	Permeability	0	0	-10	-15	125	300	700	800
Multiple-Layer Epoxy-Urethane	7	25	Skid Number	0	-10	--	--	125	150	--	--
Premixed Polyester	8	25	Adhesion	0	--	--	--	150	--	--	--
Methacrylate Slurry	6	18	Skid Number	0	-11	-13	-15	0	0	0	0
Multiple-Layer Polyester	9	10	Adhesion	0	0	0	0	1100	1250	1300	1350
Multiple-Layer Methacrylate	9	15	Adhesion	0	0	0	0	1100	1250	1300	1350
Special Blended Cement	1	--	CIS	-	--	--	--	--	--	--	--
Latex & Type III Cement	5	25	CIS	0	0	0	0	600	600	600	600
Silica Fume	5	25	CIS	0	0	0	0	600	600	600	600
High-Molecular-Weight Methacrylate Sealer	9	7	Permeability	0	0	--	--	2200	2200	--	--
Asphalt Concrete Patch	1	1	Adhesion	0	0	--	--	5000	5000	--	--
Portland Cement Concrete Patch	5	25	CIS	0	0	--	--	300	300	--	--
Magnesium Phosphate Concrete Patch	6	25	CIS	0	0	0	0	1100	1100	1100	1100

[a] 1990 average daily traffic (ADT): L = < 5,000; M = 5,000 to 25,000; H = > 25,000; VH = > 50,000.

-- Data not available to make projection.

CIS: corrosion-induced spalling.

corrosion-induced spalling (CIS).

Judging by the performance data obtained from the field evaluations, multiple-layer epoxy overlays should provide very low permeability to chloride ion for 25 years or more on decks with low to moderate average daily traffic (ADT). Also, they should remain bonded and provide adequate skid resistance for at least 25 years. Although both adhesion and skid number decrease as traffic increases, projections based on the field evaluations indicate that decks with very high ADT should have an average adhesion strength of approximately 100 psi (689 kPa) and an average skid number of approximately 36 at 25 years of age (31). However, permeability to chloride ion is projected to increase from less than 125 coulombs for decks with less than 5,000 ADT to 1,880 coulombs for decks with greater than 50,000 ADT at 25 years of age. Therefore at high and very high ADT overlays should be replaced at less than 25 years to prevent a reduction in the time to corrosion of the reinforcement, replacement at 15 years for high ADT and 10 years for very high ADT, see table 5.2 for adjustment factors.

Multiple-layer epoxy-urethane overlays should perform as well as the epoxy overlays for decks with low ADT. For decks with moderate ADT the bald tire skid number may be less than 20 in 15 years or more. Reductions in life due to low skid numbers could not be determined for higher ADT (31).

The effect of traffic volume on the life of premixed polyester overlays could not be determined, since all overlays evaluated were subjected to low ADT.

Methacrylate slurry overlays should provide negligible permeability for at least 18 years, but they could fail because of a low skid number in 18 years on decks with low traffic, 7 years for decks with moderate traffic, 5 years for decks with high traffic, and only 3 years on bridge decks subjected to very high volumes of traffic.

Multiple-layer polyester overlays provide less protection and similar skid resistance to epoxy overlays until they fail in adhesion in about 10 years.

High-modulus, brittle, multiple-layer methacrylate overlays should provide low permeability and good skid resistance until they fail in adhesion in about 15 years. Traffic volume does not seem to be a factor in the failure in adhesion of these overlays. Since these overlays do not provide the level of protection provided by the other overlays, corrosion-induced spalling is likely to occur at an earlier age. The performance data for polymer overlays based on field evaluations (table 5.2) are generally more favorable than the data based on the questionnaire response and literature review.

The projected minimum service life values shown in table 5.2 are for protection and rehabilitation treatments in which the concrete is not critically chloride-contaminated, as described in chapter 3.

5.3.1.4 Estimated Construction Prices

Thin polymer overlay price was determined from analysis of bid prices. The price includes furnishing all equipment, material, and labor needed to complete the work. A thin polymer overlay consists of a polymer material (unspecified in submittal of bid prices) applied to a deck surface with aggregate broadcast on top. The overlay can be placed in one or several lifts and is between 0.25 to 0.75 in. (0.6 to 1.9 cm) thick. The price of thin polymer overlays is influenced by quantity and number of bidders and can be estimated from the following relationship:

$$y = 56.109 + (-0.000359)X$$

where y = Predicted national adjusted price ($\$/yd^2$)
X = Job quantity (yd^2) x number of bidders

For a 2,900 yd^2 deck with four bidders, the price is $\$52/yd^2$ ($\$62/m^2$).

For a specific polymer overlay system, such as premixed polyester overlays that are placed like portland cement overlays, the price is a function of quantity only. This includes all labor, materials, and equipment necessary to furnish and install a polyester overlay. The price can be estimated from the following relationship:

$$y = 22.088 + 0.00118X + 42251.9/X^{0.9183}$$

where y = Predicted national adjusted price ($\$/yd^2$)
X = Job quantity (yd^2)

For a 2,900 yd^2 deck, the price is $\$53/yd^2$ $\$64/m^2$)

5.3.1.5 Materials

The contractor shall use materials that satisfy the requirements of the material specifications.

5.3.1.6 Surface Preparation

Surfaces must be shotblasted, sound, clean, and dry to obtain a high bond strength. Concrete that does not have a tensile rupture strength (load/area) (ACI 503R, VTM 92) of at least 150 psi (1.0 MPa) must be removed and replaced so that the overlay can bond to a sound substrate (25, 37). Large cracks (more than 0.04 in. (0.1 cm) wide) should be filled ahead of time using gravity-fill epoxies, high-molecular-weight methacrylate, or other proven crack- sealing methods (41). When possible, sand should be used to extend the crack-filling polymer.

Once the deck has been patched and large cracks have been repaired, the surface should be cleaned by shotblasting to remove asphaltic materials, oils, dirt, rubber, curing compounds, paint, carbonation, laitance, weak surface mortar, and other detrimental materials that may interfere with the bonding or curing of the overlay. Shotblasting equipment is available in a range of sizes. Units that blast a 9 in. (23 cm) wide strip and a 6 ft (1.8 m) wide strip are shown in figure 5.3. (Other approved cleaning practices such as sandblasting and waterblasting have been used, but most specifications require shotblasting.) Sandblasting must be used to clean along the edges of the deck and other areas that cannot be cleaned by shotblasting. Tensile adhesion testing (figure 5.4) is necessary to ensure that the surface preparation precedure results in adequate polymer overlay bond strength.

The test method prescribed in ACI 503R should be used to determine the cleaning practice (size of shot, flow of shot, forward speed of shotblast machine, and number of passes) necessary to provide a tensile rupture strength (ACI 503R, VTM 92) greater than or equal to 250 psi (1.7 MPa) or a failure area at a depth of 0.25 in. (0.64 cm) or more into the base concrete on more than 50% of the test area (25, 37). A test result should be the average of three tests on a test patch of the overlay. The size of the test patch should be at least 1 x 3 ft (30 x 91 cm) and shall be prepared from a batch of polymer concrete that is typical of a batch to be used in the overlay. One test result should be obtained for each span or 500 yd^2 (418 m^2) of deck surface, whichever is the smaller area.

The cleaning practice should be approved if passing tensile adhesion test results are obtained for each test patch (24, 27, 28, 29). Figure 5.5 shows that the temperature of the overlay can affect the test result; therefore, the overlay temperature at the time of the test should be recorded.

5.3.1.7 Methods of Mixing, Placing, and Curing Materials

The overlay should be placed the same day the surface is shotblasted. Areas that are not overlaid should be shotblasted again just before placement of the overlay.

- Mixing and Placing

Polymer binders shall be furnished in two or three components for combining immediately before use, in accordance with the manufacturer's instructions. Component A shall be a promoted or unpromoted resin. Component B shall be a hardener or initiator. Component C, if necessary, shall be a promoter that is mixed with component A before adding component B. Components that can segregate during storage should be stirred before use. The same paddle shall not be used to stir different components. Equipment and tools may be cleaned with toluene, xylol, or methyl ethyl ketone before the binder has set. For premixed and slurry overlays, the aggregates shall be thoroughly mixed with the binder before placing the concrete on the deck.

Multiple-layer overlays are typically constructed in the following manner. First, an epoxy binder is spread over a shotblasted deck surface with notched squeegees (figure 5.6). Gap-

Figure 5.3 Shotblast Equipment

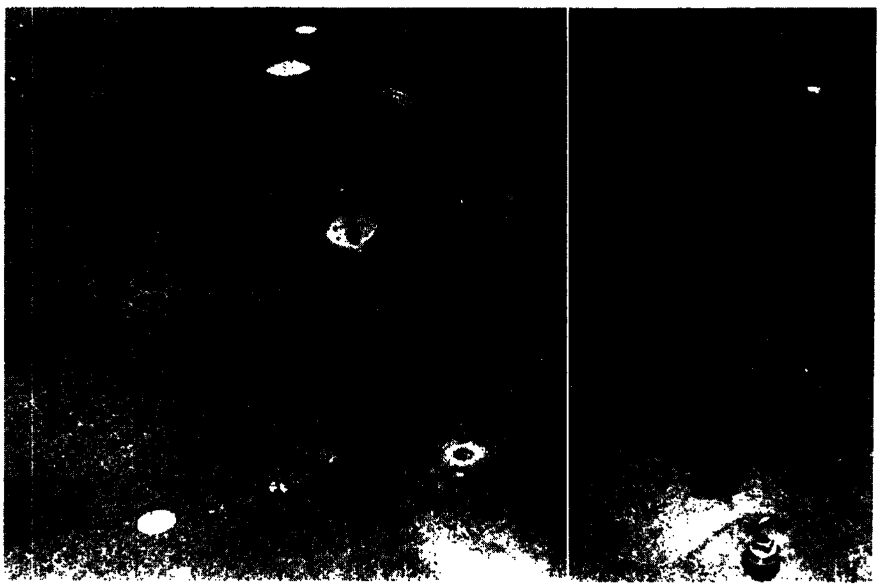


Figure 5.4 Tensile Adhesion Testing

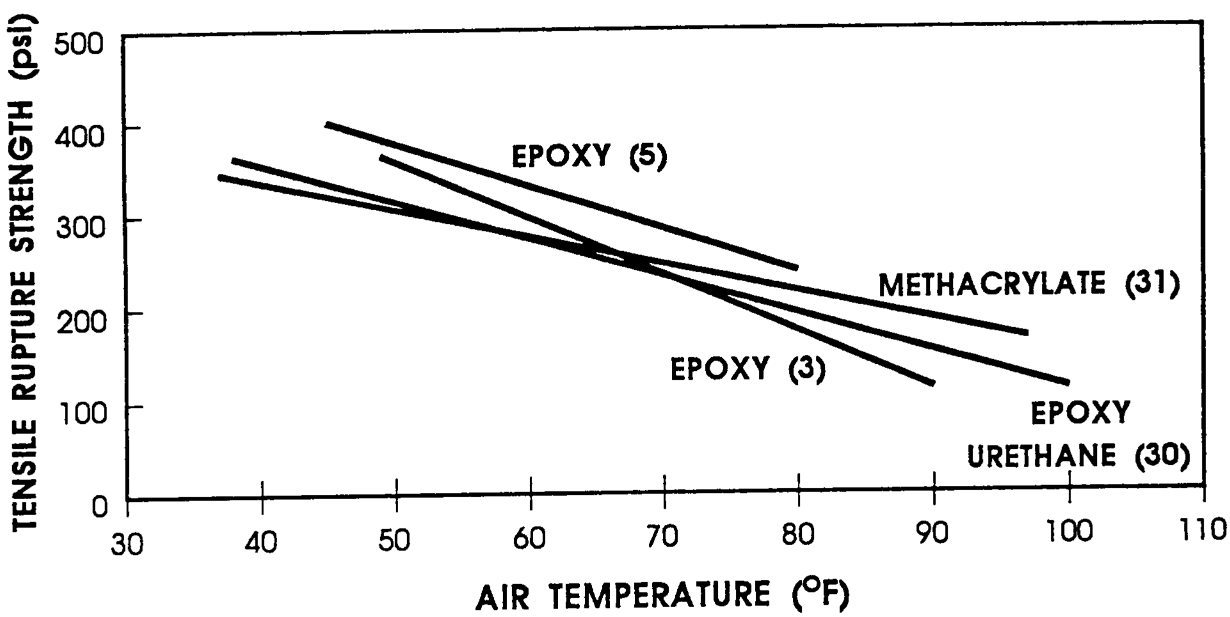


Figure 5.5 Tensile Rupture Strength (ACI 503R) versus Test Temperature for Polymer Overlays

Figure 5.6 Spreading Binder on a Shotblasted Deck Surface

graded basalt aggregate is then broadcast to excess to provide one layer of a multiple-layer overlay. The aggregate can be broadcast from the back of a dump truck (figure 5.6) or from an aggregate spreader oscillating in the transverse direction (figure 5.7). The aggregate should be placed within 5 minutes after the binder is mixed (see tables 5.3 and 5.4). Once the binder has cured, the unbonded aggregate is removed and a second layer is applied. Approximately 10 lb/yd^2 (4 kg/m^2) of aggregate is broadcast onto 2 lb/yd^2 (1.1 kg/m^2) of resin for layer 1, and approximately 14 lb/yd^2 (7.6 kg/m^2) of aggregate is broadcast onto 4 lb/yd^2 (2.2 kg/m^2) of resin for layer 2. The resin content of the overlay is about 25% by weight (27, 28, 38).

Slurry overlays are typically constructed by application of a prime coat at 0.75 lb/yd^2 (0.41 kg/m^2), followed by a slurry mixture consisting of about 5 lb/yd^2 (2.7 kg/m^2) of binder, 7 lb/yd^2 (3.8 kg/m^2) of silica sand, and 5 lb/yd^2 (2.7 kg/m^2) of silica flour. The slurry should be struck off with gauge rakes set to provide a layer of slurry at least 0.19 in. (0.48 cm) thick (figure 5.8). This is followed by a broadcast to excess of a gap-graded aggregate as used in multiple-layer overlays; aggregate should be broadcast to excess onto the slurry before the slurry begins to gel; finally, a seal coat is applied at 1.25 lb/yd^2 (0.68 kg/m^2). Including the prime coat and the seal coat, the binder content of the overlay is about 24% (see table 5.3) (29). When epoxy is used as a binder, silica flour is not used in the slurry, and no seal coat is required.

Premixed overlays are constructed by mixing the dry aggregates with about 12% binder by weight (see table 5.3) (26, 39). A primer is usually applied at a rate of about 0.75 lb/yd^2 (0.41 kg/m^2) to enhance the bond strength. A primer should be applied with rollers, brooms, or squeegees approximately one hour before placement of a premixed polyester styrene resin concrete overlay. Typically, the primer must gel on the deck before placement of the overlay. Polyester styrene resin concrete may be consolidated and struck off with a transverse vibrating screed (figure 5.9). Continuous batching and paving equipment has been used to place some overlays. In figure 5.10, a vibrating slipform paver is used to apply a premixed polyester styrene resin concrete overlay to a deck that has received a polyester primer approximately one hour earlier. A mobile concrete mixer supplies freshly mixed concrete to the paver. Acceptable skid resistance can be obtained by batching the overlay with skid-resistant aggregates, placing grooves in the freshly placed concrete, or broadcasting aggregate onto the surface.

The thickness of the overlays is typically about 0.25 in. (0.64 cm) for the multiple-layer, 0.30 in. (0.79 cm) for the slurry, and a minimum of 0.5 in. (1.3 cm) for the premixed.

- Curing

Before opening the overlay to traffic, it should be cured for the minimum time shown in table 5.5 and for additional time as required to obtain a compressive strength of 1,000 psi (6.9 MPa) in field-cured cubes (ASTM C 109) or as required by the manufacturer of the resin to prevent damage to the overlay.

Figure 5.7 Broadcasting Aggregate onto Binder from an Oscillating Spreader

Table 5.3 Typical Polymer Concrete Application Rates (lb/yd²)

Overlay	Multiple-Layer Epoxy	Slurry Methacrylate	Premixed Polyester
Thickness (in.)	0.25	0.30	0.75
Prime coat	—	0.75 + 0.25	0.75 + 0.25
Layer 1 resin	2.0 ± 0.25	5.0 ± 0.50	9.75 ± 0.75
Layer 1 aggregate	10.0 ± 1.0	12.0 ± 1.0	71.0 ± 1.0
Layer 2 resin	4.0 ± 0.25	—	—
Layer 2 aggregate	14.0 ± 1.0	14.0 ± 5.0	—
Seal coat resin	—	1.25 + 0.25	—
Approximate resin content (%)	25	24	13

1 in = 2.54 cm; 1 lb/yd^2 = 0.54 kg/m^2

Table 5.4 Typical Aggregate Gradation (Percent Passing Sieve)

Sieve Size	Multiple-Layer Overlays	Slurry Overlays		Premixed Overlays
0.5 in.	—	—	—	100
0.38 in.	—	—	—	83-100
No. 4	100	—	—	65-82
No. 8	30-75	—	—	45-64
No. 16	0-1	100	—	27-48
No. 20	—	90-100	—	—
No. 30	—	60-80	—	12-30
No. 40	—	5-15	—	—
No. 50	—	0-5	—	6-17
No. 100	—	—	—	0-7
No. 140	—	—	100	—
No. 200	—	—	98-100	0-3
No. 270	—	—	96-100	—
No. 325	—	—	93-99	—

1 in. = 2.54 cm

Figure 5.8 Use of Gauge Rakes in Slurry Overlays

Figure 5.9 Use of a Transverse Vibrating Screed for Premixed Overlays

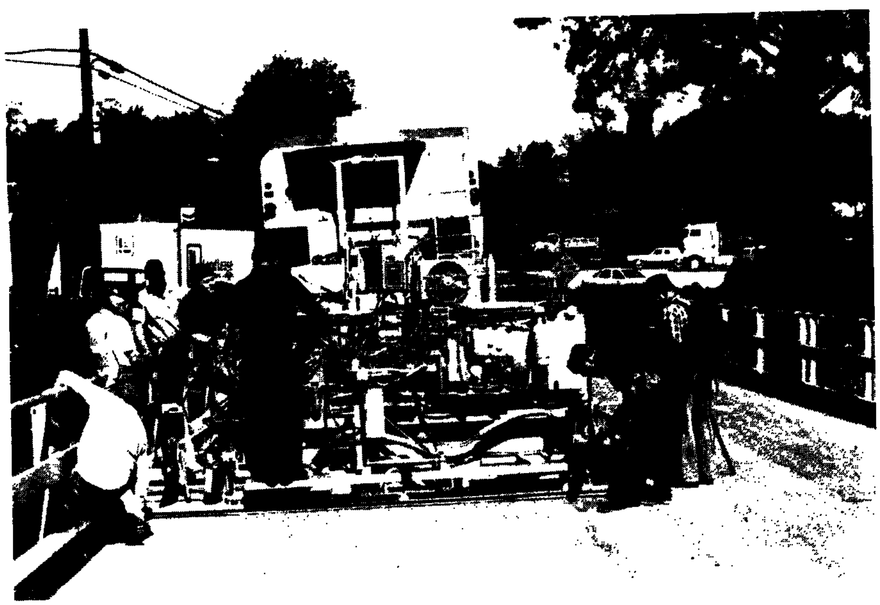


Figure 5.10 Overlay Placement Using Continuous Paving Equipment

Table 5.5 Typical Properties of Binders and Polymer Concrete

Property	Epoxy (28)	Polyester (26)	Methacrylate (29)	Test Method
Viscosity (cps)	700-2,500	75-200	200-1,300	ASTM D 2393
Gel time (min)	15-45	20-120	15-45	AASHTO T237
Tensile strength (psi)	2,000-5,000	Minimum 2,500	Minimum 1,200	ASTM D 638
Tensile elongation (%)	30-70	Minimum 35	100-200	ASTM D 638
Bond strength (psi)	Minimum 250a	Minimum 500b	Minimum 250a	a-VA TM 92 b-CAL TM 551
Compressive strength (psi)	Minimum 5,000	—	—	ASTM C 109
Cure time @ 90°F (hour)	2	2	2	—
Cure time @ 75°F (hour)	3	3	3	—
Cure time @ 60°F (hour)	6-8	5-6	4	—

1 psi = 6.89 kPa; °C = 0.55 (°F - 32)

5.3.1.8 Quality Assurance and Construction Inspection Program

The tests and test results required to ensure that an overlay with a life of 15 to 25 years is installed may be summarized as follows:

- *Resin*

Table 5.5 presents a typical range of values for viscosity and gel time (26, 28, 29). The viscosity of the individual or mixed components is specified to control the coating of the aggregates and the surface on which the polymer is placed. Binder resins with a low viscosity are suitable for highly filled premixed and slurry mixtures and prime coats. Resins with a higher viscosity are usually used for multiple-layer overlays to allow for proper coating and filling of the space between the gap-graded aggregates. The gel time is specified to ensure that there is adequate time to place the overlay materials (minimum gel time) and that the curing will be completed within the time required to apply traffic to the overlay (maximum gel time). Some specifications have a maximum gel time of 20 min to ensure that a high early strength is obtained and a complete cure is a achieved (27). Additional properties of the uncured binders, such as specific gravity, stability, and component content, are also sometimes specified.

Table 5.5 also shows the typically specified cured properties of binders (26, 28, 29) that have had a long service life. It would appear that a tensile strength of 2,500 psi (17.2 MPa) is compatible with an elongation of 50% for epoxies and polyesters. A lower tensile strength of 1,200 psi (8.3 MPa) is satisfactory for methacrylate binders because the tensile elongation is typically much higher. A sample of resin should be obtained for each 1,000 gal (3,785 L) of resin to be used. The viscosity, gel time, tensile strength, tensile elongation, and bond strength of the resin should be measured and the resin accepted when values as shown in table 5.5 are obtained.

- *Aggregate*

A sample of aggregate should be obtained for each 100,000 lb (45,360 kg) of aggregate to be used. The gradation should be measured and the aggregate accepted when values as shown in table 5.4 are obtained. Aggregates that will be subject to wear shall be silica or basalt and shall have a Mohs scale hardness of about 7.

Aggregates should be specified to be dry (less than 0.2% moisture), angular-grained silica sand or basalt and free from dirt, clay, asphalt, and other organic materials (27, 28).

- *Concrete*

Table 5.5 shows typical mechanical properties of polymer concretes used in overlays that have exhibited a long service life (18, 23, 24, 25, 26, 27, 28, 29, 38). The Virginia Department of Transportation requires a minimum bond strength (ACI 503R, VTM 92) of

250 psi (1.7 MPa), and a minimum compressive strength (ASTM C 109) of 5,000 psi (34.5 MPa) at 24 hr (28). The California Department of Transportation specifies a minimum flexural bond strength (California Test 551) of 500 psi (3.4 MPa) (26). The samples of resin and aggregate should be used to prepare specimens of polymer concrete. The specimens should be tested and should exhibit the properties shown in table 5.5.

- *Surface Preparation*

Procedures should be approved when test patches constructed with approved materials, as described earlier, are found to have a minimum tensile rupture strength of 250 psi (1.7 MPa) (37). When the tensile tests cause failures in the base concrete at depths greater than 0.25 in. (0.64 cm) and at strengths less than 150 psi (1.0 MPa), the concrete should be removed and replaced with higher-quality concrete before preparation of the surface and placement of new test patches. When the surface is being prepared, it should be cleaned thoroughly, at least as well as before the placement of test patches. Before placement of the overlay materials, a visual inspection of the surface should be made to ensure that the surface is properly prepared, free of dust, and dry.

- *Application*

The application should be monitored to ensure that the gel times as shown in table 5.5 and the application rates and mixtures proportions as shown in table 5.3 are achieved.

- *Curing*

Before the overlay is opened to traffic, it should be cured for the minimum time shown in table 5.5 and also as required to obtain a compressive strength of 1,000 psi (6.9 MPa) in field-cured cubes (ASTM C 109). The curing time is a function of the curing agent or initiator type and amount, binder content, and curing temperature. The times in table 5.5 are typical of the minimum curing times required to prevent loss of aggregate under traffic and typical of the times required for a mortar cube (ASTM C 109) to obtain a compressive strength of 1,000 psi (6.9 MPa) (18, 23, 27, 28). A minimum curing time of two hours is possible at 90°F (32°C), but the minimum curing time usually needs to be increased as the curing temperature decreases. Curing times of six to eight hours may be required for multiple-layer epoxy overlays placed at 60°F (16°C). At lower temperatures, days may be required for cure, or proper cure may not be achieved. However, methacrylates and special formulations of other polymer concretes can be prepared to cure in three hours or less at temperatures below 60°F (16°C).

5.3.1.9 Material Specifications

Polymer binders and concretes shall have the properties shown in table 5.5. Aggregates shall be dry (less than 0.2% moisture), angular-grained silica sand or basalt, with a gradation as specified in table 5.4, free from dirt, clay, asphalt, and other organic materials, and shall have a Mohs scale hardness of about seven.

5.3.2 Sealers

Sealers are used for rapid deck protection because they can be applied easily and quickly. They are also well suited for off-peak traffic period application because, in general, no problems arise when traffic is placed on a deck in which one span or lane is treated and one is not. Only high-molecular-weight methacrylate is discussed below; other sealers, particularly silanes and siloxanes, are discussed in detail in chapter 3.

5.3.2.1 Description

Sealers are placed on bridge decks and other concrete surfaces to reduce the infiltration of chloride ion and water (7, 40, 41, 42, 43). The materials can usually be applied by spray, roller, brush, or squeegee. Most sealers have a low solids content (< 40%) and tend to penetrate the surface pores and capillaries of the concrete; after evaporation of the carrier, they leave a thin hydrophobic film 0 to 10 mil (0 to 250 μm) thick on the surface. However, some sealers, such as high-molecular-weight methacrylate, have a high solids content (usually 100%) and leave a film 10 to 30 mil (250 to 750 μm) thick.

Decks that are likely candidates for sealers have the same characteristics as those that are candidates for polymer overlays (see section 5.3.1.1), with the exceptions that decks with less than 2.0 in. (5 cm) cover and decks with low skid numbers should receive overlays rather than sealers to provide adequate cover and skid resistance.

5.3.2.2 Limitations

To provide adequate skid resistance, sealers, particularly those with a high solids content, must be placed on heavily textured surfaces or overlaid. Satisfactory textures to which sealers can be applied can be obtained by tining the fresh concrete, by shotblasting the hardened surface, or by sawcutting grooves 0.13 in. (0.32 cm) wide by 0.13 in. (0.32 cm) deep by approximately 0.75 in. (1.9 cm) to 1.5 in. (3.8 cm) on centers in the hardened concrete. Also, the deck must be patched before placing the sealer. Surfaces must be sound, clean, and dry to obtain maximum penetration and adhesion of the sealer. Decks that are not good candidates for sealers have the same characteristics as those that are not good candidates for polymer overlays (see section 5.3.1.2). In addition, decks with less than 2.0 in. (5 cm) cover or low skid numbers are not good candidates.

5.3.2.3 Estimated Service Life

Like other sealers, high-molecular-weight methacrylates placed on decks are limited by the rate of wear of the treated deck surface. Decks exposed to low ADT have an estimated life of seven years (see table 5.2).

5.3.2.4 Estimated Construction Prices

Cost was determined from analysis of bid prices and includes all labor, materials, and equipment necessary to perform the work, including surface preparation and broadcasting of sand on the treated surface. Price is influenced by quantity, contract amount, and MPT and can be predicted by the following relationship:

$$y = 8.9813 + (-2.687 \times 10^{-6})X$$

where y = Predicted national adjusted price ($/yd²)
X = Job quantity (yd^2) times contract amount ($) divided by MPT amount ($)

The price of a high-molecular-weight methacrylate for a 2,900 yd^2 deck with a contract amount of $156,000 and MPT of $12,000 is $8.88/$yd^2$ ($10.60/$m^2$).

5.3.2.5 Construction Procedures

The successful application of a sealer includes use of acceptable materials, adequate surface preparation, proper mixing and placement of materials, and adequate curing before subjecting the sealer to traffic.

- Materials

The contractor shall use materials that satisfy the requirements of the material specifications.

- Surface Preparation

Surfaces must be shotblasted, sound, clean, and dry to obtain good penetration and adhesion. Concrete that does not have a tensile rupture strength of at least 150 psi (1.0 MPa) should be removed and replaced so that the sealer can adhere to a sound substrate (25, 37). Large cracks (more than 0.04 in. [0.1 cm] wide) should be filled ahead of time using high-molecular-weight methacrylate or other proven crack-sealing materials (40). When possible, sand should be used to extend the polymer, either by preplacement in cracks or by mixing a mortar.

Once the deck has been patched and large cracks have been repaired, the surface should be cleaned by shotblasting (see figure 5.3) and other approved cleaning practices to remove asphaltic materials, oils, dirt, rubber, curing compounds, paint, carbonation, laitance, weak surface mortar, and other materials that may interfere with the penetration or curing of the sealer. Sandblasting must be used to clean along the edges of the deck and other areas that cannot be cleaned by shotblasting. Surfaces should be shotblasted within 24 hours before applying the sealer. Surfaces that cannot be sealed within the lane closure period shall be cleaned again before application of the sealer. The deck should be dry before placing the sealer.

A test application of sealer of at least 1 x 3 ft (30 x 91 cm) should be made on each span or 500 yd^2 (418 m^2) of deck surface, whichever is the smaller area. The evaluation of test applications should include one or more of the following tests:

1. Tack-free time
2. Water beading (hydrophobic surface appearance)
3. In-place permeability
4. Permeability tests on cores (AASHTO T277)
5. Macro texture (ASTM E 965)

The cleaning practice and application procedures should be approved if satisfactory curing, penetration, protection, and macro texture are obtained for each test application. Temperature can affect the penetration and curing of the sealer; therefore the temperatures of the air, deck, and sealer at the time of the applications should be recorded. When temperatures differ significantly from those when the test applications were done, it may be advisable to evaluate more representative applications.

- Methods of Mixing and Placing Materials

The sealer should be placed the same day the surface is shotblasted. Areas that are not sealed should be shotblasted again just before placement of the sealer. Materials should be mixed and applied according to the manufacturers recommendations unless the results from the test applications indicate that other application procedures would give better results. The materials can usually be applied by spray, roller, brush, or squeegee (see figure 5.11). Application rates typically range from 100 to 200 ft^2/gal (2.5 to 5.0 m^2/L).

- Curing

Materials shall be cured until they are tack-free or as otherwise recommended by the manufacturers. Table 5.1 shows that curing time typically increases as the temperature decreases.

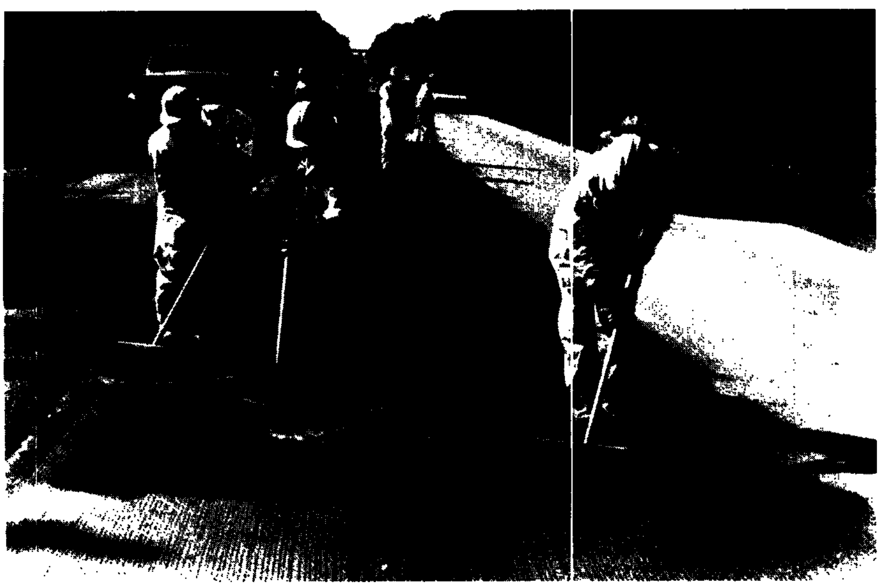
5.3.2.6 Quality Assurance and Construction Inspection Program

The tests and test results required to ensure a sealer with a life of 7 years or more are summarized as follows:

- *Sealer*

Table 5.6 shows typical properties for high-molecular-weight methacrylate. A sample of sealer should be obtained for each 1,000 gal (3785 L) to be used. The sealer should be tested for solids content, permeability, and tack-free time. A certification of compliance with specifications should be obtained for all materials supplied. Final acceptance of the shipment should be based on obtaining satisfactory performance when used in test applications.

a. With an airless sprayer



b. With brooms

Figure 5.11 Application of a High-Molecular-Weight Methacrylate Sealer to Tined Surfaces

Table 5.6 Typical Properties of High-Molecular-Weight Methacrylate

Sealer	High-Molecular-Weight Methacrylate	Test Method
Color Part A	Amber	—
Part B	Pale yellow	—
Part C	Red, violet	—
Mixing ratio A:B, A:C (vol.)	100:4, 100:2	—
Percent solid (wt.)	100	—
Shelf life (months)	12	—
Specific gravity	0.9-1.1	ASTM D 2849
Flash point (°F)	> 180	ASTM D 3278
Mixed viscosity (cps)	8-20	ASTM D 2393
Pot life (min.)	20-45	AASHTO T237
Tack-free time (hours)	3-8	California T551
Initial cure (hours)[a]	3-8	—
Bond strength (psi)	500	California T551
Application rate (ft^2/gal.)	100-125	—

1 gal = 3.785 L

°C = (°F - 32)/1.8

1 psi = 6.879 kPa

1 ft^2/gal = 0.0246 m^2/L

[a] Open to traffic.

For details, see references 7 and 26.

- *Surface Preparation*

The surface shall be prepared as described in section 5.3.2.5.

- *Application*

The sealer shall be applied as described in section 5.3.2.5.

- *Curing*

Before opening the sealed deck to traffic, the surface should be tack-free and cured in accordance with the manufacturer's recommendations and the requirements of the specification.

5.3.2.7 Material Specifications

High-molecular-weight methacrylate sealers shall have properties similar to those shown in table 5.6.

5.3.3 Asphalt Overlay on Preformed Membrane

Asphalt overlay on preformed membrane was the rapid deck protection system most frequently cited by SHAs in a questionnaire survey (31, 36). The most significant challenge in successfully applying this method during off-peak traffic periods is to provide for the movement of traffic over areas with and without the overlay and to protect the lap joints of the membrane that occur between subsequent lane closures. Typically, speed restrictions and other signing and delineating measures are used. Also, the cost of the asphalt overlay is increased because portions of the overlay must be placed during each lane closure. Asphalt overlay on preformed membrane is discussed in detail in chapter 3.

5.4 Repair Methods

Sealers, asphalt overlays on membranes, and polymer overlays can be used for rapid deck repairs, but such use is generally not cost-effective because by definition all critically chloride-contaminated concrete (see chapter 3) is not removed and premature failure of the systems would occur because of corrosion-induced spalling.

5.4.1 Patching

5.4.1.1 Description

The most frequently used method of rapidly repairing a bridge deck involves removal of delaminated concrete, sandblasting the concrete surface, and filling the cavity with a high-performance concrete (40, 44, 45, 46). Sometimes cracks are also repaired and a rapid curing protective system is installed (7, 8). This method has several advantages. The patching, crack repair, and application of the protective system can be done in stages. The patched area can usually be opened to traffic in two to four hours. Concrete removal costs are low because very little concrete is removed, and the high cost of the patching materials is offset by the low volume of material required.

The disadvantage of the method is that spalling will continue because all salt-contaminated concrete is not removed, and thus corrosion is not stopped. Other disadvantages may be as follows: (1) all poor-quality concrete is not removed, (2) there is insufficient time to prepare the surface, (3) the rapid-setting materials are not properly consolidated or placed, (4) the patches crack because of shrinkage, (5) the repairs must be opened to traffic before sufficient strengths are developed, and (6) the repair materials are not similar to or compatible with the materials repaired (7, 8, 40, 43, 44, 45, 46, 47, 48).

5.4.1.2 Asphalt Concrete

Transportation agencies have a responsibility to provide a deck riding surface that is safe. Consequently, when decks spall the cavity is usually filled with asphalt concrete until a more permanent repair can be made. In warm weather an asphalt concrete mixture that hardens as it cools (hot mix) is used to fill potholes. In cold weather a mixture that cures by evaporation of solvents (cold mix) is used. A proper repair includes removal of dust, debris, and unsound concrete from the cavity, application of a tack coat, and placement and compaction of the patching material (49). Asphalt patches should be used only as a temporary repair, and they should be replaced with a hydraulic cement concrete patch as soon as practical.

5.4.1.3 High-Early-Strength Hydraulic Cement Concrete

The most common method of permanent spall repair is patching with hydraulic cement concrete. Patches may be shallow (above level of reinforcment but at least 1.3 in. [3.3 cm] thick), half depth (at least 1 in. [2.5 cm] below top mat of reinforcement but not deeper than half the deck thickness), and full depth (3). A typical repair includes squaring up the area to be patched, saw cutting the perimeter to a depth of 1 in. (2.5 cm), removing concrete to the required depth with pneumatic hammers weighing less than 30 pounds (13.6 kg), blasting the concrete surface and reinforcement with sand or slag, filling the cavity with the patching material, consolidating and striking off the material, and application of liquid or other curing material (3). When full-depth patches are constructed, it is necessary to suspend forms from the reinforcing steel or to support forms from beam flanges (areas greater than 3 ft^2 [0.28 m^2]). Hydrodemolition may also be used to remove concrete before patching.

Many types of patching materials can be used (50, 51, 52). Patches can be constructed and cured to a strength suitable for traffic in less than 8 hours using special blended cements such as Pyrament; type III portland cement with admixtures such as corrosion inhibitors, high-range water reducers, latex, and silica fume; and rapid-hardening cementitious materials that satisfy the requirements of ASTM C 928 (5, 12, 13, 14, 22, 53, 54, 55, 56).

The most frequently used material is the rapid-hardening cementitious material meeting the requirements of ASTM C 928 (see figure 5.12). Many of these materials achieve a compressive strength of 2,500 to 3,000 psi (17.2 to 20.7 MPa) in 3 hours or less, depending on the temperature (see table 5.1). Typically the prepackaged materials are mixed in a small concrete mixer of approximately 6 ft^3 (0.17 m^3) capacity. For convenience the bags may contain cement and sand. Contractors extend the mix by adding up to 40% coarse aggregate (typically less than 0.5 in. [1.3 cm] maximum size) by weight. For economy, contractors purchase the special blended cement and add bulk fine and coarse aggregates in simple basic proportions of 1:1:1 by weight.

When one or more closely spaced cavities require 1 to 2 yd^3 (0.76 to 1.5 m^3) or more of patching material, special blended cement concretes have been batched using mobile concrete mixers and ready-mix trucks (13, 19, 20). Figure 5.13 shows special blended cement ready-mix concrete being placed to replace the top half of a bridge deck. The deck was saw cut 1 in. (2.5 cm) deep into segments 11 ft (3.4 m) wide by 8 ft (2.4 m) long, and the top halves of one or two segments were replaced each day with a lane closure that started at 8 a.m. and ended by 5 p.m. Concrete was removed until 11 a.m. each day, the ready-mix concrete containing 940 lb/yd^3 (558 kg/mV) of special blended cement was placed by noon, and the lane was opened between 3 p.m. and 5 p.m. each day depending on the curing temperature.

The advantage of using ready-mix concrete or a mobile concrete mixer is that optimum mixture proportions are usually prescribed, whereas when a small portable mixer is used there is a tendency to use less than optimum mixtures such as 1:1:1 by weight. High-quality patches can be obtained with some of the cements when good mixture proportions are specified (low maximum water/cement ratio). However, some of the cements have a high alkali content; when these are used with reactive aggregates, early age deterioration due to alkali silica reactions is a matter of concern (57). In addition, many of the cements exhibit high shrinkage compared with bridge deck concrete when used at manufacturers' recommended proportions (43, 44, 58).

Patching can also be done as a high-early-strength PCC overlay is placed. However, this option does not lend itself to a rapid repair because of the time required to prepare the deck surface and the cavities to be patched.

5.4.1.4 Polymer Concrete

Patching with polymer concrete has been found to be effective when the thickness of the patches is less than 0.8 in. (2 cm) (3). The surface to be patched must be sound and dry. The polymer is troweled into place so that edges may be feathered. A prime coat may or may not be required. A number of binders can be used (15, 50, 55).

a. Cement, sand, coarse aggregate and water are batched at the site

b. Typical patching operation

Figure 5.12 Use of a Prepackaged Rapid-Hardening Hydraulic Cement Concrete Material for Partial-Depth Patching on a Bridge Deck

Figure 5.13 Use of Special Blended Cement Ready-Mix Concrete to Replace a Segment of the Top Half of a Bridge Deck

5.4.1.5 Steel Plate over Concrete

Materials that develop strength slowly are usually easier to place, more compatible with the old concrete, and more economical than rapid-curing materials. Patching with materials that do not attain a high-early-strength can be done if the patched area is covered with a steel plate that prevents wheel loads from damaging the concrete. The technique has been used by the New Hampshire Department of Transportation, the District of Columbia Department of Transportation and the Buffalo and Fort Erie Public Bridge Authority.

5.4.1.6 Limitations

Asphalt patches have a short service life, typically less than one year. Most hydraulic cement concrete patching materials shrink more than bridge deck concrete. Less than optimum cure is usually achieved when the hydraulic cement concrete patches are placed with lane closures of less than 56 hours. Curing time increases as temperature decreases. Special cements must be used at temperatures below 55°F (13°C), and patching at temperatures below 40°F (4°C) is not usually done with hydraulic cement concrete (see table 5.1). Patches do not retard corrosion when critically chloride-contaminated concrete is left in place.

5.4.1.7 Estimated Service Life

Hydraulic cement concrete patches typically have a lower permeability to chloride ion than the concrete they replace. Their permeability does not usually control service life. Asphalt concrete patches tend to be more permeable, and as they absorb water, early failure is likely due to hydraulic pressure from traffic loads, the freezing and thawing of the water, or accelerated freeze-thaw damage to the material underneath. The skid resistance of the patches does not control service life because the skid resistance is usually the same as that of the concrete they replace so long as proper surface textures are applied.

Bond strength can control the life of all types of patches, and surfaces must be sound and properly prepared to provide high bond strength. It is believed that hydraulic cement concrete patches usually delaminate because of failures in the concrete below the bond line or adjacent to the patch. Service life estimates are shown in tables 5.2 and 5.7. From table 5.7, the life of an asphalt concrete patch is approximately 1 year and that of a hydraulic cement concrete patch approximately 10 years. Table 5.2 shows that hydraulic cement concrete patches can have a life of 25 years when critically chloride-contaminated concrete is removed before patching. The effect of concrete cover and chloride ion content on the life of the patch and the surrounding concrete is discussed in chapter 2. The old concrete rather than the patching concrete will likely control the time to corrosion of the reinforcing bars in decks with hydraulic cement concrete patches.

Table 5.7 Service Life of Patches Based on Questionnaire Response and Literature Review (Years)

Reference	System	Questionnaire Response			Literature Review		
		Avg.	Low	High	Avg.	Low	High
Transp. Research Board (51) U.S. Dept. of Transp. (52) Weyers, Cady, Hunter (35)	Asphalt concrete patch	1.7	1.0	3.0	0.6	0.1	1.0
New York State DOT (34) Weyers, Cady, Hunter (35)	PCC patch	5.9	1.8	10.0	14.8	4.3	35.0
Weyers, Cady, Hunter (35)	Polymer concrete patch	20.0	15.0	25.0	5.5	—	—
Weyers, Cady, Hunter (35)	Other hydraulic concrete patch	11.9	2.0	20.0	3.8	—	—
——	Steel plate over concrete	15.0	—	—	—	—	—

5.4.1.8 Estimated Construction Prices

The price for patching is dependent upon depth of the patch and the patch material. Prices are presented in section 3.3.1.

5.4.1.9 Construction Procedures

The successful application of a patch includes use of acceptable materials, adequate surface preparation, proper mixing and placement of materials, and adequate curing before subjecting the patch to traffic.

5.4.1.10 Materials

The contractor shall use materials that satisfy the requirements of the specifications. Hydraulic cement concrete mixtures should be proportioned for minimum shrinkage and required strength. Mixture proportions should be approved by the engineer.

5.4.1.11 Surface Preparation

- Asphalt Patches

  1. Remove all deteriorated concrete.
  2. Sandblast if possible.
  3. Airblast to remove loose material and dust.
  4. Tack surface.

- Hydraulic Cement Concrete Patches

  1. Mark areas so that all concrete with a half-cell potential more negative than -250 mV CSE is removed and areas are squared up with as few sides as possible (preferably four).

  2. Saw-cut surface to depth of 1 in. (2.5 cm) and avoid cutting reinforcing steel by measuring the cover depth before sawing.

  3. Remove concrete with hammers weighing less than 30 lb (13.6 kg) worked at a plane of 45 to 60 degrees to the plane of the concrete surface.

  4. Sandblast concrete surfaces to remove dust, loose concrete, and materials that may interfere with the bonding or curing of the patch. Sandblast to white metal all exposed reinforcing bars (take care to remove rust from the underside of the bars). Reinforcing bars that have lost more than one-fourth of their original cross sectional area should be lapped with new bars of the same size

and shape.

5. Airblast to remove dust.

6. Wet surfaces to achieve a saturated surface dry condition.

7. Airblast to remove puddles of water.

5.4.1.12 Methods of Mixing and Placing Materials

- Asphalt Patches

Materials shall be placed and compacted in accordance with specifications or manufacturer's instructions.

- Hydraulic Cement Concrete Patches

1. Place a portion of the concrete into the cavity, brush the mortar fraction onto the prepared concrete surface, and fill the cavity with concrete before the mortar dries.

2. Consolidate and strike off hydraulic cement concrete.

3. Texture the surface so that it matches the surrounding concrete surface. Typically, fresh concrete surfaces are tined or grooves are sawed into the hardened concrete surface for skid resistance.

4. Moist-cure with wet burlap and polyethylene as long as possible, except when using special materials such as magnesium phosphate concrete that are not designed to be moist-cured.

5. When temperatures are below 55°F (13°C), the patch shall be covered with insulating blankets.

6. Apply membrane-forming curing compounds at the rate of 1 gal (3.8 L) per 150 ft^2 (14 m^2) before opening the patch to traffic and before the surface of the patch looks dry if wet burlap is not applied.

7. The minimum acceptable curing time is the time required to achieve a compressive strength of 2,000 psi (13.8 MPa) before opening to traffic and 3,000 psi (21 MPa) at 24 hours.

5.4.1.13 Quality Assurance and Construction Inspection Program

Construct patches using the procedures described above. Table 5.8 shows typical properties of patching materials. Preapproved products lists can be used to identify materials that have worked in the past. However, materials and mixture proportions should be approved by the engineer. Materials should be checked for compatibility with substrate and overlay materials. Before placement, measure the slump (AASHTO T119), air content (AASHTO T152, T196 or T199), and temperature of the concrete (ASTM C 1064). Monitor concrete temperature, air temperature, relative humidity, and wind speed, and determine evaporation rate (ACI 308). Do not place hydraulic cement concrete if the evaporation rate is above 0.1 lb/ft^2/hr (0.5 kg/m^2/hr).

Prepare compressive test cylinders and measure strength at the age at which concrete will be opened to traffic and at 24 hours, 7 days, and 28 days (AASHTO T22, T23). Cores may also be tested for strength (AASHTO T24). Inspectors shall be certified in hydraulic cement concrete construction.

5.4.1.14 Material Specifications

Patching materials shall have the properties shown in table 5.8 (3).

5.4.2 High-Early-Strength Hydraulic Cement Concrete Overlays

5.4.2.1 Description

Hydraulic cement concrete overlays are placed on decks to reduce the infiltration of water and chloride ion and to improve the ride quality and skid resistance (16, 59, 60, 61). Overlays may also be placed to strengthen or improve the drainage on the deck. The overlays are usually placed with internal and surface vibration and struck off with a mechanical screed. The overlays usually have a minimum thickness of 1.5 in. (3.8 cm) for concretes modified with 15% latex by weight of cement and 2.0 in. (1 cm) for most other concretes, such as LSDC (see figure 5.14). These overlays are discussed in detail in chapters 3 and 4; characteristics related to high early strength are covered below.

Some concretes, such as those containing 7% to 10% silica fume, or special blended cements like Pyrament have permeabilities similar to latex-modified concrete and should perform adequately at a thickness of 1.5 in. (3.8 cm). High-early-strength hydraulic cement concrete mortars about 1 in. (2.5 cm) thick have been used as overlays, but these overlays tend to crack and do not provide much protection.

- Semirapid Overlays

More conventional high-early-strength portland cement overlays such as those prepared with

Table 5.8 Typical Properties of Hydraulic Cement Concrete Patching Materials

Coarse aggregate size	ASTM C 33
Thickness < 2 in. (5.1 cm)	#7, #8
Thickness ≥ 2 in. (5.1 cm)	#57, #7, #8
Fine aggregate	ASTM C 33
Cement content	≥ 635 lb/yd³ (375 kg/m³)
Water/cement ratio by weight	≤ 0.45
Slump (ASTM C 143)	≥ 3 in. (8 cm)
Compressive strength when opened to traffic (AASHTO T22)	≥ 2,000 psi (13.8 MPa)
Compressive strength at 24 hours (AASHTO T22)	≥ 3,000 psi (20.7 MPa)
Compressive strength at 28 days (AASHTO T22)	≥ 4,000 psi (27.6 MPa)
Bond strength (R3TM-3)[a]	≥ 2,000 psi (13.8 MPa)
Length change at 28 days (ASTM C 157)	≤ 0.05 %
Scaling resistance (R3TM-4)[a]	≤ 8%
Setting time (R3TM-1)[a]	≥ 10 minutes

[a] FHWA Region 3 Test Method

a. Mortar is broomed into saturated surface and concrete is consolidated and struck off.

b. Wet burlap curing material is applied immediately following the tining operation.

Figure 5.14 Placement of a High-Early-Strength Latex-Modified Portland Cement Concrete Overlay on a Scarified and Shotblasted Deck Surface types I and II portland cement and microsilica or type III cement and latex, can be constructed and cured with a lane closure of less than 56 hours (5). The deck can be patched before placing the overlay or as the overlay is placed. When the deck is patched before placing the overlay, a patching material must be selected that will provide for good bond between the patch and the overlay. Polymer concretes, polymer-modified concretes, and other concretes that can interfere with the adhesion of the overlay should not be used for patching before placing an overlay. The deck should be scarified, sandblasted (within 48 hours before application of overlay), sprayed with water, and covered with polyethylene to obtain a sound, clean, saturated surface dry condition (saturated concrete with no free water on surface) before placing the overlay (3, 5).

- Rapid, Very Rapid, and Most Rapid Overlays

Overlays can be constructed and cured to a strength suitable for traffic in less than 8 hours using special blended cements such as Pyrament; type III portland cement with admixtures such as corrosion inhibitors and high-range water reducers; and rapid-hardening cementitious materials that satisfy the requirements of ASTM C 928 (5, 11, 19, 21, 22, 54, 56). Because of the time required for concrete removal, decks are usually patched before the lane closure in which the overlay is placed.

Hydraulic cement concrete overlays can also be constructed with alumina cement and magnesium phosphate cement. The placement procedures described for other high-early-strength hydraulic cement concrete overlays would be generally applicable to these cements. Because of their rapid setting time, alumina cement and magnesium phosphate cement are usually sold in 50 lb (23 kg) bags as a rapid-hardening cementitious material (ASTM C 928) (10, 17, 62). A slower-setting hot-weather version of magnesium phosphate cement concrete can be mixed in a ready-mix truck and placed as an overlay. Surface preparation requirements are the same as for other high-early-strength hydraulic cement concrete overlays, except that (1) the deck surface should be dry and (2) scrubbing of the mortar fraction into the surface ahead of the overlay may not be necessary. These materials have the added advantage that they can be air-cured rather than moist-cured. Since these materials are typically used for patching, they are discussed in more detail in section 5.4.1.

5.4.2.2 Limitations

High-early-strength hydraulic cement concrete overlays, such as those prepared with 7% silica fume or 15% latex and type III cement, can be successfully placed and cured with lane closures of less than 56 hours and should have a service life not much less than that of conventional overlay installations (16, 60, 61). Considerable developmental work with materials and equipment is needed to overcome problems with the installation and the acceptance of the overlay concretes that can be installed and cured with lane closures of less than eight hours. Most hydraulic cement concrete overlay mixtures that can be opened to traffic in less than eight hours shrink more than bridge deck concrete. Also, less than optimum curing is usually achieved when the hydraulic cement concrete overlays are placed with lane closures of less than 56 hours.

Curing time increases as temperature decreases. Special cements designed to cure at low temperatures must be used at temperatures below 55°F (13°C), and placing overlays at temperatures below 40°F (4°C) is not usually done successfully with hydraulic cement concrete (see table 5.1). Overlays do not stop corrosion when critically chloride-contaminated concrete is left in place.

5.4.2.3 Estimated Service Life

The literature review revealed service life estimates for PCC overlays of 14 to 25 years with an average of 18 years (31, 36). The questionnaire response produced estimates of 10 to 23 years, with an average of 16 years. However, these estimates are not for high-early-strength overlays. The database is not adequate to predict accurately the service life of high-early-strength hydraulic cement concrete overlays. One of the first of these overlays to be installed and evaluated was in Virginia in 1986 and consisted of latex and type III cement (65). A similar overlay was placed in 1992 to rehabilitate the bridge over Pimmit Run on the George Washington Memorial Parkway (Project NPS-GWMPIA65). In 1990, a special blended cement concrete overlay was installed in Oklahoma (21); in 1991, a similar mixture was used to replace the top half of a deck in Virginia (19); and in 1992, a high-early-strength overlay consisting of silica fume and type II cement was installed in Virginia (Project 0726-015-6123, SR01).

Evaluations of high-early-strength latex-modified concrete overlays indicate that they should have a service life comparable to that of standard latex-modified concrete overlays, which is 22 to 26 years. Recent evaluations of overlays constructed with 7% and 10% silica fume suggest that these overlays should last as long as latex-modified concrete overlays (16, 60, 61). Data on permeability to chloride ion, skid resistance, and bond strength suggest that high-early-strength overlays should last as long as similar standard overlays. Because of the high alkali content of one special blended cement, a shorter life may be experienced because of alkali-silica reactions. High-early-strength hydraulic cement concrete overlays should be more susceptible to construction problems because of the low water/cement ratio and short working time. They should be susceptible to a shorter service life because of the potential for construction problems and the short curing time before opening to traffic. Properly constructed overlays should last 24 years (see table 5.2).

5.4.2.4 Estimated Construction Prices

Prices are presented in section 3.3.5.

5.4.2.5 Construction Procedures

The successful application of a high-early-strength hydraulic cement concrete overlay includes use of acceptable materials, adequate surface preparation, proper mixing and placement of materials, and adequate curing before subjecting the overlay to traffic.

- Materials

The contractor shall use materials that satisfy the requirements of the specifications. Hydraulic cement concrete mixtures should be proportioned for minimum shrinkage and required strength. Mixture proportions should be approved by the engineer.

- Surface Preparation

1. Use scarifiers, shotblasters, grinders, hydrodemolition equipment, or other concrete removal equipment to remove concrete to the depth required for the overlay.

2. Patch as described earlier, except saw cutting is not necessary when patches are placed before placing the overlay. (Patches may be placed as overlay is placed.)

3. Sandblast or shotblast concrete surfaces to remove dust, weak concrete, and materials that may interfere with the bonding or curing of the overlay. Sandblast exposed reinforcing bars to white metal.

4. Airblast to remove dust.

5. Wet surfaces to achieve a saturated surface dry condition.

6. Airblast to remove puddles of water.

- Methods of Mixing and Placing Materials

1. Brush the mortar fraction of the overlay concrete onto the prepared surface just ahead of the overlay placement.

2. Before the mortar dries place, consolidate, and strike off the overlay concrete.

3. Texture the surface by tining the fresh concrete or by later saw cutting the hardened concrete.

- Curing

1. Moist-cure as long as possible, except when using special materials such as magnesium phosphate concrete or concretes prepared with some special blended cements that are not designed to be moist-cured.

2. The temperature shall be 50°F (10°C) and rising when the overlay is placed.

3. Apply membrane-forming curing compound before opening the overlay to traffic and before the surface of the overlay looks dry (if wet burlap is not applied).

4. The minimum acceptable curing time is the time required to achieve a compressive strength before opening to traffic of 3,000 psi (20.7 MPa) at 24 hours.

5.4.2.6 Quality Assurance and Construction Inspection Program

Construct overlays using the procedures described above. Table 5.9 shows typical properties of overlay materials. Preapproved product lists can be used to identify materials that have worked in the past. However, materials and mixture proportions should be approved by the engineer.

Materials should be checked for compatibility with the substrate. Before placement, measure the slump (AASHTO T119), air content (AASHTO T152, T196 or T199), and temperature of the concrete. Monitor concrete temperature, air temperature, relative humidity, and wind speed and determine evaporation rate (ACI 308). Do not place hydraulic cement concrete if the evaporation rate is above 0.1 $lb/ft^2/hr$ (0.5 $kg/m^2/hr$).

Prepare compressive test cylinders and measure strength at age in which concrete will be opened to traffic and at 24 hours, 7 days, and 28 days (AASHTO T22, T23). Inspectors shall be certified in hydraulic cement concrete.

5.4.2.7 Material Specifications

Overlay materials shall have the properties shown in table 5.9 (3).

5.5 Rehabilitation Methods

Rapid rehabilitation methods include asphalt overlays on preformed membranes, described in chapter 3, polymer overlays, described in section 5.3.1; and high-early-strength hydraulic cement concrete overlays, described in section 5.4.2. For these methods, all actively corroding (more negative than -250 mV CSE) and critically chloride-contaminated concrete must be removed (see section 3.2.1). Details of these methods are presented in chapter 3.

Table 5.9 Typical Properties of Hydraulic Cement Concrete Overlay Materials

Coarse aggregate size	ASTM C 33
Thickness < 2 in. (5.1 cm)	#7, #8
Thickness ≥ 2 in. (5.1 cm)	#57, #7, #8
Fine aggregate	ASTM C 33
Cement content	≥ 635 lb/yd³ (375 Kg/m³)
Water/cement ratio by weight	≤ 0.45
Slump (ASTM C 143)	2 - 5 in. (5 - 13 cm)
Air content (ASTM C 231)	5% to 9%
Compressive strength when opened to traffic (AASHTO T22)	≥ 3,000 psi (20.7 MPa)
Compressive strength at 24 hours (AASHTO T22)	≥ 3,000 psi (20.7 MPa)
Compressive strength at 28 days (AASHTO T22)	≥ 4,000 psi (27.6 MPa)
Bond strength (ACI 503R, VTM92)	≥ 250 psi (1.7 MPa)
Length change at 28 days (ASTM C 157)	≤ 0.05%
Scaling resistance (R3TM-4)[a]	≤ 8%
Setting time (R3TM-1)[a]	≥ 60 minutes
Permeability to chloride ion (AASHTO T277) at 6 weeks	≤ 1,500 coulombs

[a] FHWA Region 3 Test Method

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6

Concrete Removal Methods

6.1 Introduction

This chapter is a guide for personnel who face the day-to-day task of introducing new and appropriate technologies for concrete removal on bridge repair and rehabilitation projects. The economics of concrete removal methods and combined methods are presented elsewhere (1). The chapter addresses partial removal and localized full-depth removal of concrete from decks and other components of concrete bridges.

The extent of concrete removal on a given bridge component can vary from large regular areas covering most of the component to small irregular patches in random locations. Repair and rehabilitation require that deteriorated and/or contaminated concrete be removed to a specified depth over a given area at a particular location on the bridge structure. The selection of a concrete removal technique depends on these three factors - depth, area, and location - as well as the method used to identify the concrete to be removed.

Four methods are generally used to identify concrete that must be removed as the first step in either repairing or rehabilitating an existing bridge. The first two, visual inspection and sounding, identify deteriorated concrete and rely on the fact that corrosion has reached the stage where cracking, delamination, or spalling are either visible or sufficient to produce a dull sound when the surface is dragged with a metal chain or struck with a masonry hammer. The other two methods, core sampling and half-cell potential measurement, are able to detect contaminated concrete and determine the risk of corrosion in the area of measurement.

The work required to remove deteriorated concrete (identified by visual inspection or sounding) differs from the work required to remove chloride-contaminated concrete (identified by core sampling or half-cell potential measurement) in two important aspects:

1. *The effort required.* Deteriorated concrete identified by inspection or sounding is relatively easy to remove due to preexisting cracks or delaminations.

Contaminated concrete identified by core sampling or half-cell potential measurement is more difficult to remove, as it may not be structurally damaged in any way.

2. *The area involved.* Visual inspection and sounding identify specific local areas of deterioration, whereas core sampling and half-cell potential measurements are normally taken as indicators of contamination over a large area.

The method used to identify the concrete to be removed influences the area of concrete to be removed in two ways:

1. Visual inspection and sounding (ASTM D 4580) identify deterioration in the concrete. The areas are likely to be small and irregular in shape as identified by the actual cracking, delamination, and spalling. The localized nature of the work limits the alternatives for concrete removal. The deteriorated concrete is, however, relatively easy to remove because of the existing cracks and fracture planes.

2. Half-cell potential measurements (ASTM C 876) and core sampling (ASTM C 42) can determine the presence of chloride-contaminated concrete. The results are generally seen as reflective of the overall condition of the structural component. Concrete removal that follows is normally performed in a systematic manner over a large area. Sound but contaminated concrete will be removed.

Concrete removal on a bridge deck differs substantially from concrete removal on other components. The flat horizontal surface of a deck provides easy access and permits use of high-production mechanized methods. Cleanup and containment of debris are also easier on the horizontal surface.

Other structural components have smaller surfaces that frequently are vertical or overhead and difficult to reach. Difficulties with access and debris containment preclude the use of heavy equipment and make concrete removal less productive and more expensive. The need for scaffolding to provide access for workers and equipment adds to the difficulty and safety hazards involved with the work.

The location of the work influences the area of removal in two ways:

1. Decks present large surface areas that are exposed to chlorides in deicing salts (to remove snow in northern climates) and sea spray (in a marine environment). This, together with exposure to traffic, means that areas of contamination and deterioration are likely to be large, with removal operations following a systematic pattern.

2. Other structural elements present small and irregular surface areas that are not uniformly affected by deicing salts and sea spray due to differing patterns of exposure and runoff. Areas of contamination and deterioration are thus likely to be small and irregular in shape.

The depth to which the concrete is removed has a profound effect on the method to be used and the cost of the work. The following classification, as shown in figure 6.1, is used throughout this chapter:

1. *Surface removal.* This is defined as the minimum amount of work needed to remove surface contamination and provide a clean, long-lasting bond between the existing material and the material used to repair or rehabilitate the bridge.

2. *Cover concrete removal.* Cover concrete is defined as the concrete that lies outside or above the first layer of reinforcing steel. The removal task does not involve any interaction with the reinforcement and is not hindered by its presence.

3. *Matrix concrete removal.* Matrix concrete is the concrete that lies around and just below the first layer of steel reinforcement. The removal tasks are severely hindered by the need to work in confined spaces around, below, and between individual bars. Contaminated concrete in this zone is thus extremely difficult to remove. The removal of deteriorated concrete is somewhat easier because of fracture planes caused by cracking and delamination. The depth of the zone is defined to extend a small distance (about 1 in. [2.5 cm]) below the steel to allow for the flow of replacement material into all the voids created.

4. *Core concrete removal.* Core concrete forms the core of the structural element and lies between the reinforced zones. The removal task is inhibited by the reinforcing steel that was exposed during the removal of the matrix concrete. Conventional cutting, grinding, and sawing techniques cannot be used. The quantity of material in this zone is dictated by the size and shape of the structural element. Thin deck sections contain little core concrete material, while piers and pile caps may contain a fairly substantial quantity. The volume of material to be removed is limited by the extent of chloride contamination.

Concrete removal tasks performed to repair or rehabilitate an existing reinforced concrete bridge are very different from the tasks performed simply to demolish the structure. This is because removal tasks must meet three quality constraints that do not apply to demolition: selectivity, residual damage, and bond quality.

In selectively removing concrete from bridge components, only the contaminated or deteriorated concrete and rusted reinforcing steel marked for removal must be removed. Removal in excess of the required minimum is expensive in terms of both removal and replacement cost and contributes nothing to the quality of the completed product.

Removal tasks must be performed in a manner that ensures that the remaining concrete and reinforcing steel retains its structural integrity. Equipment used to perform the work must not overload the structure, and care must be taken not to remove so much concrete that

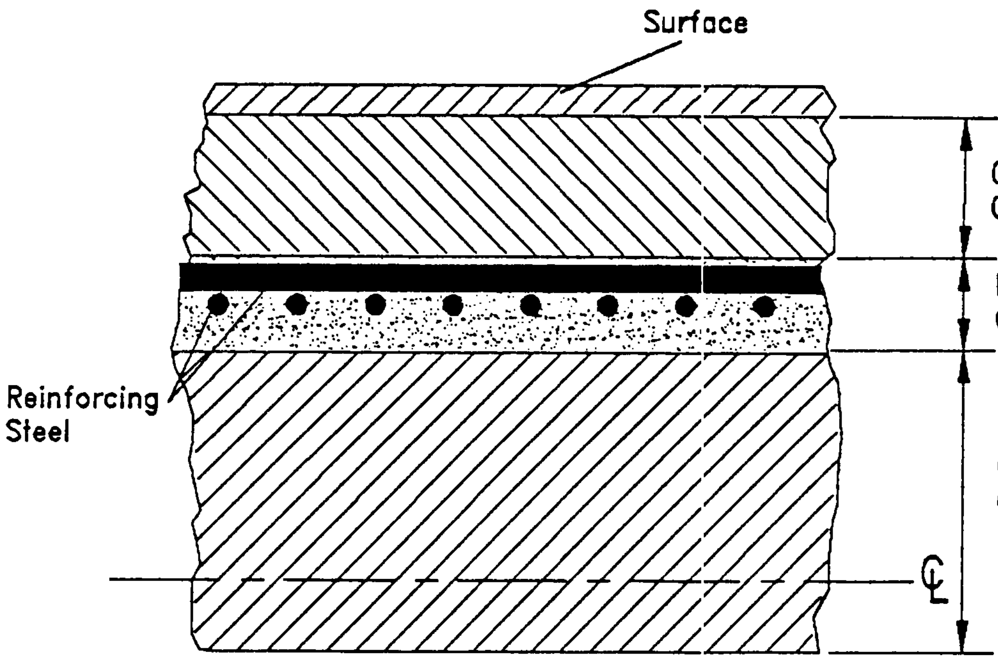


Figure 6.1 Depth Classification For Concrete Removal

the member being repaired or rehabilitated is critically weakened. Any impact forces used to remove damaged concrete should be applied in a manner that minimizes cracking in the residual concrete and minimizes damage to the bond between the remaining concrete and steel. Methods to remove rust and chlorides from the steel should also minimize damage and loss to the remaining steel.

Removal tasks are only part of the repair-rehabilitation process. Any new concrete needed to replace damaged and contaminated concrete will need to bond effectively with the remaining concrete and steel. Remaining surfaces must be clean and sufficiently textured to provide the required bond.

The quality constraints of selectivity, residual damage, and bond quality make removal a more demanding and expensive task than demolition. They also preclude the use of many high-impact, high-production techniques developed for concrete demolition and limit suitable techniques to those that comply with the constraints.

Methods that are available for concrete removal are classified according to the four removal depth categories: surface, cover, matrix, and core removal.

It is frequently necessary to remove surface contaminants such as oil, rubber, and rust from the work area in order to provide a sound, long-lasting bond between the existing structure and the new materials used to repair or rehabilitate the bridge. The objective is to clean rather than to remove material. The following four methods are frequently used:

1. *Scrabbling*. A scrabbler uses pneumatically driven bits to impact the surface to remove concrete to a depth of between 0.03 in. to 0.25 in. (0.1 cm to 0.6 cm). Scrabblers vary in size from large, self-propelled machines that can only work on large horizontal surfaces to small, hand-held tools for use in restricted, vertical, or irregular surfaces. Vacuum collection systems are frequently used to collect the concrete debris.

2. *Planing*. A plane or diamond grinder removes concrete by abrasion. Numerous diamond-tipped concrete saw blades are mounted close to one another on a horizontal spindle that is rotated to cut and remove up to 0.5 in. (1.2 cm) of concrete in a single pass. The process requires water to cool the blades, and the resulting slurry of concrete particles must be vacuumed up for collection and disposal.

3. *Sandblasting*. Sandblasters use compressed air to propel sand particles at high velocity. The impact of the particles produces a very abrasive action that cleans and roughens the exposed concrete or steel. The size and capacity of the equipment varies substantially. Small, hand-held tools are used on vertical or irregular surfaces. Vacuum systems are used to recover the sand and resulting debris.

4. *Shotblasting*. Shotblasters use a rotating paddlewheel to propel steel shot against the concrete surface at high velocities. The impact is capable of removing concrete to depths up to 0.5 in. (1.2 cm). The roughness of the substrate concrete is controlled by the selection of different shot sizes. Machines vary in size, but their use is limited to horizontal surfaces because a collection chamber must be used to control the rebounding shot. A vacuum system is used to pick up the concrete debris and steel shot, which are then separated so that the shot can be reused.

The removal of cover concrete over a relatively wide area is frequently necessary in bridge rehabilitation projects. The work involves removal to a depth less than the cover depth of the steel; thus no work between, around, or under the reinforcing mat is included in the task. Scrabbling, planing, sandblasting, and shotblasting can all be used in repeated passes to achieve the required depth. This is an inefficient use of these methods, and the only really effective way of doing the work when large areas are involved is by using a concrete milling machine. A milling machine removes concrete by the cutting action of numerous tungsten-tipped teeth mounted on a rotating drum or mandrel, as shown in figure 6.2.

Milling machines can achieve very high levels of production, but they can only be used on horizontal surfaces such as bridge decks. They are also only capable of removing concrete above the reinforcing steel, and severe damage can occur when milling teeth cut or snag the reinforcement. Despite these limitations, the use of milling for the removal of cover concrete is an emerging technology suited to the large volume of high-production work. See section 6.4 for details of the milling process.

Hydrodemolition can also be used as a high-production, equipment-intensive method for the removal of cover concrete. More often, hydrodemolition is used to remove cover and matrix

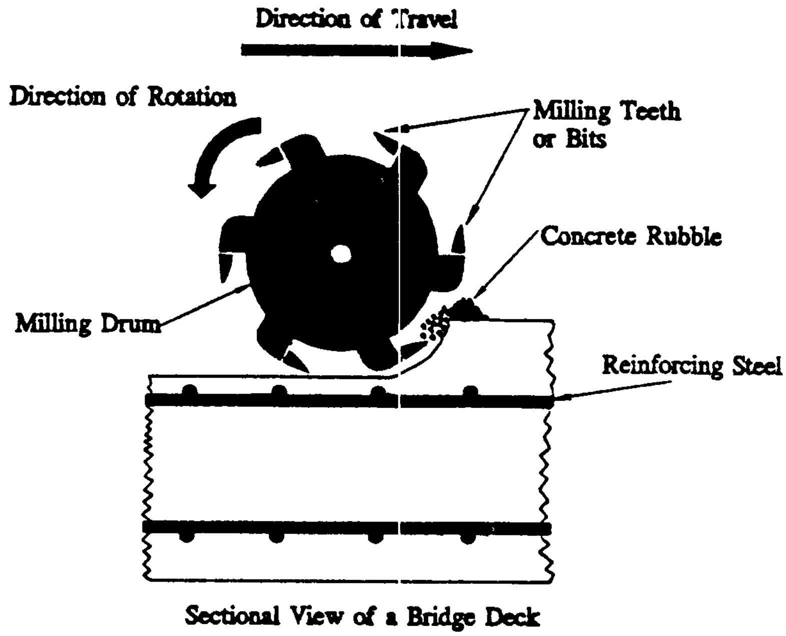


Figure 6.2 Action of a Milling Machine on a Bridge Deck

concrete simultaneously, and it has recently been used to following milling machines after they have removed much of the cover concrete.

Hand-held pneumatic breakers are frequently used to remove cover concrete when the areas involved are inaccessible to a milling machine or too small to be milled. As with hydrodemolition, hand-held breakers are primarily used for matrix concrete removal.

It is necessary to remove matrix concrete when contamination, spalling, and delamination have progressed into the concrete layer that surrounds and encases the upper reinforcing mat. The work involved is awkward; it must be performed between, around, and under the steel without damaging the steel, cracking the substrate concrete, or destroying the bond between steel and concrete in areas where the concrete is not to be removed. There are basically two methods available: pneumatic breakers and hydrodemolition. Both of these techniques can be used to remove cover and matrix concrete in a single operation, or they can be used to remove only matrix concrete after a more specialized and high-production method, such as milling, has been used to remove the cover concrete.

1. *Pneumatic breakers.* The pneumatic breaker (frequently known as a jackhammer but properly known as a paving breaker) is currently the most prevalent method for concrete removal in bridge repair and rehabilitation work. The breaker is hand-held and powered by compressed air to deliver a series of high-frequency blows that fracture the concrete in a small, easily controlled area. The production of pneumatic breakers depends on two factors: the size of the breaker and the skill of the operator.

   Breakers are sized according to their weight. This can vary from 8 to 120 pounds (4 to 55 kg) or more. Heavier breakers are more productive because they are able to impart more energy with each blow. They are also more destructive. SHAs typically limit the weight of breakers that can be used for selective concrete removal to less than 45 pounds (20 kg) to minimize residual cracking and preserve the bond between the residual concrete and the repair concrete in areas that are not removed. The angle of attack (from the breaker's axis to the concrete surface) is also frequently limited to 45 degrees for the same reasons. The skill of the operator is important with regard to both the quantity and quality of the work performed. This factor must not be overlooked when assessing the viability of the method. See section 6.3 for more details on the use of pneumatic breakers.

2. *Hydrodemolition.* Hydrodemolition is a high-production capital-intensive method for concrete removal. An extremely high-pressure water jet (12,000 to 35,000 psi [80 to 240 MPa]) is used to destroy the cement matrix of the concrete and liberate the aggregate. The equipment needed to generate the pressure and focus the water jet on the concrete to be removed is expensive and complex. The results under the right conditions can be impressive.

   The majority of hydrodemolition work involves the removal of matrix concrete on bridge decks. However, the equipment can be calibrated to remove concrete to almost any depth, and the nature of the process is such that there is an element of self-adjustment in depth depending on the soundness of the material encountered; it will even remove deteriorated concrete full-depth when such areas are encountered.

   Hydrodemolition can be used on inclined, vertical, and overhead surfaces, but cost-effectiveness is reduced by the inordinate cost of the specialized equipment needed to safely direct the jet and contain the debris when working on other than horizontal surfaces. See section 6.5 for more details on the use of hydrodemolition machinery.

6.2 Labor- and Capital-Intensive Operations

Of the three concrete removal methods, two, milling and hydrodemolition, are radical departures from the traditional labor-intensive pneumatic breakers method. These mechanized methods will reduce concrete removal cost. However, to harvest the cost-reduction savings, one must understand the cost-controlling factors of labor-intensive and equipment-intensive operations.

Labor-intensive methods are less complex from a technical point of view. They are likely to be tried, tested, and accepted by owner and contractor. The technical risks involved when using a labor-intensive method, such as pneumatic breakers, for removing deteriorated concrete are minor. Everyone knows that the method will work and that the required quality constraints will be met if the work is done with reasonably qualified operators. Specifications have been developed with this method in mind, and inspectors have substantial experience in accepting or rejecting the work.

The low technical risk associated with labor-intensive methods is matched by low and easily managed risks from a scheduling and works planning point of view. Labor is a versatile resource that can be deployed on many operations, and thus the need for careful planning to ensure continuity in the performance of a given task is not critical. This is important when working in congested areas, when access to the job site is difficult, and when the extent of deteriorated concrete is not well known. Under these conditions, the versatility of a labor-intensive operation is important and can play a significant part in achieving satisfactory results.

The flexibility of labor-intensive operations is again important when considering economic risk. No major capital investments requiring monthly interest and redemption payments are involved. Most of the contractor's costs will vary in direct proportion to the quantity of work done. This is well suited to the method of payment used in unit price contracts, where the value of work is also proportional to quantity. Risk in repair or rehabilitation contracts where the quantity of concrete removal and bar cleaning is not well known is thus substantially reduced by using labor-intensive methods.

The three areas outlined above, technical risk, flexibility in planning, and the economic risk associated with variations in quantity, favor the use of labor-intensive methods. But many concrete removal tasks require substantial amounts of energy and are expensive, slow, and physically demanding if technology and mechanization are not used to advantage.

Equipment-intensive methods such as hydrodemolition and milling rely on the use of large, sophisticated machines to apply the energy needed to remove concrete. The production attainable on a given area of bridge deck in a given period can be very high. This reduces construction time and results in shorter delays to the traveling public. Economies of scale are possible and, under proper conditions, mechanization can significantly reduce concrete removal costs. The machines are less physically demanding to operate than hand tools such as pneumatic breakers, and the work is safer because fewer people are employed in the hazardous work zones associated with bridge repair and rehabilitation projects.

These factors, particularly reduced construction time, count heavily in favor of mechanized methods. There are, however, significant risks for the contractor. The first of these is a technical risk: Mechanized methods such as hydrodemolition have only recently won general acceptance. This means that the technology has yet to mature with regard to the mechanical reliability of the equipment and the operational techniques used in different applications. It also means that specifications suited to mechanized methods are not as yet generally available and that inspectors have little background to draw on when accepting or rejecting the work.

The second major risk relates to flexibility of operational planning. The equipment used in mechanized operations is specialized and not suited for use on any other type of work. This means that the work must be carefully planned and sequenced if the equipment is to be kept productively employed and if schedules are to be met. This is frequently very difficult on bridge repair and rehabilitation projects where traffic control and construction phasing can cause equipment to stand idle for substantial periods resulting in average production rates significantly lower than the maximum that can be achieved.

The substantial investment required for mechanization introduces a significant economic risk. Capital costs are proportional to the time taken to complete a task, not the amount of work done. Continuity of operations and productivity thus become extremely important, as does the quantity of work that must be done with a particular machine on a particular contract. This causes mechanization to be a high-risk choice in contracts where the quantity of deteriorated concrete to be removed is not well known and where variations are expected.

The contract, and the manner in which it allocates risk between the owner and the contractor, establishes the framework within which the contractor makes the decisions needed to perform the work at a reasonable balance between risk and reward. The owner sets the contractual requirements and is thus in a position to create a contractual environment that either enables or inhibits mechanization from achieving its full potential. Aspects that merit attention include funding levels, continuity of the work, size and scope of the project, project location, traffic control, quality standards, and inspections.

The timing of maintenance operations can be delayed or accelerated over a fairly wide range, depending on the availability of funds and other macroeconomic issues. This results in a situation where the volume of work let on contract fluctuates substantially from year to year. It is thus all but impossible for owners and contractors to manage resources in an economical manner over an extended planning horizon. Fluctuations in work load have a particularly adverse effect on the confidence needed to make major capital investments in equipment, and much can be done to promote the use of mechanized methods by providing competitive contractors with a steady stream of work.

The size and scope of projects have an effect on the selection of the appropriate method for concrete removal. Most large projects include demolition tasks, such as the removal of existing sidewalks and barriers, as well as concrete removal tasks for rehabilitation and repair. The size, scope, and pace of the concrete removal work itself is thus a better determinant of the appropriate method. The following examples show the range of options available:

1. If the project requires that all contaminated and deteriorated concrete be removed over the full width and length of the deck, and if this work dictates the critical path of the project, then a high-production mechanized method such as hydrodemolition should be used.

2. If the project requires that sidewalks and handrails be demolished and new barriers be installed to bring the bridge up to current standards, and that localized areas of deterioration be repaired as a parallel operation, then a low-pace flexible method based on the use of pneumatic breakers should be employed to ensure that the main critical path activity can proceed unhindered.

Concrete removal is never more than part of a larger process and it is important for SHAs and contractors to keep this in mind when specifying and selecting appropriate mechanized methods.

Rehabilitation and repair projects require that sections of the bridge be closed and that regular traffic patterns be disturbed. This causes distress to the motoring public and increases the risk of accidents. This is particularly true when projects are located in heavily trafficked urban areas and when procedures needed to control traffic and ensure safe flows become complex and expensive. Mechanization affects both the area that must be closed to traffic and the total amount of time required to perform the work.

1. *Area*. Mechanized methods require that substantial portions of the structure, particularly the deck, be closed and made available to both production and support equipment. The benefits of mechanization cannot be achieved if this is impossible. More expensive labor-intensive methods become viable if working space is not available in a manner that ensures continuous operation.

2. *Time available*. Mechanized methods are able to achieve high production under the right circumstances. They are thus suited to projects where traffic and weather requirements dictate a high pace of work. Labor-intensive methods become more attractive when the tempo of work is not high and when traffic control requirements are such that continuous operation cannot be maintained.

High fixed costs are associated with owning the equipment needed to remove concrete using high-production mechanized methods. This means that contractors face very high risks if construction schedules are delayed or if work cannot be performed continuously. Rehabilitation and repair projects are particularly difficult to manage in regard to these two factors because of traffic control problems and because so many unforeseen factors become known as work proceeds. Successful use of mechanized methods requires that contractors and SHA project managers work together to schedule work and resolve variations. Contract conditions that require that the contractor assume all schedule risk, regardless of changes in access and working hours, will inhibit the use of mechanized methods.

The sensitivity of equipment-intensive methods to changes in the quantity of work performed means that mechanization is a very high-risk choice in contracts where the quantity of concrete to be removed is not well known.

As with scheduling and continuity, the risk of quantity variations does not lie with the contractor. The classic unit price methodology should be amended to share the risk between contractor and SHA in a more equitable manner, including the following amendments:

1. The provision of monthly or weekly pay items to cover fixed costs.

2. Clear provisions for change in unit rate with change in quantity.

Labor-intensive methods produce quality of a different type than that produced by mechanized methods. Pneumatic breakers and saws in the hands of skilled operators can produce work that follows lines, levels, and tolerances precisely. Mechanized methods such as hydrodemolition cannot do this. Much is lost if, for instance, the specification requires that peaks behind the rebar be removed to produce a smooth surface; in such a case, both hydrodemolition and pneumatic breakers would have to be used at a substantial increase in cost. Tolerances and inspection expectations must be changed if mechanized methods are to achieve their full potential. Designs and specifications must be based on realistic quality standards at levels that can be met by mechanized methods.

The contractor organizes and manages of the work, and the skill with which the various operations are managed has a profound effect on a given method for concrete removal. Mechanized methods employing specialized equipment are less flexible than labor-intensive methods. They therefore place a high demand on the contractor's ability to manage the work and create an environment in which the methods can achieve their full potential. Old approaches that rely on improvisation and on-the-spot reallocation of labor will simply not suffice when faced with the challenge of using new technologies such as milling and hydrodemolition. Four issues need to be addressed if mechanization is to be successful. These are works planning, specialization and subcontracting, manufacturer relationships, and owner relationships.

Mechanized methods have the potential of achieving very high rates of production.

However, these can only be achieved if the work is planned in detail and if all the required resources are available. This planning must be done within the confines of the project location and traffic control constraints and must provide for continuity of work.

Works planning must include all the repair and maintenance operations needed to ensure high levels of reliability and availability in the equipment used. Spare parts, consumables such as fuel and oil, and wear items such as cutting teeth and nozzles must be kept in stock and skilled personnel must be available. This is particularly true in concrete removal operations where the equipment works at high levels of stress in an abrasive environment.

Most bridge rehabilitation projects involve a mix of tasks ranging from the removal and replacement of bearings to deck rehabilitation and the replacement of signs and lights. This mix of work is usually done by multi-skilled crews who switch from one task to another as required by the status of the project. This flexibility is not possible when using milling or hydrodemolition equipment, and specialized crews or subcontractors are therefore frequently employed to perform the concrete removal tasks in one or more periods of intense activity. Specialized crews or subcontractors use the equipment more effectively. In addition, operating and mechanical maintenance skills are retained within the specialized unit, resulting in higher availability. Contractors must develop the skills needed to manage specialized crews and subcontractors. Poor planning resulting in stop-start operations will negate the advantages and result in high mobilization costs as crews and equipment are moved on and off the project.

Milling and hydrodemolition have been identified as two emerging technologies capable of improving concrete removal. The fact that they are emerging rather than established technologies means that developmental work is still required to improve the operational characteristics and reliability of the equipment used. Contractors who wish to implement these technologies, particularly hydrodemolition, should improve communication and liaison between themselves and manufacturers to expedite the full commercialization of these technologies. Manufacturers should be encouraged to participate in the process and continue to work with contractors as suppliers and operators of equipment rather than vendors. A change in traditional commercial relationships is needed, and contractors should encourage the manufacturers' involvement in field operations.

The successful implementation of new and innovative technologies in construction requires a special relationship between owner and contractor. Traditional contract forms as referenced by lump sum or unit price contracts are based on the assumptions that the nature of the work is fully known and that the methods used are tried and tested. Neither of these assumptions is true for most bridge repair and rehabilitation projects, thus traditional contract forms frequently give rise to variation and dispute.

6.3 Pneumatic Breakers

Hand-held pneumatic breakers are widely used and well-established tools for removing contaminated and deteriorated concrete. Their light weight and excellent maneuverability are suited to remove damaged concrete from small, isolated areas and from vertical and overhead surfaces on all bridge components. They can be used on cracked, spalled, or delaminated concrete, and on chloride-contaminated concrete when the depth of removal is known from the evaluation of the structure and indicated on the plans.

6.3.1 Description and Equipment

This section provides a basic technical description of the equipment, components, and operating parameters associated with concrete removal operations that use pneumatic breakers. An understanding of the equipment and equipment components is a first step in developing an appreciation for the technology.

Pneumatic breakers are typically classified by their weight, even though breakers of a similar weight do not necessarily generate the same impact force. Available weights range from under 20 lb (9 kg) for small chipping hammers suited for light-duty applications to just under 100 lb (45 kg) for large-production breakers. Small chipping hammers are limited to less than 45 lb (20 kg) so that they can be safely used on vertical or overhead surfaces. The upper weight limit for large-production breakers is governed by the quality requirements of the job and the weight that can be handled by the operator with ease and safety on horizontal surfaces. Figure 6.3 presents a schematic diagram of a typical pneumatic breaker.

The percussive force used by pneumatic breakers to fracture concrete is primarily determined by the impact energy and the frequency at which the impacts occur. The impact energy is based on the mass of the piston, the size of the cylinder, and the inlet port diameter. Impact energy ranges from approximately 15 lb (7 kg) per blow for small tools to more than 180 lb (82 kg) per blow for large tools. The frequency of impact, or blows per minute, ranges from 900 blows per minute to more than 2,000 blows per minute, depending on the valve design.

The air consumption of a pneumatic hammer can be directly calculated as the cylinder capacity times the strokes per minute. Air consumption generally ranges from approximately 35 ft^2/min to just over 70 ft^2/min (1 m^3/min to 2 m^3/min). The air pressure required by most pneumatic breakers is between 60 and 90 psi (420 kPa to 620 kPa).

6.3.1.1 Cutting Tools

Various cutting tools are available for use with hand-held pneumatic breakers. The shank end, which is inserted into the tool-retaining mechanism, is common to all. The cutting or working end can vary from a broad spadelike blade to a sharp well-honed point. The vast majority of concrete removal work is done with a pointed tool, although a relatively narrow (3 in. to 4 in. [7.5 cm to 10 cm]) blade-type tool is sometimes used to remove cracked and

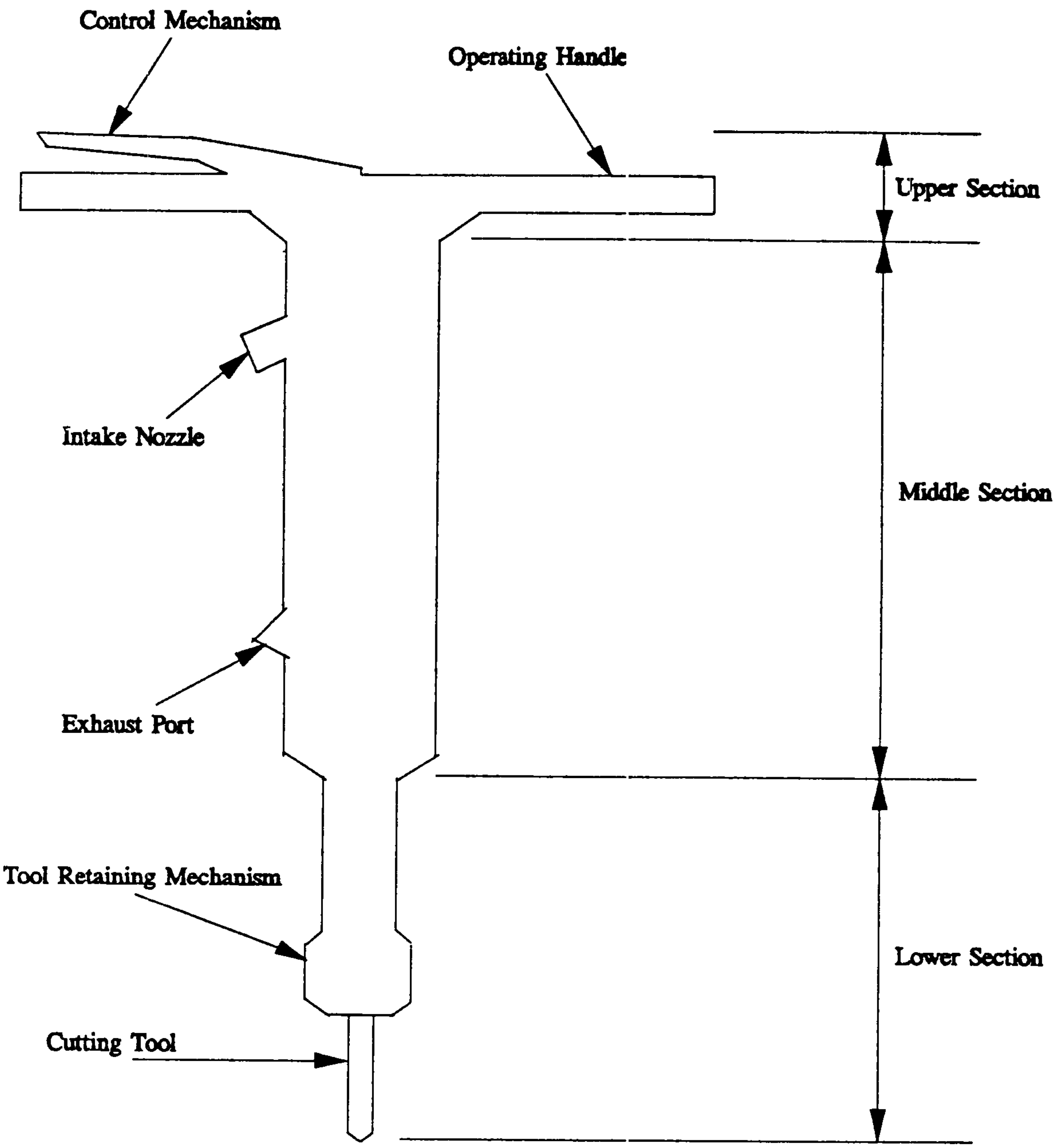


Figure 6.3 Pneumatic Breaker Components

deteriorated concrete. Proper maintenance of the cutting tool is important to the productivity and quality of the work.

6.3.1.2 Compressors and Distribution

The volume and pressure requirements of the compressor can be determined as the aggregate of the demands of all the tools to be used. The maximum air requirements for all the tools will rarely be needed; thus a single compressor can supply air to tools whose aggregate total demand exceeds compressor capacity by up to 20%. However, the tools will generally not receive the full pressure and quantity of air generated by the compressor due to losses throughout the air distribution system. Hose friction, bends, valves, and air leaks will all reduce the pressure available to the tools.

6.3.2 Work Characteristics

This section focuses on the primary applications for hand-held pneumatic breakers and describes the characteristics that constitute an efficient project. An understanding of these characteristics will ensure that pneumatic breaker operations achieve their full potential.

6.3.2.1 Project Type and Location

Breakers are generally used to varying degrees on all types of bridge repair and rehabilitation projects. The project type and location for which breakers are most effectively used is determined by the location of the concrete to be removed, the location of the bridge, the pace of the project, and the availability and cost of labor. Each of these is discussed in turn.

1. *Location of removal.* Breakers are used as the primary method of concrete removal on projects involving patching, on projects requiring the removal of concrete from bridge components that are not accessible to larger pieces of equipment, and on projects where concrete must be removed in small areas from between, around, and below the reinforcing. They are also used in support of large, high-production, equipment-intensive methods on most rehabilitation projects.

2. *Bridge location.* Breakers are ideally suited to operate in congested urban areas with high traffic volumes. In these areas their advantages over other methods in terms of setup time and space become apparent. Thus they are well suited to jobs that require many mobilizations or where the available hours of operation are limited.

3. *Project pace.* Many jobs allow traffic to be closed off for only short periods of time due to traffic patterns or safety considerations. Because of their small size and high degree of mobility, breakers are well suited to projects that have a limited working window. If time is limited or if there is a large quantity of

material that cannot be removed by other methods, breaker production can be increased by adding additional crews.

4. *Availability of labor.* Breakers are a labor-intensive method of concrete removal, sensitive to both the cost and the availability of labor. Urban areas generally have an abundant labor supply but also have higher wage rates. Rural areas typically have lower wage rates, but laborers may have to be sent to the job site because of an insufficient supply of local labor. The criticality of the labor supply is largely dependent on the size of the job and the number of crews required.

6.3.2.2 Type and Extent of Deterioration

Breakers are primarily used to remove small, isolated areas of deteriorated concrete from the bridge deck or other bridge components. They are one of the few methods available for removing concrete from vertical and overhead surfaces such as beams, girders, and piers. A smaller chipping hammer weighing less than 45 pounds (20 kg) is typically used for these applications. The weight of the tool will, however, hinder the operator when working on vertical or overhead surfaces. A system of scaffolding and lights may be required to gain access and visibility when working on bridge substructure components, and nets or other safety devices may be required to contain debris.

A major advantage of using breakers with skilled operators on bridge repair jobs is the ability of these operators to selectively remove only the damaged concrete. A skilled breaker operator can differentiate between various levels of concrete deterioration by the resistance of the concrete to the breaker. Because the operator can selectively remove only concrete that is deteriorated, the quantity of sound material removed is minimized. Quantity variations are, however, not eliminated because the actual area of deterioration frequently differs from the quantity originally estimated.

6.3.2.3 Material to Be Removed

Breakers are best used for removing concrete that is cracked or delaminated, as the fracture planes caused by the damage can be used by the operator to increase production. However, breakers are not limited to removing only deteriorated concrete and are often used to remove contaminated or sound concrete. They are also frequently used in support of other methods such as milling or hydrodemolition.

The properties of the concrete being removed and the characteristics of the individual type of breaker affect production and economy. No definitive data are available because different combinations of impact energy and frequency of impact produce different results on concrete with a given strength or aggregate type.

6.3.2.4 Area of Removal

Breakers are typically used to remove small areas of concrete. The small size of the removal area is perhaps the primary factor defining a breaker's ideal work environment. The tool's geometry and the economics associated with the use of small versatile tools make breakers most efficient in such an environment.

Large-equipment-intensive methods are generally restricted to removing concrete from bridge decks unless the equipment is greatly modified. These methods also are limited to removing concrete from areas that have definite boundaries and dimensions as dictated by the geometry of the machine. Breakers have no such size limitations and are therefore capable of removing small and irregular sections of pavement. The small size and light weight of breakers also make it possible to work in areas where access is difficult and where maneuverability is limited.

6.3.2.5 Depth of Removal

The abilities of a breaker to work in confined areas are best used if concrete must be removed around, between, and under the reinforcing steel. The small cutting tool allows the breaker to effectively remove concrete from confined spaces in matrix or core concrete. Special caution must be exercised when operating in these areas to avoid damaging the reinforcing steel or destroying the bond of the reinforcing steel in the adjacent concrete. Production is generally much lower when removing concrete in these regions because of the extra time and effort required.

6.3.2.6 Debris Removal and Cleanup

The debris generated from the breaker operations consists of pieces of concrete and aggregate in a variety of sizes. The larger pieces can be removed by hand and loaded into a wheelbarrow or a loader bucket. The small pieces and dust can then be blown away with an air wand. Before patching, the entire surface must be sandblasted to clean residual concrete and rust from the reinforcing steel. Disposal of the debris generated from breaker operations is generally not a major concern because the debris is readily accepted by most materials processing centers or dump sites.

6.3.2.7 Theoretical Production Rate

The theoretical production rate is the estimated rate at which the breaker can remove deteriorated concrete under ideal conditions with no delays or hindrances. The two primary inputs to this estimate are the size and weight of the tool. A tool with a greater weight will generally have a larger air cylinder and a piston of greater mass to produce higher impact energy. Not all tools of a similar weight will perform similarly due to differences in geometry and construction.

6.3.2.8 Modified Production Rate

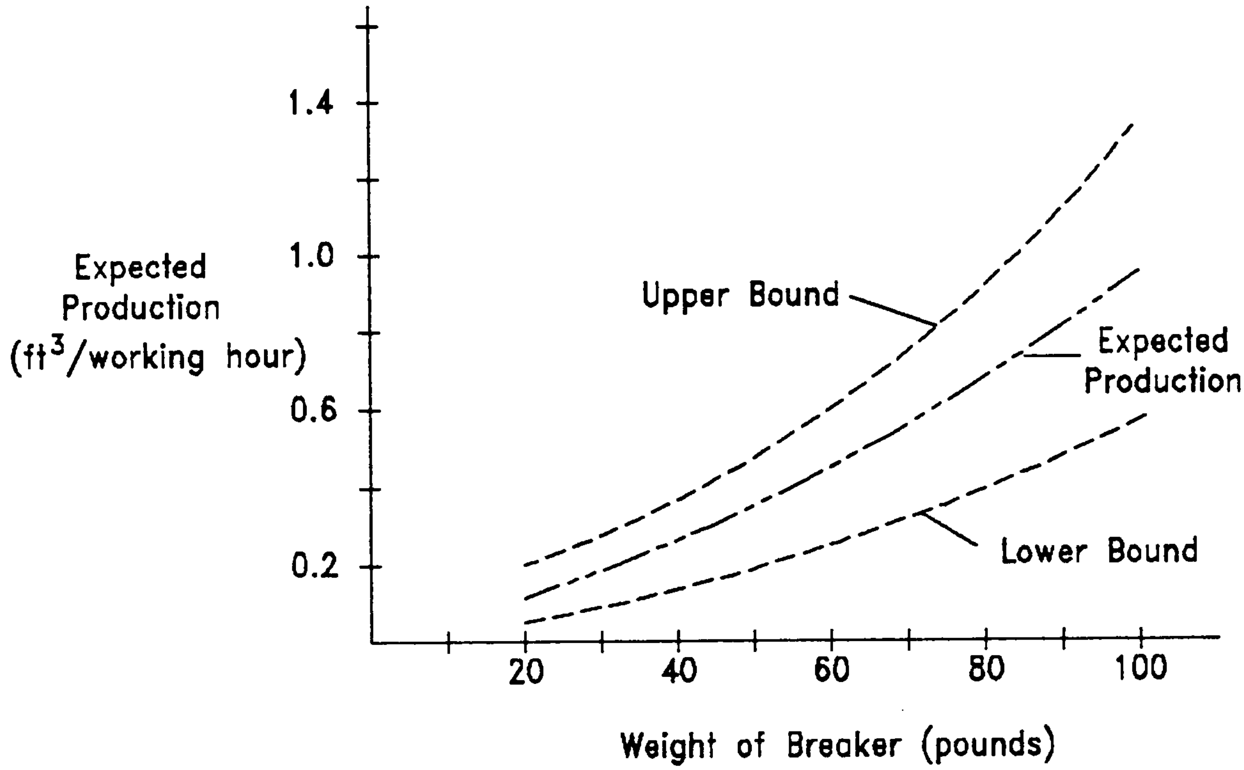
Several factors and conditions reduce the theoretical rate of production. Primary among these are the location, depth, and nature of the concrete to be removed, the level of deterioration in the concrete, interference from reinforcing steel, operator experience, and equipment reliability. The area of removal will generally not have a significant effect on individual breaker production rates. Breakers are well suited for jobs with small, noncontinuous areas of removal because of their small size and high mobility.

The relationship between these factors and the theoretical production is not clear and certainly not linear. Figure 6.4 graphically illustrates this relationship. In the figure, the upper bound is achievable when shallow horizontal areas of relatively weak or deteriorated cover concrete are removed by experienced operators. The lower bound represents a situation where solid contaminated matrix or core concrete is to be removed in awkward locations with less proficient operators.

6.3.3 Managing and Controlling Quality

Parameters for the success of the project must be established and maintained to ensure that breaker operations achieve the desired level of quality. Five quality concerns are imperative to success:

1. *Complete removal of deteriorated concrete.* It is essential that the breaker operations remove the concrete from the necessary depth and area to ensure that all deteriorated concrete is removed. Because a properly skilled breaker operator is capable of selectively removing only the deteriorated concrete, the actual quantity of concrete removed may vary from the estimated quantity of removal. In such a case, the amount of deteriorated concrete may have been over- or underestimated. The emphasis of the removal operations should be to ensure that all the deteriorated concrete is removed regardless of how closely it corresponds to the area and estimated depth delineated for removal.



(1 ft^3 = 0.028 m^3; 1 lb = 0.45 kg)

Figure 6.4 Pneumatic Breaker Rate of Production

2. *Damage to residual concrete.* The impact forces used to fracture and remove the deteriorated or contaminated concrete may produce microcracks in and immediately below the surface of the residual concrete. These cracks accelerate the deterioration of the residual concrete and weaken the bond between the residual material and the overlay or patch material.

   The extent of microcracking is determined by the magnitude and direction of the impact force. To control the situation, SHAs limit the weight of breakers used to remove concrete from bridge decks to 35 lb (16 kg) and specify that the impact angle must be between 45 and 60 degrees from the impact surface. Adherence to these limits is important.

3. *Damage to reinforcing steel.* The percussive force used by the breaker to fracture the concrete often damages the reinforcing steel or the bond between the concrete and the steel. If the cross-sectional area of the reinforcing bar is substantially reduced, either due to gouging caused by the breaker or by corrosion, then the entire damaged section of the bar should be removed and

replaced. If the concrete is removed around the steel, then the work should continue for an additional distance to ensure that there is sufficient surface area on the steel to form a bond and there is adequate space below the rebar for the coarse aggregate in the patch material. *All steel surfaces must be clean and free of chloride-contaminated rust and cement paste.*

4. *Surface characteristics.* The surface produced by the work must have the necessary characteristics for it to effectively bond with the replacement material. The breaker operations produce a rough, textured surface that is very uneven and irregular. This texture bonds well with patch or overlay material, but it is not suitable to be opened to traffic prior to resurfacing.

5. *Environmental concerns.* Effects of the breaker operations must be monitored to ensure minimal impact on the surrounding environment. The primary environmental issues of concern are dust, noise, and flying debris created both by the breaker operations and from the subsequent debris removal process.

   Safety standards require that operators wear steel-toed boots, hard hats, ear protectors, and goggles. Passing traffic and the public must be protected by the provision of all necessary barricades and screens. Noise suppression devices on the compressors and the breakers themselves should also be used to maintain acceptable safe noise levels (2).

6.4 Milling

Milling is a capital-intensive method of concrete removal that uses high-production machines to strip contaminated and deteriorated concrete from above the reinforcing steel. Milling machines are ideally suited to bridge deck rehabilitation projects requiring the removal of large volumes of concrete from above the reinforcing steel. Their inability to remove deteriorated concrete from below the reinforcing steel or from inaccessible areas, such as at joint faces and drains or around other obstacles, means that methods such as pneumatic breakers are invariably required to support the operations and complete the detail work.

6.4.1 Technical Description and Equipment

This section describes the components of a milling machine and their function in relation to the concrete removal operation. Figure 6.5 highlights the key components of a typical machine.

6.4.1.1 Cutting Mandrel

The cutting mandrel is a cylindrical metal drum mounted horizontally on the underside of the milling machine. It carries the cutting teeth used to break the pavement. Its width, which varies from a few inches on very small machines to more than 12 ft (3.6 m) on the largest machines, dictates the width of the material cut.

Cutting mandrels also vary in diameter, ranging from about 8 in. (20 cm) to more than 4 ft (1.25 m). The smaller mandrels generally rotate faster, using speed to cut the pavement, whereas larger mandrels tend to rotate more slowly, using the weight and horsepower of the machine to break the pavement.

Carbide-tungsten tipped cutting teeth (figure 6.6) are used to break the pavement. The teeth are usually slightly over 3 in. (7.6 cm) long. About half of this length is made up of the mounting shaft, while the other half comprises the conical holder and carbide-tungsten tip. Tip life varies from as low as four operating hours to a maximum of 24 operating hours.

The teeth are secured to the cutting mandrel through blocks that are either bolted or welded to the mandrel. A typical mounting block is illustrated in figure 6.7. The mounting block is designed to hold the mounting shaft in a manner that makes changing teeth as simple and fast as possible. Changing teeth once a day is typical for bridge work, but the frequency with which teeth will need to be replaced depends upon the amount of work done and the hardness of the material being removed.

The configuration of the teeth on the mandrel plays an important role in the operation of a milling machine. The teeth are mounted on the cutting mandrel in a spiral that runs inward from the sides. This directs the cut material toward the center, where it can be either loaded onto a conveyor for removal or left to be removed by other methods.

The spiral usually wraps the drum between one and three times with teeth staggered to strike the pavement at 0.5 in. (1.2 cm) intervals. The teeth are mounted at a slightly skewed angle so that they rotate as they travel through the pavement to wear evenly for maximum life.

The mandrel drive system rotates the mandrel in a direction opposite to the direction of travel of the machine, as illustrated in figure 6.8. This causes the cutting teeth to strike the pavement forward and up in order to fracture the concrete in tension. This is not only more efficient but also reduces cracking and other damage to the substrate concrete.

6.4.1.2 Depth Control and Leveling

A depth control mechanism is needed to control the depth of cut as required by the demands

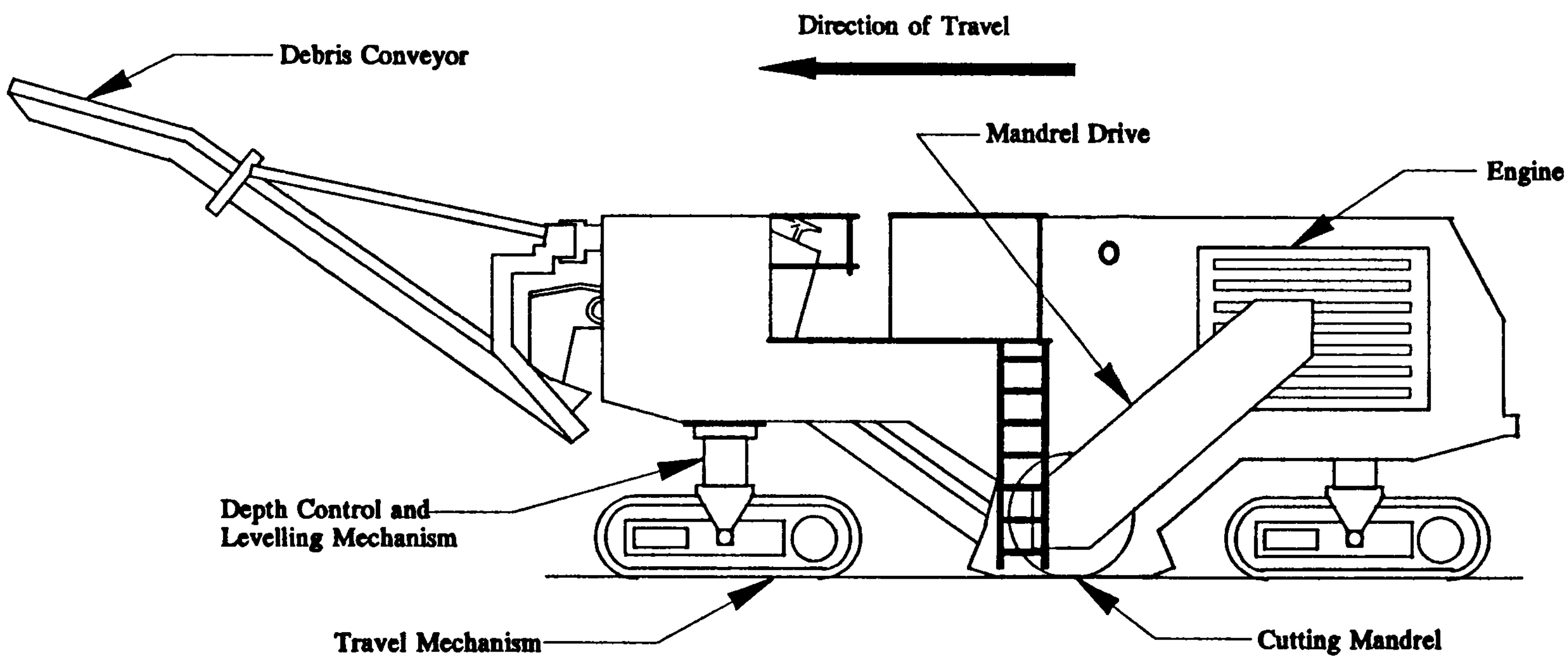


Figure 6.5 Milling Machine Components

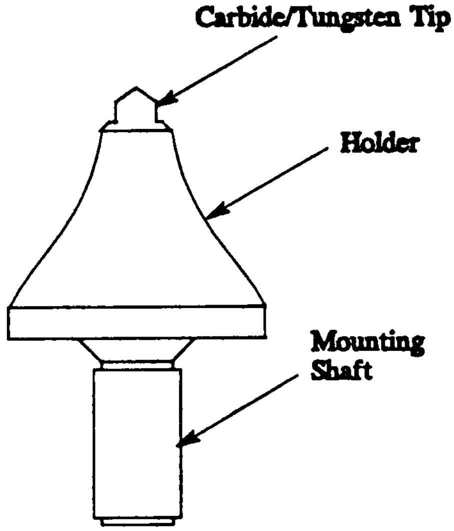


Figure 6.6 Carbide-Tungsten-Tipped Cutting Teeth

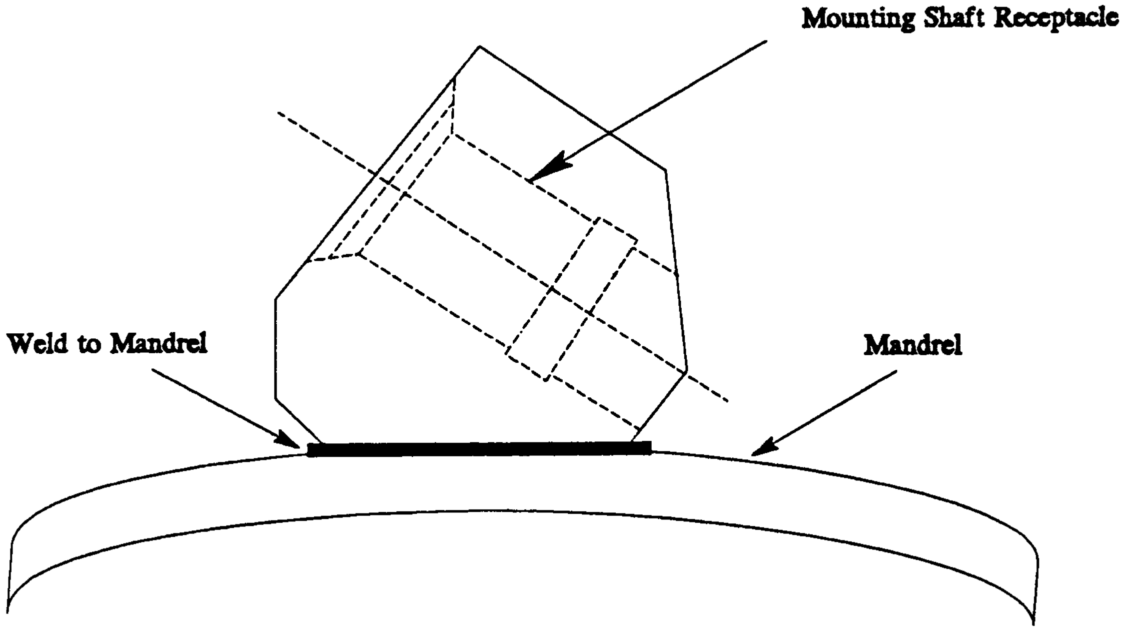


Figure 6.7 Mounting Block

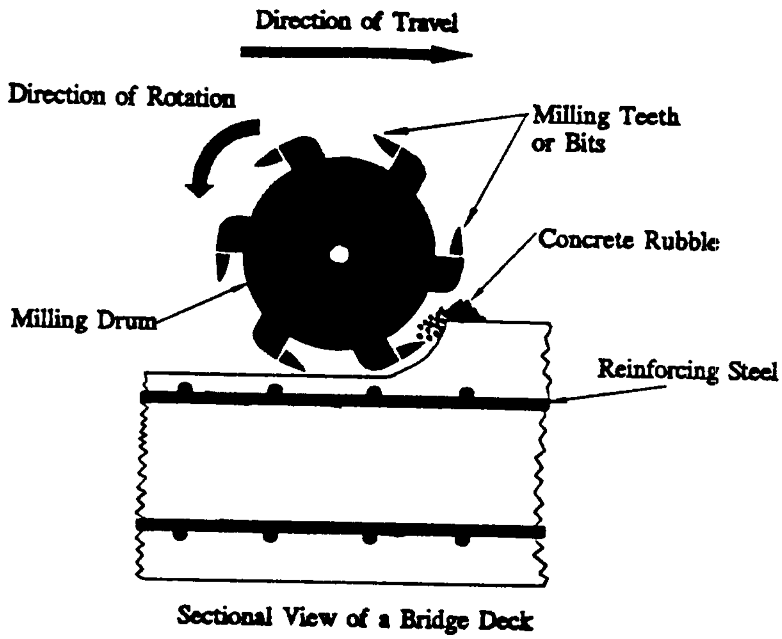


Figure 6.8 Cutting Action of a Milling Machine

of the job. Milling machines are limited to the removal of cover concrete, hence the depth of steel limits the maximum depth of cut. Large, heavy machines may be able to achieve this in one pass; lighter, less powerful machines require several passes.

Depth control is achieved either by adjusting the height of the machine as a whole or by adjusting the height of the cutting mandrel relative to the machine. When the whole machine is adjusted, the drive tracks or tires are attached to the machine through adjustable hydraulic cylinders, and the cutting mandrel is fixed directly to the machine frame. If depth is controlled by moving the mandrel, the mandrel is attached to the frame by adjustable hydraulic cylinders. This provides quick response to depth adjustments.

Most machines have the option of adjusting the depth of the cut either manually or automatically. Automatic depth control systems work through elevation sensors that can be operated off of a reference wheel, an averaging ski, or a string line. The accuracy of the depth control is generally limited by the maximum aggregate size.

Level and cross fall are achieved by raising either the machine or the ends of the cutting mandrel to the required height.

6.4.1.3 Power, Weight, and Cutting Speed

Milling machines are powered by diesel engines ranging in power from 24 horsepower (18 kW) for the smallest machines to 750 horsepower (560 kW) for the largest machines (3). The typical range for bridge deck work varies between 250 and 500 horsepower (180 kW to

375 kW).

Weight correlates closely with power and is important when evaluating a milling machine. Machine weights range from less than 4,000 lb (1,800 kg) for the smallest machines to more than 100,000 lb (45 mt) for the largest. A heavier machine is able to exert a greater downward force to keep the machine in the cut and increase production. Although a heavier machine is generally capable of working more efficiently, many bridges have load restrictions that must be taken into account when selecting a machine. Some SHAs have, and many are considering, a 42,000 lb (19 mt) limit on the weight of the milling machines for bridge deck work.

Milling machines run on tracks or tires that are independently powered by a hydrostatic motor. This allows the machine speed to be adjusted without affecting the engine speed. Tires are used on most small machines and some mid-size machines. They have the advantage of greater maneuverability, less damage to the bridge deck, and greater dampening of vibrations.

Mid- and large-size machines are usually equipped with tracks. The primary advantages of tracks their ability to carry heavier loads and exert increased traction. Tracks also distribute a machine's weight over a larger area and thus reduce the possibility of exceeding point load limits on the bridge.

Machines usually have separate speed ranges for operating and traveling. Operating speeds range from 33 ft/min to 150 ft/min (10 to 45 m/min) but will largely be determined by the material being milled. Traveling speeds are generally 4 to 5 mph (6 to 8 kph) but can be as high as 24 mph (40 kph).

The relationship between the weight of the machine, mandrel width, operating speed, and optimum cutting depth is determined by the manufacturer and set out in the specifications for the machine.

6.4.1.4 Debris Conveyor

Most larger machines are equipped with a hydraulically controlled conveyor system for the removal of debris. Conveyers either discharge to the front of the machine or to the rear. The latter provides greater visibility for the operator due to reduced dust and reduced forward obstruction (4).

Front-discharging conveyors make it possible for the trucks receiving the milled material to travel in the same direction as the milling machine and traffic. This decreases the amount of time required to switch trucks and thus makes the operation more efficient. Also, any material that falls off the truck or conveyor is recycled by the milling machine, leaving a cleaner finished surface.

Conveyors, whether front-loading or rear-loading, generally are capable of swinging from side to side to facilitate loading the material into a truck traveling alongside. Although conveyors are very efficient on large highway milling projects, they are frequently not used on bridge rehabilitation jobs because they restrict maneuverability.

6.4.2 Work Characteristics

This section discusses the nature of the projects and the work tasks suited to the use of milling machines. The factors that cause the operations to be productive and the factors that limit them from achieving their full potential are analyzed. Note that there is always a need for some hand cleanup work with milling operations.

6.4.2.1 Project Type and Location

The ideal work environment for high-production milling machines is provided by one large bridge or several smaller but consecutive bridges where the entire deck surface must be removed to a specified depth above the reinforcing steel. By milling a large area at once, equipment is used more efficiently and reduced mobilization costs are achieved. Milling machines are not designed for and not economical when used on small areas requiring intermittent operation and substantial maneuverability.

The location of a bridge is important because mobilization costs are high if the distance between work sites is large. Reduced travel distances will result in lower mobilization costs and will also allow the contractor to react more quickly to schedule changes.

Urban job sites often have several limitations. Bridges are usually quite small and access is often limited. Operating hours are often restricted and work may be permitted only at night. Rural job sites generally have better access and fewer limitations on operating hours but may be more costly in terms of mobilization.

6.4.2.2 Type and Extent of Deterioration

Milling machines are only able to remove concrete from above the top mat of reinforcing. If contamination or deterioration is limited to concrete above the top mat of reinforcing, or if milling is being performed only as preparation for an overlay, a milling machine will be ideally suited to perform the work. If the required depth of removal extends below the top of the reinforcing steel, then additional or alternative methods of concrete removal such as pneumatic breakers or hydrodemolition will be required.

The wide cutting drum and the continuous action of the cutter restricts the machine from selectively removing damaged concrete. Concrete that is cracked will offer less resistance to the cutting mechanism and will allow the machine to attain higher rates of production.

6.4.2.3 Preparatory Work

Prior to commencing milling work, the area to be milled, typically one lane or half the width of the bridge, is closed to traffic. This can be accomplished through temporary means such as cones or barrels if traffic volume is low or if the bridge is to be closed for a short time. High-speed roads with high traffic volumes generally require that concrete barriers be installed to provide added safety to both the motorists and the crews employed on the milling operations.

Sounding and marking the deck prior to milling is generally not necessary because milling typically removes concrete over the entire deck area. However, the depth to the top mat of reinforcing should be determined at random locations over each span of the bridge deck before milling begins. Although the depth to the reinforcement generally varies less than 1 in. (2.5 cm) from the average cover depth, the expense in time and effort required to determine the rebar cover is much less than the cost incurred if a bar is pulled up. If the size of the reinforcing bar is known, the depth to the reinforcement can be fairly accurately measured with a commercially available pachometer, which magnetically senses the location and depth of the reinforcing steel.

The depth of the rebar is to be measured for each span. A minimum of 40 random measurements must be taken for each span. For spans with a surface area greater than 8,000 ft^2 (740 m^2), 40 random measurements must be taken for each 8,000 ft^2 (740 m^2). For span sections less than 8,000 ft^2 (740 m^2), the number of measurements is to be proportional to the residual square footage. That is, for a span of 4,000 ft^2 (370 m^2), take 40 random measurements; for a 12,000 ft^2 (1,110 m^2) span, take 60 random measurements. If the cover depth is 2 in. (5.08 cm) with a standard deviation of 0.25 in. (0.635 cm), then it would be fairly safe (< 1% change) to mill to a depth of 1.35 in. [2 in. - (2.58)(0.25 in.)][5.08 cm - (2.58)(0.635 cm)]. *Note*: 2.58 is the number of standard deviations associated with less than a 1% change for a normal distribution; see any standard statistics book.

6.4.2.4 Material to Be Removed

In order for the depth of removal to be accurately monitored, any previously placed asphalt overlay must be removed prior to the concrete removal operation. Milling machines are ideally suited to remove most overlay materials, but an allowance must be made for the additional time required for the overlay removal.

The properties of the concrete to be removed from the bridge deck will affect the milling machine's efficiency and production. The primary components of the concrete that affect the operation are the size and hardness of the aggregate.

It is generally much easier for a milling machine to break the bond between the cement paste and the aggregate than to fracture the aggregate. However, this will be difficult if the aggregate mix is very dense and if the bond between mortar and aggregate is very high.

Aggregate hardness also affects production, and contractors must make allowances for this by lowering estimated production and increasing maintenance downtime in areas that have typically hard aggregates.

6.4.2.5 Area of Removal

The area to be removed is the primary factor affecting milling machine efficiency and productivity because it determines the extent to which the machine can be operated at optimum capacity. Continuous, uninterrupted operation enables the machine to reach high production rates and achieve lower unit costs. It also permits a greater number of hours in operation and thus the contractor is able to recover the costs of owning the machine in a more efficient and timely manner. This is of great importance with capital (equipment)-intensive methods where owning and capital recovery costs play a major role.

6.4.2.6 Depth of Removal

The depth of concrete above the top mat of reinforcing and the depth of contaminated or deteriorated concrete requiring removal are important project parameters affecting the use of a milling machine. The depth to the reinforcing bars determines the maximum depth to which the milling machine can operate. If the concrete requiring removal extends beyond this depth or is in areas inaccessible to the milling machine, additional concrete removal methods will be required.

6.4.2.7 Obstructions

The quality of the original deck construction plays a major role in determining the efficiency of any subsequent milling operation. Reinforcing bars and other built-in steel components frequently interrupt the milling operation and preclude the machine from achieving its full potential. Increased caution must be taken when the depth of reinforcement varies. The machine size and drum width should be selected to provide the maneuverability needed to avoid obstructions. Some areas cannot be reached because joints, drains, drain covers and the like obstruct the milling process, and such areas must be removed using pneumatic breakers in a secondary operation.

6.4.2.8 Theoretical Production Rate

The first production figure to be calculated is the theoretical production rate. This is the maximum rate of production that can be attained for short periods of time under ideal operating conditions. The theoretical production rate is determined by the machine power and weight, cutting width and depth, and anticipated operating speed. The cutting width is determined by the width of the cutting mandrel; a wider mandrel requires fewer passes of the machine.

The cutting depth is determined by the extent of damage and the location of the reinforcement steel. If the concrete must be removed to a depth very close to the reinforcement, the machine must operate slowly to be certain that the reinforcement bars are not damaged or pulled up by the machine.

The operating speed varies between 10 to 25 ft/min (3 to 8 m/min) and will be set by the operator based on the weight and power of the machine, the hardness of the material being removed, and the depth of removal.

6.4.2.9 Modified Production Rate

A modification factor is used to scale down or modify the theoretical production rate so that it more accurately reflects the production that will actually be achieved. It is a highly subjective number based on many inputs, both tangible and intangible. The factor may range from 0.3 for a job that is expected to progress very slowly, to 0.8 to a fairly simple job with no anticipated delays.

Continuity of operation is a primary parameter affecting the modification factor. If the removal is continuous, the factor will be high. If the removal is noncontinuous, as in patching and other work requiring the removal of small areas, then the actual production can be expected to be substantially lower than the ideal production.

The time required to remove the milled debris will also cause a reduction in the modification factor. Many machines are equipped with debris removal conveyors that load the milled material directly into a vehicle for transport away from the job site. If a milling machine is not equipped with a conveyor, or if the contractor opts not to use one in order to gain maneuverability, then the material will need to be removed by loader in a separate operation that delays production. A mechanical broom or industrial vacuum truck may be required to remove any material not picked up by the conveyor or shovel.

Downtime for maintenance on the milling machine will also contribute to the modification factor. Routine maintenance can be expected to reduce production by between 10% and 20%. This maintenance will include such items as changing the teeth, oil, lubrication, filters, and hoses.

6.4.3 Managing and Controlling Quality

Specific requirements must be established and maintained to ensure that work quality is of an acceptable level. This section addresses five issues concerning these requirements.

1. *Machine size.* A machine that is not the proper size to perform the work can result in a product of inferior quality. A machine that is too small will not have the necessary production to be economical, and it may also damage the residual concrete if the machine is pushed beyond its capabilities. Small

machines are, however, inexpensive to mobilize, maneuverable, and better suited to small, low-production, or intermittent jobs.

Large machines are well-suited to large, continuous production projects. Many are designed and built for use on open pavement and may lack the control and maneuverability required for bridge milling work. Additionally, the weight and vibrations of a large machine can easily damage the remaining deck or structural components of the bridge.

Appropriate precautions must be taken to ensure that the bridge structure is not loaded beyond its capacity. Any weakening of the bridge due to a reduction in depth and the relocation of the neutral axis caused by removing surface concrete should be taken into account when evaluating the structural capacity of the bridge.

2. *Damage to residual concrete.* Impact methods of concrete removal such as milling may produce a layer of damaged concrete with small cracks extending 0.5 to 0.75 in. (1.2 to 1.8 cm) into the residual concrete. These microcracks reduce the strength of the concrete, lower the bond between concrete and steel and reduce the bond between the existing structure and any overlay material.

   The cutting mandrel on milling machines rotates upward and away from the cut so as to break the concrete in tension. This reduces the potential for microcracking, which only becomes a serious problem if a heavy and powerful machine is forced to progress at too rapid a rate.

3. *Damage to reinforcing steel.* Great caution must be exercised when operating a milling machine in the vicinity of the top reinforcing steel mat. On projects that require that the milling operation remove the concrete to a depth that approaches the cover over the reinforcing bars, the machine should proceed very slowly and under close control of the machine operator and groundsman. Because the cover over the reinforcing bars is not always uniform, the machine may occasionally catch a bar and pull it up out of the concrete. This frequently damages the machine by breaking teeth and holders, and can cause extensive damage to the deck.

   Milling to critically controlled depths is a very costly, time-consuming, and risky operation. It is frequently more cost-effective to use a milling machine to remove the concrete to a reasonable depth above the reinforcement, and to then use other methods such as pneumatic breakers or hydrodemolition to remove any remaining damaged concrete.

4. *Surface characteristics.* The milling process leaves the residual concrete with a rough textured finish, which can be either opened to traffic immediately or

covered with an overlay material. The texture of the finish produced by a milling machine is dependent on the tooth configuration, the mandrel rotation speed, and the rate at which the machine progresses. The grid pattern produced by milling allows the overlay material to interlock with the milled surface, forming a tight bond.

5. *Environmental concerns*. The dust generated by the milling and debris removal operations can obstruct the vision of both the machine operator and the passing motorist. It should thus be monitored and maintained at acceptable levels so as not to endanger traffic or affect the surrounding environment. Dust can be controlled by water spray bars on the machine. Water spray cools the heads and teeth.

   Equipment noise should also be monitored, especially if the work is being performed in densely populated urban areas. Water runoff is generally not a problem, but appropriate containment measures should be taken if runoff is likely to impede traffic or pollute the environment.

6.5 Hydrodemolition

This section examines hydrodemolition as a method for removing concrete as a part of the bridge rehabilitation process. It is a capital-intensive technology that uses complex equipment to produce and direct a high-pressure water jet to erode the cement matrix between the concrete aggregate. It is capable of attaining a high rate of production while selectively removing deteriorated or contaminated concrete to the desired depth. It is effective in cleaning the reinforcing steel and preparing the surface for a subsequent overlay.

6.5.1 Description and Equipment

Hydrodemolition in its simplest terms involves the pressurization of water and the controlled delivery of a waterjet to demolish the cement matrix. This requires a sophisticated equipment system consisting of two distinct components: a power unit and a demolishing unit (5).

6.5.2 Power Unit

Figure 6.9 shows a typical power unit used to provide the high-pressure water required for hydrodemolition. It comprises a drive engine, a high-pressure pump, water filters, a water reservoir tank, and other accessory equipment (6). The power unit is housed in a large metal container on a flatbed tractor-trailer.

The water supplied to the power unit is passed through a series of filters before storage in the reservoir tank. The filters remove solids from the water to prevent excessive wear on the high-pressure system.

The high-pressure pump is driven by a 300 to 500 horsepower (225 to 370 kW) diesel engine. The engine size varies depending upon the specific system make and the capacity of the pump. Two different types of high-pressure pumps may be used. A plunger- or piston-type pump is able to pressurize the water to between 12,000 and 20,000 psi (83 to 138 MPa) at a flow rate of 20 to 70 gpm (75 to 265 L/min). An intensifier pump is capable of pressurizing small flows of water to ultrahigh pressures. One hydrodemolition system currently uses intensifier pumps that deliver water at 35,000 psi (240 MPa) and 13 gpm (50 L/min).

Hydrodemolition systems may use one or two power units. Using two power units running in tandem doubles the flow rate, roughly doubling the productive capacity of the hydrodemolition system.

6.5.2.1 Demolishing Unit for Bridge Decks

The demolishing unit used for bridge decks is a microprocessor-controlled wheeled vehicle, as illustrated in figure 6.10. A water delivery nozzle is attached to a trolley that traverses back and forth along a cross-feed beam at a programmed rate. The nozzle is rotated or oscillated at a constant programmed frequency. At the end of the trolley's programmed cycle, the entire demolishing unit advances or indexes forward a set distance. Microprocessor controls dictate all movements of the nozzle, the nozzle trolley, and the demolishing unit to ensure precise control over the fluid dynamic properties of the waterjet and to provide consistent quality.

Limit switches located at opposing ends of the cross-beam can be adjusted within the length of the beam to produce a cut of desired width, enabling the unit to remove various sizes of rectangular areas.

High-pressure water is delivered from the power unit(s) to the nozzle by high-pressure flexible hosing. The flexible hosing consists essentially of a hose within a hose. The inner hose carries the high-pressure water; the outer hose serves to shield the inner hose from cuts and acts as a safety containment should the inner hose burst. The system is also designed with an emergency water shut-off valve that activates automatically if a hose loses pressure or ruptures.

6.5.2.2 Equipment for Vertical and Overhead Surfaces

The demolishing unit is the predominant piece of equipment used in hydrodemolition on bridge decks. Some makes of equipment have special attachments that enable the cross-beam to be held upright or overhead for concrete removal on vertical and overhead surfaces. This type of equipment is not used frequently on bridges because the substructure elements have small, irregular surface areas that are difficult to access.

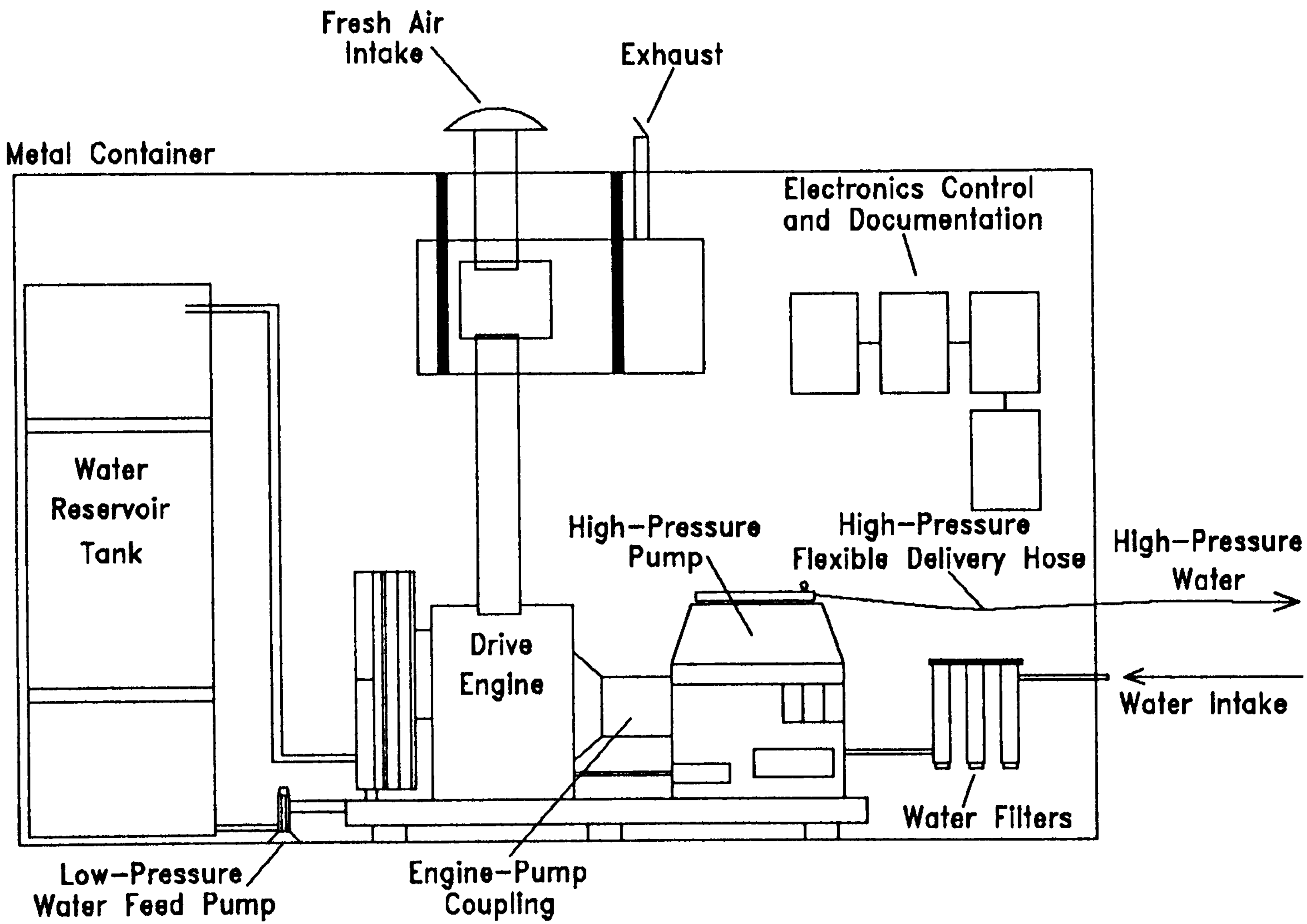


Figure 6.9 Hydrodemolition Power Unit

Manufacturers do make hand-held wands that operate at lower pressures and flow rates. These require a person to hold the wand and direct the water jet over the concrete surface. The loss of microprocessor control over the water jet's movement causes the quality to vary, and safety considerations make hand-held water jets all but impossible to use.

Some experimental equipment exists for special applications of concrete removal work such as columns and tunnels.

6.5.2.3 Operating System

The equipment required to perform hydrodemolition work consists of a trailer containing the power unit, the demolition unit itself and equipment needed for debris removal and cleanup. If a water source is not available, a water supply truck will also be required.

The operating parameters for the hydrodemolition system are established through a process of estimation and testing. The summary of the process is outlined in figure 6.11, which shows that the contractor initially sets the equipment operating parameters based on job parameters, concrete parameters, and past experience.

A trial area of sound concrete is hydrodemolished using estimated operating parameters. After evaluating the results of the trial area, the system parameters are adjusted until the desired mean removal depth is achieved. The system is then tested on an area of deteriorated concrete and the operating parameters recalibrated until the concrete is removed to the desired level of soundness.

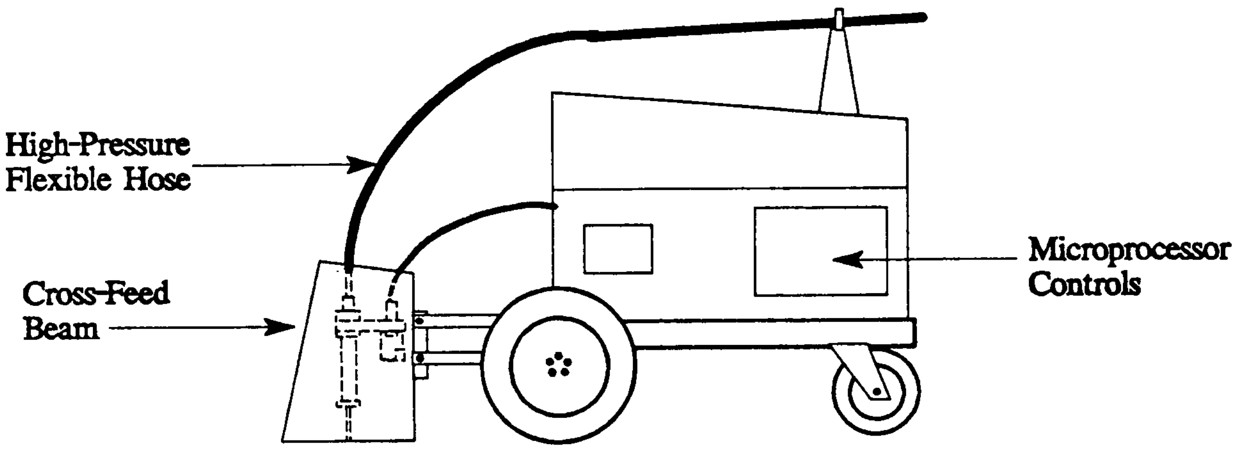
The microprocessor control ensures constant, repeatable results. However, if the concrete material or job parameters change, the equipment must be recalibrated.

6.5.3 Work Characteristics

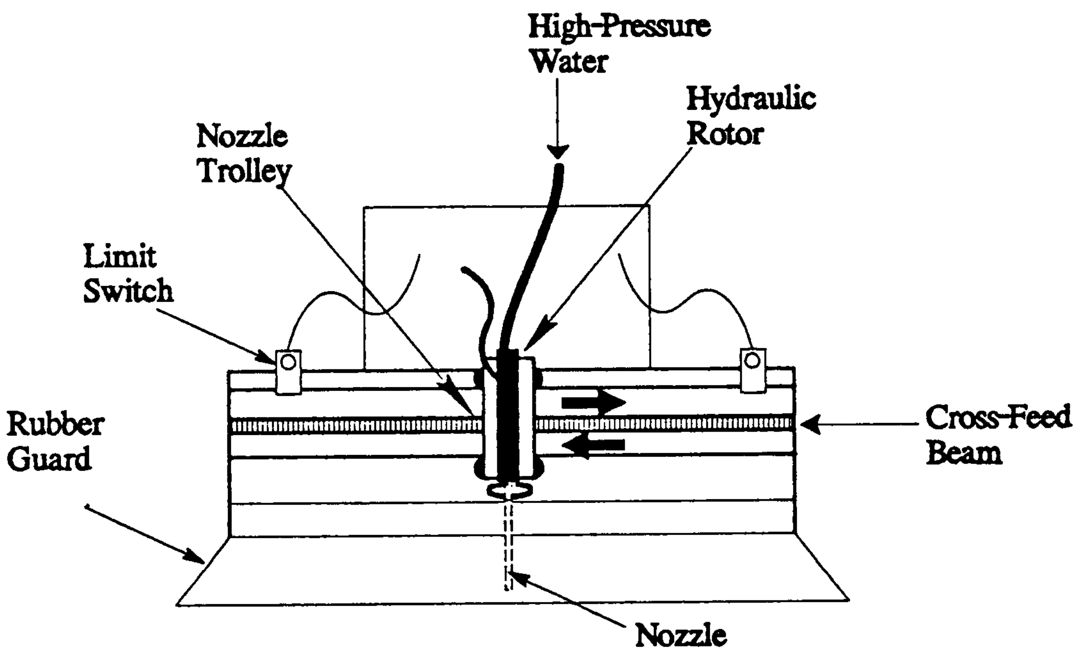
Hydrodemolition is primarily applicable to projects that require the extensive removal of deteriorated or contaminated concrete to a desired depth or level of soundness over a large, continuous horizontal area. Bridge decks that contain large quantities of contaminated or deteriorated concrete are ideally suited to hydrodemolition's ability to remove concrete from around and below the reinforcement while operating within the restraints imposed by the equipment geometry.

6.5.3.1 Project Type and Location

The high capital costs and the high degree of mechanization involved make hydrodemolition



Side View



Front View

Figure 6.10 Hydrodemolition Demolishing Unit

most favorable if used on projects that enable it to be operated uninterrupted.

Access to the site should be such that a sizable portion of the deck is available to allow the hydrodemolition equipment to perform the necessary work with as few setups as possible.

Hydrodemolition will require that a portion of the bridge be completely closed to traffic for an extended period of time. Although the high productivity associated with hydrodemolition enables the concrete removal to be performed quickly, the surface produced is not suitable to be reopened prior to patching or overlaying, which may not occur for several days.

The specialized nature of owning and operating hydrodemolition equipment means that it is frequently more economical for the work to be done by specialty subcontractors. They are better able to achieve the number of hours required to recover the high initial investment and able to meet the specialized maintenance demands of the equipment.

6.5.3.2 Type and Extent of Deterioration

Hydrodemolition equipment can be calibrated to remove sound chloride-contaminated or deteriorated concrete to the depth necessary to achieve an acceptable level of contamination in the residual concrete. Selective removal is achieved by applying a constant amount of energy to the concrete in a manner that causes all material with less than the required strength to be removed, regardless of depth.

6.5.3.3 Preparatory Work

The work area must be cordoned off with either concrete barriers or traffic barrels, depending on the extent of the rehabilitation and the traffic conditions. A primary concern related to the method of traffic control is the containment of the water and paste-like slurry that might flow into adjacent lanes open to traffic.

If concrete barriers are used to cordon off the work area, a silicon caulk can be used to form a seal between the bottom of the barriers and the bridge deck, thus preventing the runoff water from flowing into adjacent lanes of traffic. Hay bales are often used to filter the debris and suspended solids out of the runoff when traffic barrels are used as the method of traffic control.

In either case, it is not possible to completely prevent water from running into adjacent open traffic lanes to provide additional safety, construction warning signs should be used to inform motorists that unexpected wet pavement lies ahead.

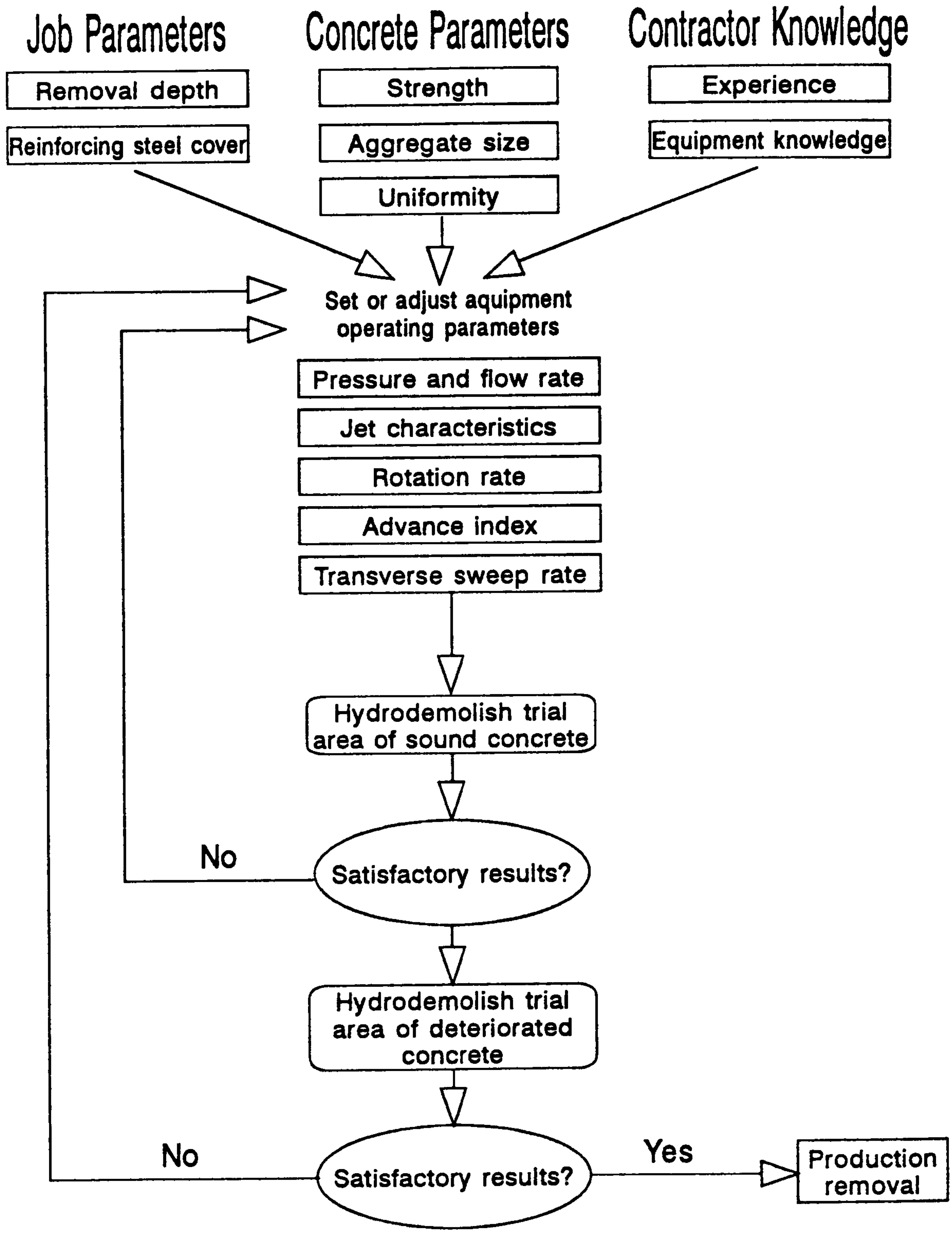


Figure 6.11 Summary of Hydrodemolition Calibration Process

6.5.3.4 Material to Be Removed

The strength, uniformity of strength, and aggregate size of the concrete will determine how effectively a hydrodemolition system is able to remove the material. These material properties also determine the resultant surface profile obtained from hydrodemolition. Each of these properties must be evaluated to establish the optimum fluid dynamic operating parameters and the expected results of hydrodemolition.

1. *Strength of material.* Hydrodemolition removes concrete by applying a waterjet of greater energy than can be absorbed by the material being removed. The strength of the concrete will therefore determine how much energy it is capable of absorbing and the associated waterjet energy required to remove it.

2. *Uniformity of strength.* A hydrodemolition system is calibrated to remove concrete of a uniform strength to a specified depth. Any deviation in the strength of the concrete encountered will result in an inconsistent depth of removal. A lower-strength concrete will be removed to a greater depth, and a higher-strength concrete will not be removed to as great a depth.

3. *Aggregate size.* The concrete aggregate will primarily affect the hydrodemolition operation by determining the texture of the resultant surface. Hydrodemolition removes concrete by destruction of the cement matrix; it does not split or cut the aggregate. The resulting surface profile is not smooth and its texture is determined by the maximum aggregate size, as illustrated in figure 6.12.

6.5.3.5 Area of Removal

Economic recovery of the high owning and operating costs of hydrodemolition equipment cannot be realized unless the equipment can operate over a large and continuous area.

The area that can be removed by hydrodemolition is physically limited by the machine geometry and method of operation. The cross-beam width and limit switch settings will determine the width of the cut and therefore the number of passes required.

6.5.3.6 Depth of Removal

Hydrodemolition is capable of removing concrete above, around, and below reinforcing steel. The mean depth of removal is determined by concrete parameters and adjustable equipment operating parameters detailed in reference 7.

6.5.3.7 Debris Removal and Cleanup

The demolished concrete and other waste products arising from hydrodemolition form a combination of rubble, slurry, and runoff water. Cleanup is accomplished by vacuuming or manually shoveling the coarse particles and flushing the slurry and fine particles away with fresh water.

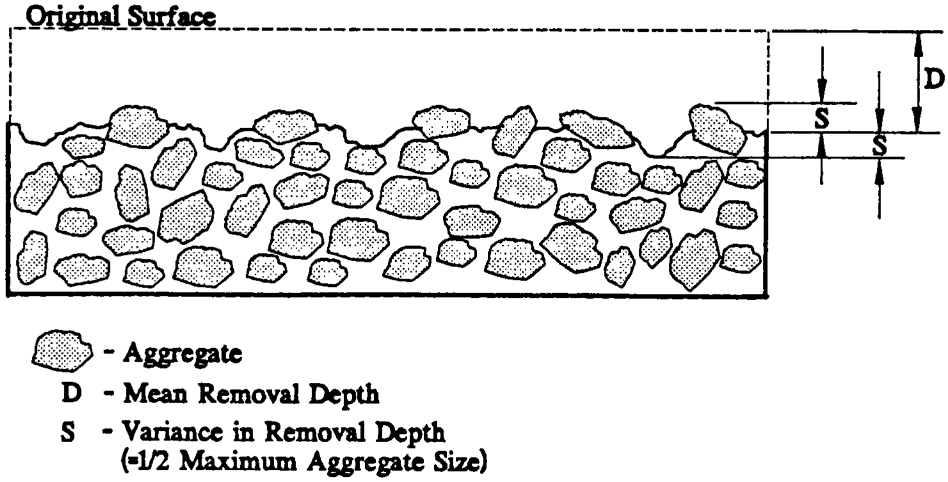


Figure 6.12 Mean Depth of Removal as a Function of Aggregate Size

The slurry should not be allowed to dry on the prepared surface because the paste strongly adheres to the deck. If drying of the paste is allowed to occur, the deck must be thoroughly waterblasted or sandblasted to provide a clean, bondable surface. The requirement to provide a good bonding surface for patches or overlays means that waterblasting or sandblasting operations must be performed or repeated no more than 24 hours before the overlay placement.

An industrial vacuum truck is often used to clean the hydrodemolition waste. Vacuuming takes place behind the advance of the demolishing unit using a hand-held vacuum nozzle. The vacuumed area is frequently flushed with fresh water and revacuumed before the required level of cleanliness is reached.

Using an industrial vacuum truck generally requires a two-lane-wide work space. Figure 6.13 shows a typical hydrodemolition system setup using an industrial vacuum. While the use of an industrial vacuum reduces the volume of water runoff, it does not eliminate the need for water runoff control.

Another method of hydrodemolition cleanup involves hand-shoveling the rubble and flushing the deck with clean water. This method is more labor-intensive and requires controlling a larger volume of runoff water than vacuuming, but it is better suited to confined work spaces

where large equipment cannot be used. Figure 6.14 shows a typical hydrodemolition system setup using a manual cleanup operation.

In addition to the runoff control and debris filter, the deck drains are often plugged to allow the water to run down the bridge deck in an effort to settle the cement and fine aggregate particles out of suspension. Sandbags may be used to direct the flow of the runoff, and hay bales or pea gravel dikes are often used to filter the suspended solids out of runoff water.

6.5.3.8 Theoretical Production Rate

The theoretical production rate is the quantity of concrete removed for each unit of time the waterjet spends actually hitting the concrete. This is a function of equipment parameters, concrete properties, and job parameters.

1. *Equipment parameters*. The primary equipment parameter affecting the instantaneous production of a hydrodemolition system is the power of the water jet measured in terms of both the pressure and the flow rate of the water.

2. *Concrete properties*. The concrete's strength, aggregate size, and aggregate type all influence hydrodemolition's instantaneous productivity. The stronger the concrete, the lower the instantaneous productivity. The aggregate type is significant because it affects the cement-to-aggregate bond strength.

3. *Job parameters*. The required removal depth is the main job parameter that influences instantaneous production. If the concrete above the top mat of reinforcing is first removed by milling, then the loss in production due to the depth of removal can be reduced.

Figure 6.15 shows the range of theoretical productivity for hydrodemolition systems. These productivity rates are based on the hydrodemolition equipment manufacturer's productivity estimating guidelines for "typical" 4,000 psi (28 MPa) concrete and a removal depth of up to 3 in. (7.5 cm).

6.5.3.9 Modified Production Rate

Modification factors are used to scale down the theoretical production to reflect specific job characteristics. The primary parameters that contribute to the modification factor are the area of removal, the continuity of operations, and equipment downtime for maintenance and repair. The ability to accurately select this production modification factor is dependent on the hydrodemolition contractor's experience.

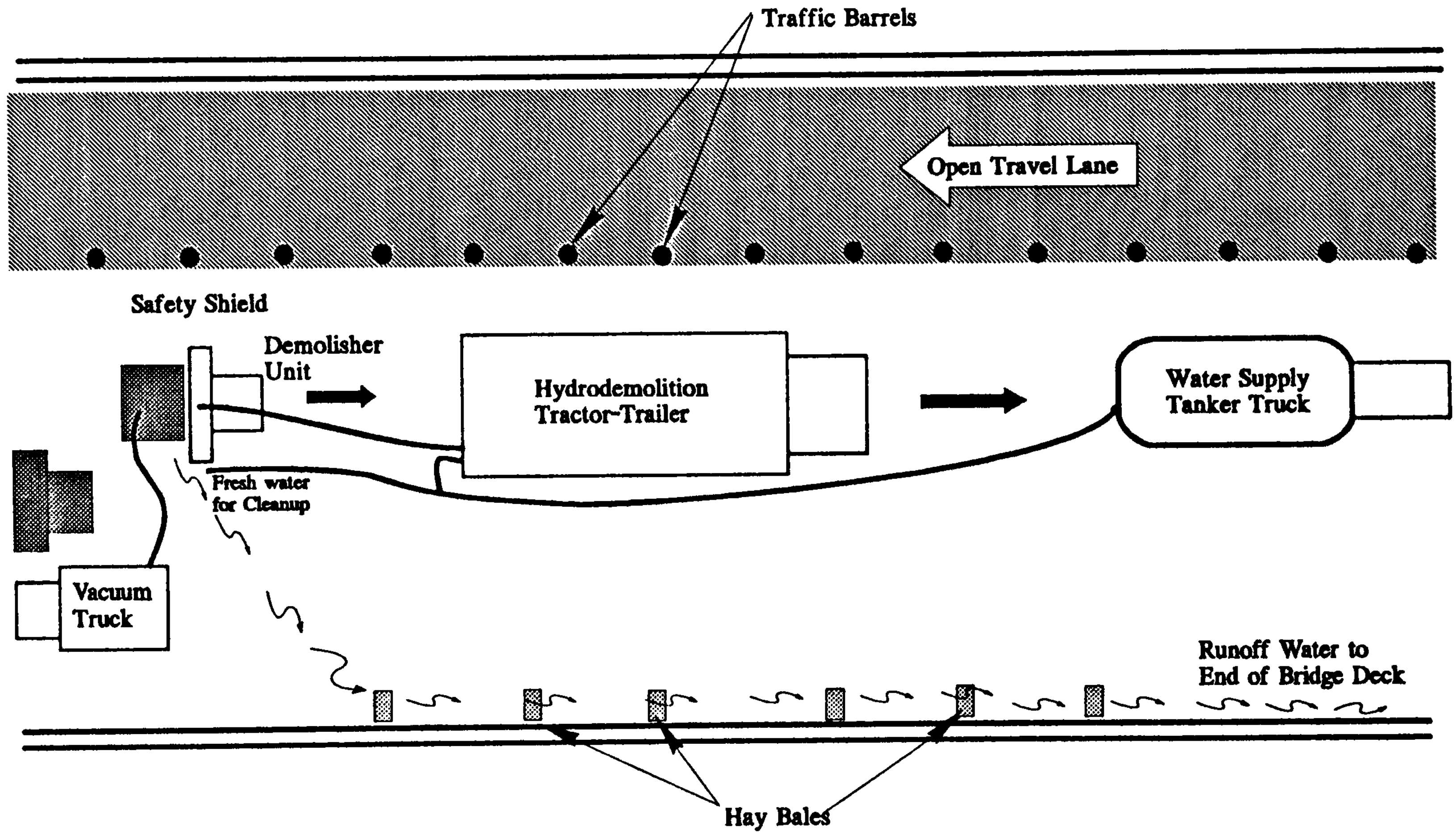


Figure 6.13 Hydrodemolition Setup Using a Vacuum Truck

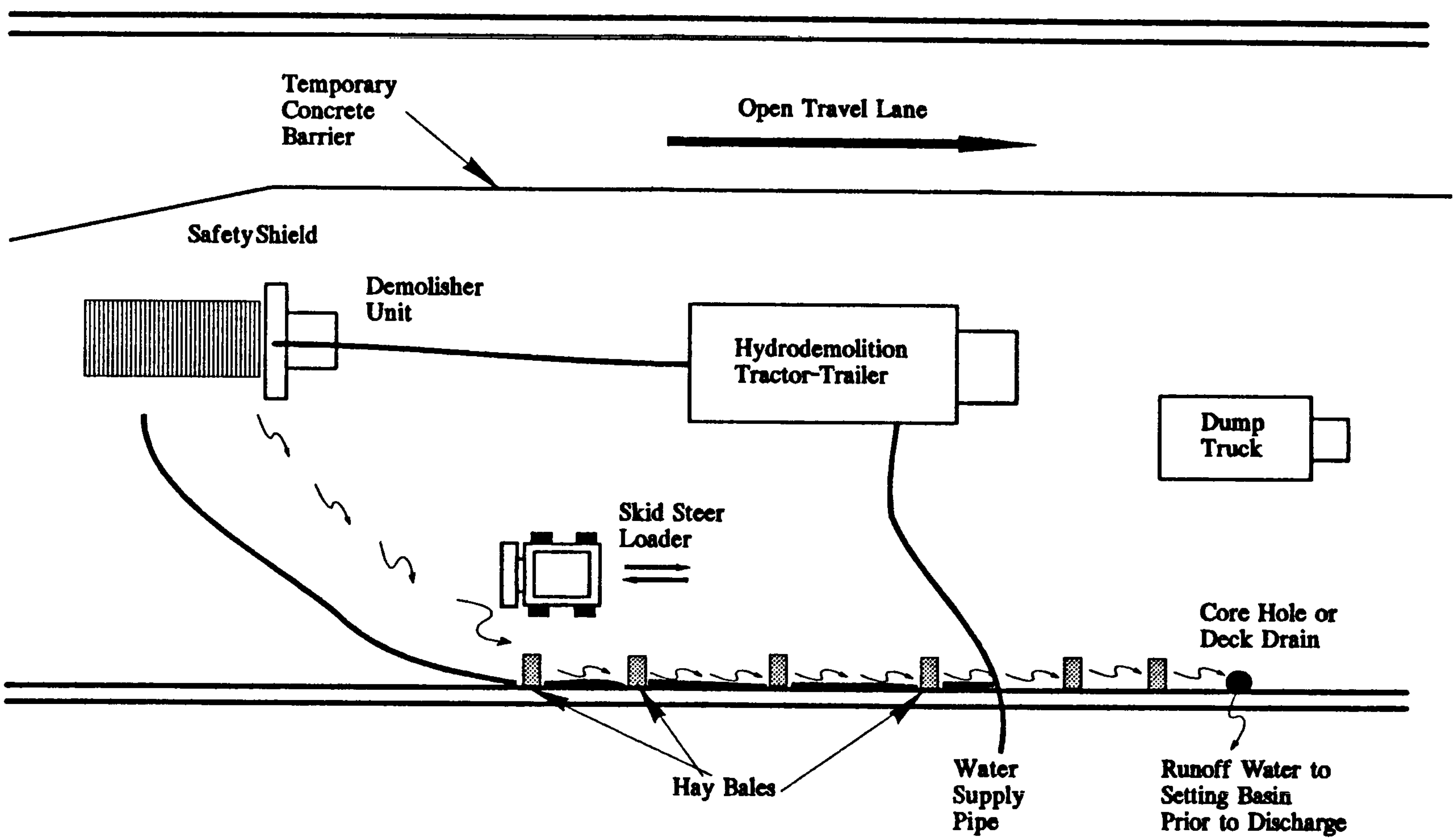
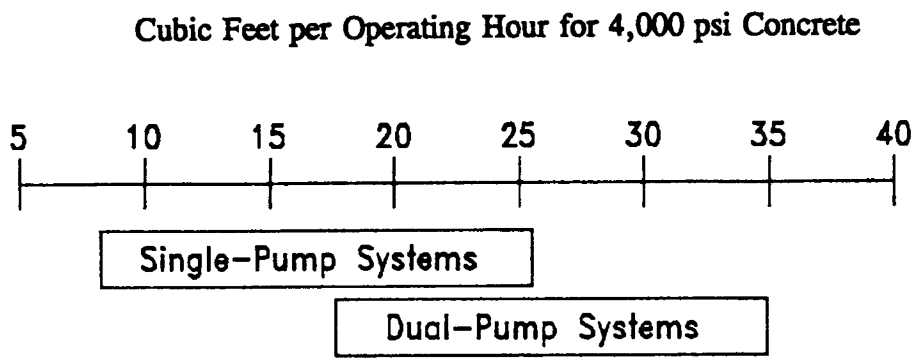


Figure 6.14 Hydrodemolition Setup Using Manual Cleanup

The modification factor ranges from 0.40 to 1.00. A modification factor of 0.80 to 1.00 is typically used for a job where operations are able to proceed uninterrupted over a large continuous area. A modification factor of 0.40 to 0.80 is typically applied to jobs that involve removing concrete from small noncontinuous areas.

Downtime for maintenance and repair will further reduce the theoretical production rate, depending on the reliability of the particular system.



(1 psi = 6.895 kPa)

Figure 6.15 Range of Theoretical Productivity for Hydrodemolition Equipment

6.5.4 Managing and Controlling Quality

An understanding of the issues relative to managing and controlling the quality of hydrodemolition operations is necessary to ensure effective use of the technology. The issues addressed in this section are quality requirements, residual cracking, and specifications.

6.5.4.1 Quality Requirements

The primary quality concern associated with concrete removal by hydrodemolition is ensuring that the machine is correctly calibrated to remove the concrete to the specified depth or level of soundness. The opposite of hydrodemolition's inherent advantage of selective removal occurs when there is a high-strength concrete or other material on top of or within an area of lower-strength concrete. This condition, shown in figure 6.16, typically is encountered when rehabilitating a bridge deck that has been previously patched.

If the hydrodemolition system is calibrated to remove the high-strength concrete, excessive amounts of original low-strength material may be removed from around the perimeter or underneath the high-strength material. Precise depth control is not possible when the concrete's strength is not uniform.

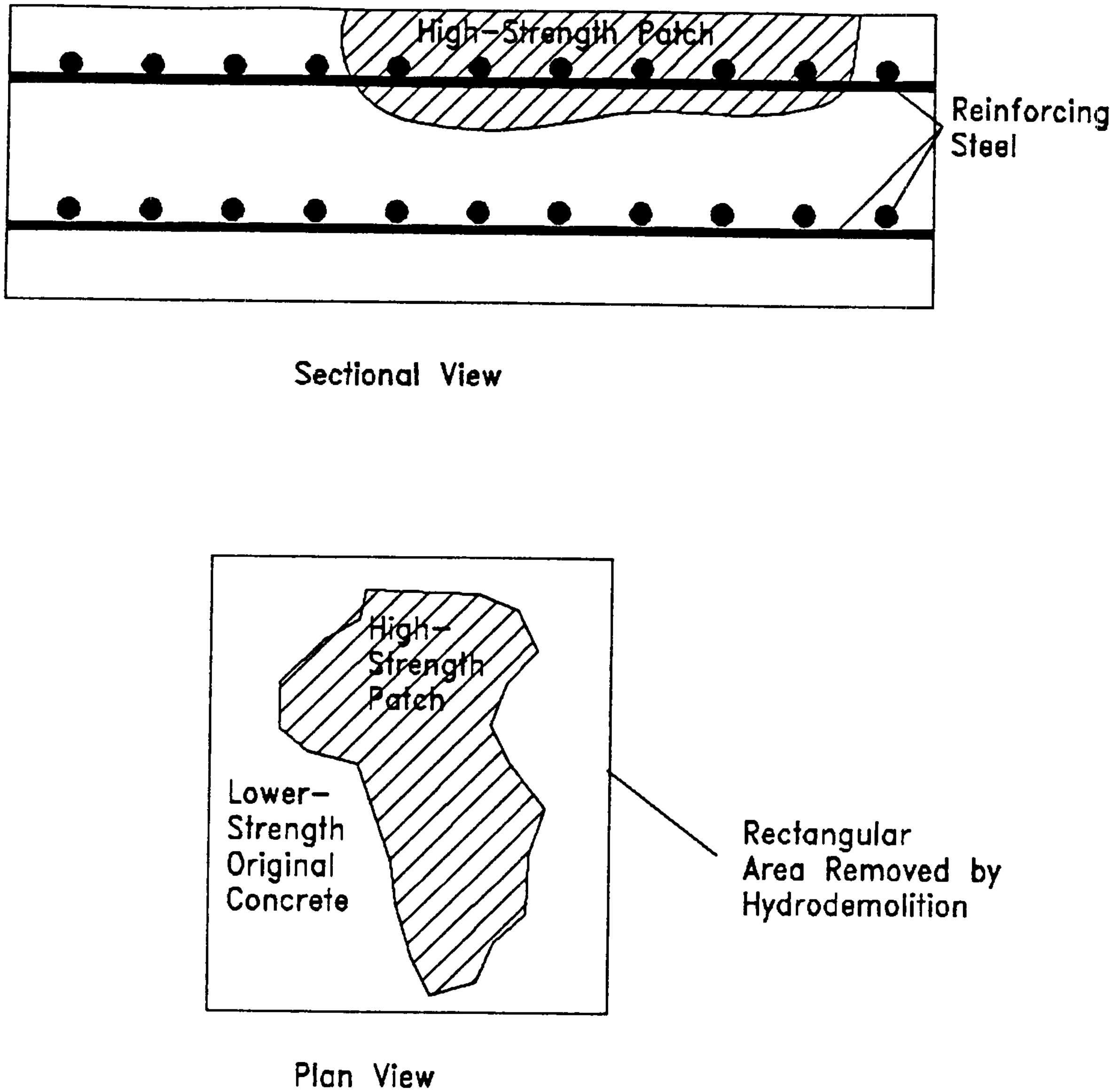


Figure 6.16 High-Strength Patch in a Bridge Deck

Existing patches that have a lower strength than the original deck concrete do not present the same problem as higher-strength patches. Bituminous concrete patches in a bridge deck are easily removed by hydrodemolition.

However, there may be problems if an unanticipated area of extensively deteriorated or low-strength concrete is encountered. If calibrated for a higher-strength material, the hydrodemolition system will remove the concrete to a greater depth than desired, possibly all the way through the deck.

The best way to handle this problem is to anticipate its occurrence by carefully inspecting the underside of the bridge deck. Anticipating this situation enables the contractor to provide a means for catching the debris and protecting the area under the bridge before hydro-demolition.

Hydrodemolition is capable of removing concrete from around and below reinforcing steel bars without causing damage to the bars or damaging the concrete-to-steel bond. However, problems with rebar shadowing can occur because the reinforcing steel bars shield the concrete directly beneath them from impingement by the water jet. Figure 6.17 illustrates what happens as the hydrodemolition nozzle advances across a steel reinforcing bar. The shaded triangular area directly below the steel bar is shielded from impingement by the water jet. If the mean removal depth is below the water jet's intersection point, then the unimpingeable area will be removed by the scouring action of the loose aggregate. If the mean removal depth is above the water jet intersection point, then the unimpingeable area will not be removed and a rebar shadow will remain.

Hydrodemolition's ability to remove concrete from directly below reinforcing steel bars is a function of the bar size, the maximum aggregate size, the water jet angle of impingement, and the chosen mean removal depth. Larger steel bars move the water jet intersection point deeper and create larger unimpingeable areas. To avoid rebar shadowing, the mean removal depth should be set so that it is below the water jet intersection point and the rebar clearance is at least equal to the maximum aggregate size.

Hydrodemolition cleans the reinforcing steel bars as it removes concrete from around them. The swirling action of the high-velocity water and the fine aggregate particles from the demolished concrete act to provide a wet sandblast that effectively removes rust deposits and bonded cement from the reinforcing steel bars.

Careful planning is required to effectively control the large volume of water generated by the removal operation. The planning must consider such items as the deck's geometry, equipment access, and maintenance of traffic.

6.5.4.2 Specifications

Pneumatic breakers and milling machines are relatively well-known technologies for concrete removal. Most SHAs have developed specifications for use with these methods, have tried them and found them adequate. This issue has therefore not been discussed for those technologies. The same is not true for hydrodemolition, and many SHAs are still actively developing suitable specifications.

A prevalent problem with the hydrodemolition specification is the use of method-specific clauses that limit the operating parameters of the hydrodemolition equipment. These specifications tend to favor particular makes of hydrodemolition equipment while excluding others, regardless of ability to produce a product of acceptable quality. Hydrodemolition specifications should be performance specifications that address only the aspects of the work that are necessary and critical to achieving the desired quality for the final product. A hydrodemolition guide specification has been developed and is presented elsewhere (1).

Every bridge rehabilitation job must be evaluated on an individual basis to determine if hydrodemolition is capable of effectively performing the concrete removal. Hydrodemolition is not capable of selectively removing high-strength patches or overlays in a bridge deck. The SHA engineer must be aware of the existence of high-strength patches in a deck so that provisions can be made in the plans and specifications for their removal by alternate methods.

The method of payment used in a bridge rehabilitation contract can influence the performance of the work and the job's total cost. In rehabilitation work involving entire surface removal, quantity overruns often cause large job cost overruns for the SHAs. The cost overruns occur on jobs in which hydrodemolition removes a greater volume of concrete than estimated. If this is true, that concrete probably was of low quality and should have been removed to increase the service life of the overlay. The cost increase is not manifested in the hydrodemolition bid item because it is bid not on a square-foot basis but rather in the bridge deck overlay item, which is generally bid by volume.

The bridge deck overlay bid item generally includes the cost of the construction methods, such as placing and finishing, as well as the cost of the material. Although the additional cost in material is justified by an increase in volume, there is no substantial increase in the time or resources required for the placement or finishing of the material. However, because the construction methods are included in the unit volume price for bridge deck overlay, their price will go up relative to the increase in the volume of the overlay material.

This can be avoided by having the overlay material and the construction methods as two separate pay items. The overlay material can be bid and paid for by the volume of material while the construction methods pay item, which includes the placing and finishing of the material, will be paid for by the area. In this way, increases in the volume of material removed will result in increases in the quantity and cost of the overlay material only.

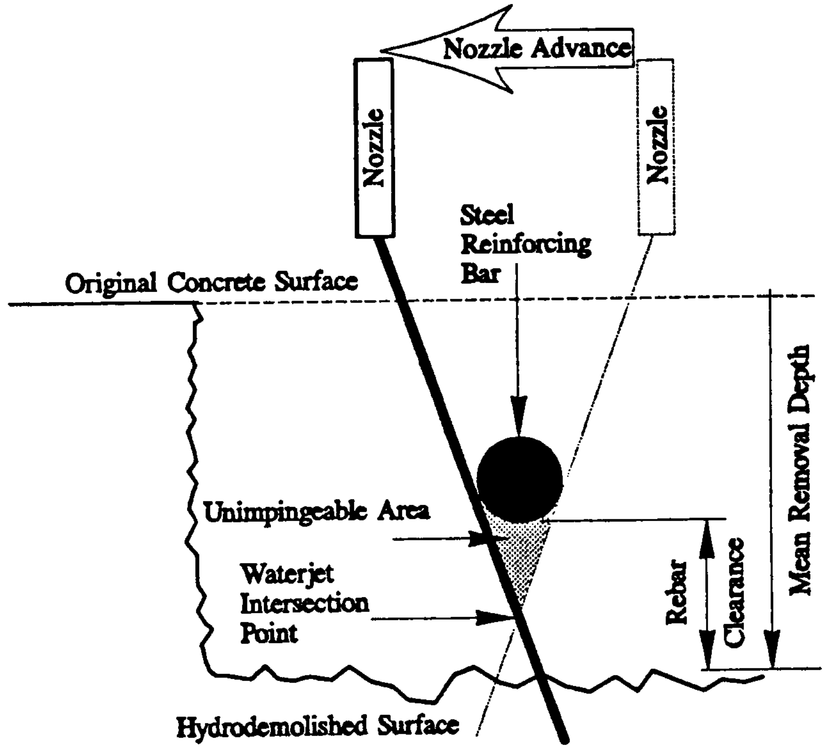


Figure 6.17 Nozzle Advance and Rebar Shadow

6.6 Combined Methods Strengths

The study of comparative abilities showed that each technology had very specific strengths and weaknesses with regard to work characteristics, production, and quality. The following is a brief summary.

1. *Pneumatic breakers*. They are the most flexible and most labor-intensive. They can be used for all sizes and shapes of area, to all depths, and on all bridge structural elements. But production rates are low.

2. *Milling machines*. They are the most inflexible. They can only be used to remove large areas of surface and/or cover concrete on decks, are not labor-intensive but capital-intensive, and have high production rates.

3. *Hydrodemolition*. This technology lies between pneumatic breakers and milling machines in terms of flexibility. Surface, cover, matrix, and core concrete can be removed, but economies are only realized if work is done on

large horizontal areas such as decks. Hydrodemolition also has the advantage of selective removal of poor concrete that might be left in place by the breaker method.

Much can be done by combining the strengths and weaknesses of the methods as determined below.

6.6.1 Milling and Breakers

Combining these two technologies on bridges with relatively minor deterioration in cover and matrix concrete results in the following sequence of operations:

1. Contaminated and deteriorated concrete is milled from the deck to a level conservatively above the reinforcing steel.

2. Areas around joints and drains inaccessible to the milling machine are removed using pneumatic breakers.

3. Areas of deterioration remaining below the reinforcing steel are determined by sounding and removed to the required depth using pneumatic breakers.

4. Deteriorated concrete in the substructure is identified by visual inspection or sounding and removed to the required depth using pneumatic breakers.

5. All exposed steel is sandblasted to remove loose material and rust immediately before patching damaged areas and overlaying the deck.

6.6.2 Hydrodemolition and Breakers

Combining these two technologies on bridges with relatively large areas of deterioration in cover and matrix concrete results in the following sequence of operations:

1. The level of contamination and deterioration in the deck is determined using half-cell potential measurement, chloride sampling, and/or sounding. Large areas that require removal are delineated, or a decision is taken to hydrodemolish the whole deck and rely on the selective removal capability of the technique to identify areas of above or below average deterioration.

2. Contaminated and deteriorated cover and matrix concrete is removed using hydrodemolition equipment calibrated to achieve the desired results.

3. Areas inaccessible to the hydrodemolition equipment and areas of particularly hard concrete are removed using pneumatic breakers.

4. Deteriorated concrete in the substructure is identified by visual inspection or sounding and removed to the required depth using pneumatic breakers.

5. All exposed steel is sandblasted to remove loose material and rust immediately prior to patching damaged areas and overlaying the deck.

6.6.3 Milling, Hydrodemolition, and Breakers

The efficiency of milling large areas of cover concrete can be used to advantage by using a milling machine to remove all the material above the reinforcing steel. This greatly increases the productivity of the hydrodemolition that follows. The reduction in volume of concrete to be removed by hydrodemolition produces a corresponding reduction in the quantity of wastewater produced, and this greatly improves the environmental and safety impacts. Breakers are used to remove the concrete from areas inaccessible to the milling and hydrodemolition machines.

6.6.4 Conclusions

A great deal of skill and experience is required to effectively match the technologies available to the project at hand. Many issues have been discussed throughout this chapter concerning how various tasks (defined in terms of the method used to identify the work, the area of removal, the location of removal, and the depth of removal) relate to the various techniques available. The five factors that dominate the selection process are:

1. *Quality*. Any chosen method must satisfy the quality constraints of selectivity, residual damage, and bond quality. If this is not done, then the method is simply not feasible.

2. *Availability*. The equipment, materials, and skills needed to implement the chosen method must be available and it must be possible to use these on the project site. Specialized subcontractors, manufacturers, and the contractor's own forces all contribute to providing the required resources.

3. *Flexibility*. The chosen method must be sufficiently flexible to accommodate changes in the scheduling, quantity, and pace of work brought about by the unforeseen changes that can occur during a project. Methods that appear to be more productive but that require precise planning and precise knowledge of existing conditions seldom fulfill their promise.

4. *Total cost*. The unit cost of performing the work by a given method is seldom the only criterion. The costs of traffic control, repeated mobilizations brought about by limited access times, and user delays must be factored into the analysis. The relative importance of concrete removal cost to total project cost is another important factor in selecting the chosen method.

5. *Contractual risk.* In most cases the contractor carries the risks associated with the selection of the method to be used. This risk is substantial when large sums must be invested in the equipment needed, when the funding and continuity of work is not clear, and when the contract and specifications appear to inhibit innovation. Many of these factors are under the control of SHA personnel.

Quality, availability, flexibility, total cost, and contractual risk can easily override the technical aspects. These vary with time and location and thus it is extremely difficult to make any firm rules as to which method must be used under all circumstances. This difficulty is somewhat eased by the realization that the methods described are more complementary than competitive. Each has its role to play under given circumstances.

In the context of good planning, highly productive modern equipment would be far more likely to encourage more people to accept and try newer methods.

6.7 References

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Appendix A
Cost-Effective Models and Solution Examples

The generalized models for replacement and rehabilitation are presented in figures A.1 and A.2, respectively. Descriptions of the models as cash flow diagrams are shown at the top of the figures. Each is preceded by the respective generalized mathematical model and notation.

$$EUAC_{Replace} = (A/P,i,N)$$

$$\left[(A-S) + \sum_{m=1}^{l} G_m\,(P/G,f,h_m + 1)(P/F,i,g_m - 1) + \sum_{k=1}^{j} F_k\,(P/F,i,n_k^1)\right] + (S - B)(i) + C$$

where A = replacement structure first cost
B = salvage value of present structure
S = salvage value of replacement structure
C = annual maintenance cost for cleaning deck, drainage system, etc.
F = single future expenditures (e.g., deck overlay, abutment underpinning, painting)
N = life of replacement bridge
G = annual increase in maintenance cost due to progressive deterioration (e.g., deck patching)
n^1 = time to single future expenditure
g = time to beginning of increasing maintenance costs due to progressive deterioration
h = duration of increasing maintenance costs due to progressive deterioration
i = interest rate

$$(A/P) = \textit{funccapital recovery factor } (A/P,i\%,n) = \frac{i(1+i)^n}{(1+i)^n-1}$$

$$(P/F) = \textit{single payment present worth factor } (P/F,i\%,n) = \frac{1}{(1+i)^n}$$

$$(P/G) = \textit{gradient present worth factor } (P/G,i,n) = \frac{1}{i}\left[\frac{(1+i)^n-1}{i(1+i)^n} - \frac{n}{(1+i)^n}\right]$$

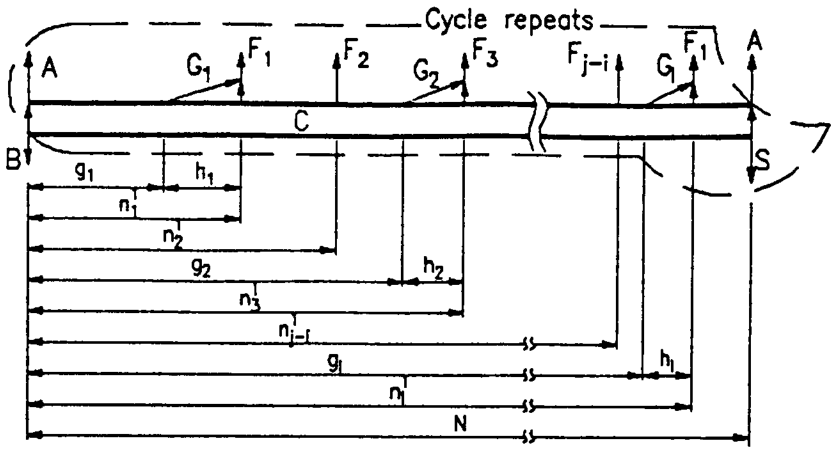


Figure A.1 Replacement Model

$$\text{EUAC}_{Rehab.} = (EUAC_{Replace})(P/F,i,N^1) + i$$

$$\left[D + C(P/A,i,N^1) + \sum_{m=1}^{l} G_m(P/G,i,h_m + 1)(P/F,i,g_m - 1) + \sum_{K=1}^{j} F_k(P/F,i,n_k^1) \right]$$

where D = initial repair cost

N^l = time to required replacement

All other symbols have the same meaning as in the generalized replacement model.

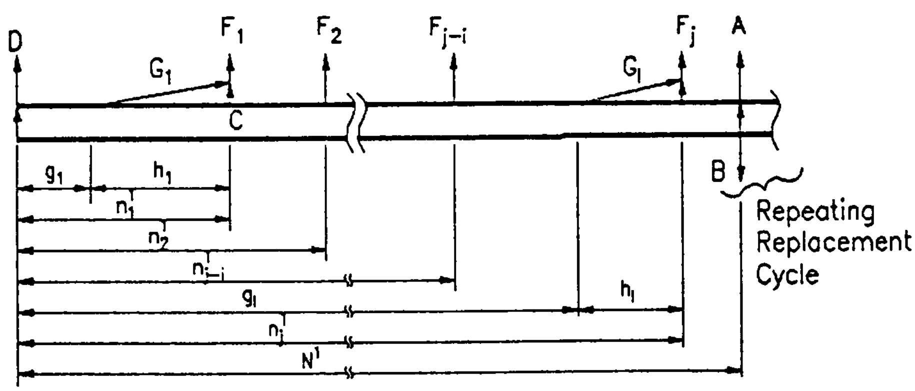


Figure A.2 Protection, Repair, and Rehabilitation Model

Interest and Inflation

Interest rate is the expression of the time value of money. Prevailing lending rates are usually not appropriate in engineering evaluations since they include an inflation factor. The true cost of long-term borrowing is generally considered to be about 4% to 6% (1).

Classical engineering economic evaluation methodology, for the most part, ignores the effects of inflation. The rationale is that if inflation affects all aspects of cash flow in the same manner, its net effect on economic decision-making is nil. However, in the financing of highway maintenance and construction, inflation has not affected all aspects of cash flow equally. Funds for new construction, capital improvements, and maintenance of the nation's highways, at both the state and federal levels, are derived primarily from fixed cents-per-gallon motor fuel taxes.

In the past, these tax revenues increased as fuel consumption increased, and, at relatively low inflation rates, funding kept pace with costs reasonably well. However, with the 1973 oil embargo a very pronounced change occurred. Rapid increases in fuel costs resulted in marked reduction in fuel consumption due to economizing on the part of motorists and the rapid changeover to smaller, more fuel-efficient automobiles. Also, in order to provide incentives for the development of alternative fuel sources, tax exemptions were provided for gasoline-alcohol blends. These factors produced a drastic reduction in the rate of growth in revenues. During the same period, costs increased sharply due to rapidly rising inflation rates. While the rise in future rates may be tempered somewhat, there is every reason to believe that this trend will continue. Thus, we are faced with a scenario where inflation has opposite effects on receipts and disbursements, and therefore engineering economic analysis must take into account the effect of inflation.

The "true" interest rate for the conditions described above is a function of three factors: prevailing interest rate, inflation rate, and rate of increase in funding. It can be shown (2) that the applicable relationship is:

$$i^* = \frac{(1 + i)\,(1 + q)}{(1 + f)} - 1$$

where i^* = "true" interest rate
i = prevailing interest rate
f = inflation rate
q = rate of increase in funding
(Note: all rates are expressed in decimal form.)

Notice that when the effects of inflation are ignored, $i^* = i$.

Using data for the period 1970-1979 (3), the following values for inflation and funding rates were determined:

Using data for the period 1970-1979 (3), the following values for inflation and funding rates were determined:

Inflation rate for highway construction costs: 9.4%
Inflation rate for highway maintenance costs: 7.4%
Increase in funding for highway maintenance and construction: 4.8%

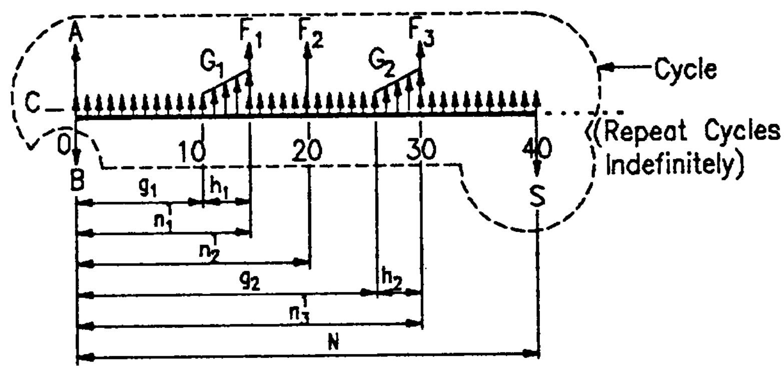
Examples

The following examples illustrate the manner in which the mathematical models are applied and the effects of inflation are accounted for in the selection of the interest rate.

Example 1. The cash flows associated with replacement and rehabilitation (by force account and by contract) for a certain bridge are shown in tables A.1 through A.3. Determine the most cost-effective approach.

A. Assume that the interest rate is the mean of the 4% to 6% usually considered to represent the range of the "true" interest values for long-term investments, i.e., $i = 5\%$.

B. Replacement Structure



Replacement Structure

Table A.1 Cash Flow Table for Bridge Replacement, Example 1

End of Year	Costs ($)	Symbols*	Items
0	45,000; 1,000	A, B	Replacement bridge minus salvage beams of present bridge
1-12	500	C	Annual maintenance and cleaning
13	500; 100	C, G_1	Annual maintenance and cleaning + deck patching
14	500; 200	C, $2G_1$	Annual maintenance and cleaning + deck patching
15	500; 300; 8,000	C, $3G_1$, F_1	Annual maintenance and cleaning + deck patching + deck and drainage repair
16-19	500	C	Annual maintenance and cleaning
20	500;1,500	C, F_2	Annual maintenance and cleaning + underpinning and bearing repair
21-27	500	C	Annual maintenance and cleaning
28	500; 100	C, G_2	Annual maintenance and cleaning + deck patching
29	500; 200	C, $2G_2$	Annual maintenance and cleaning + deck patching
30	500; 300; 6,000	C, $3G_2$, F_3	Annual maintenance and cleaning + deck patching + deck and abutment repair
31-39	500	C	Annual maintenance and cleaning
40	500; 1,500	C, S	Annual maintenance and cleaning minus salvage value or railings and beams
Bridge is replaced after 40 years.			

*For explanation of symbols, see figure A.1

Table A.2 Cash Flow Table for Rehabilitation of Present Structure by Force Account, Example 1

End of Year	Costs (s)	Symbols*	Items
0	7,500	D	Point abutment, repair wings, widen and patch deck, railing treatment, and paint beams
1-5	300	C	Annual maintenance and cleaning
6	300; 100	C, G_1	Annual maintenance and cleaning + deck patching
7	300; 200	C, $2G_1$	Annual maintenance and cleaning + deck patching
8	300; 300	C, $3G_1$	Annual maintenance and cleaning + deck patching
9E	300; 400	C, $4G_1$	Annual maintenance and cleaning + deck patching
10	300; 500; 1,200	C, $5G_1$, F_1	Annual maintenance and cleaning + underpin wings and abutments and paint beams
11-15	500	C	Annual maintenance and cleaning
Bridge is replaced after 15 years.			

*For explanation of symbols, see figures. A.1 and A.2.

Table A.3 Cash Flow for Rehabilitation of Present Structure by Contract, Example 1

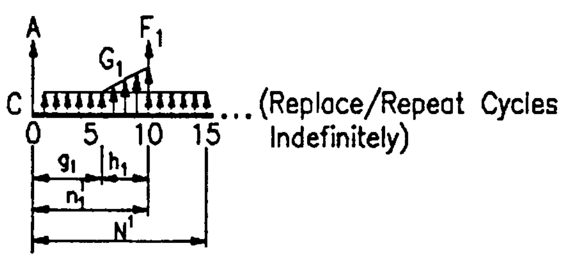
End of Year	Costs	Symbols*	Items
0	12,000	D	Paint abutment, repair wings, deck and railings
1-9	200	C	Annual maintenance and cleaning
10	200; 1,000	C, F_1	Annual maintenance and cleaning + painting
11-19	200	C	Annual maintenance and cleaning
20	200; 1,000	C, F_2	Annual maintenance and cleaning + painting
21-25	200	C	Annual maintenance and cleaning
Bridge is replaced after 25 years.			

*For explanation of symbols, see figures. A.1 and A.2.

Using the Replacement Model:

$$
\begin{aligned}
EUAC_{Replace} &= \overset{0.05828}{(A/P,5\%,40)}\,[(45{,}000 - 1{,}500)] \\
&+ \left[(100)\overset{5.101}{(P/G,5\%,3+1)}\overset{0.5847}{(P/F,5\%,12-1)} + (100)\overset{5.101}{(P/G,5\%,3+1)}\overset{0.2812}{(P/F,5\%,27-1)}\right] \\
&+ \left[(8000)\overset{0.4810}{(P/F,5\%,15)} + (1500)\overset{0.3769}{(P/F,5\%,20)} + (6000)\overset{0.2314}{(P/F,5\%,30)}\right] \\
&+ (1500 - 1000)(0.05) + 500 \\
&= \$3{,}424/\text{yr.}
\end{aligned}
$$

C. Rehabilitate Structure (Force Account)
Using the Rehabilitation Model:



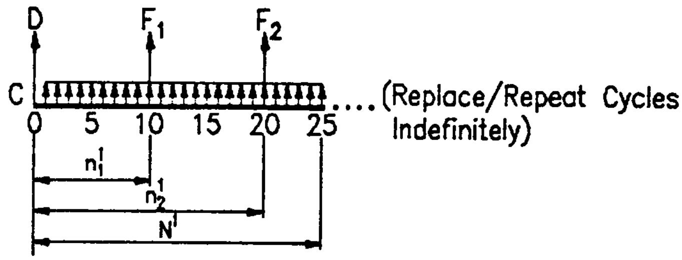
Rehabilitated Structure
(Force Account)

$$EUAC_{Rehab.} = (3424)\overset{0.4810}{(P/F,5\%,15)} + (0.05)$$

$$\left\{7500 + (300)\overset{10.3796}{(P/A,5\%,15)} + \left[(100)\overset{11.966}{(P/G,5\%6)}\overset{0.8227}{(P/F,5\%,4)}\right] + \left[(1200)\overset{0.6139}{(P/F,5\%,10)}\right]\right\}$$

$$= \$2{,}264/yr.$$

D. Rehabilitate Structure (Contract)



Rehabilitated Structure
(Contract)

$$EUAC_{Rehab.} = (3424)\overset{0.2953}{(P/F,5\%,25)} + (0.05)$$

$$\left\{12{,}000 + (200)\overset{14.094}{(P/A,5\%,25)} + \left[(1000)\overset{0.6139}{(P/F,5\%10)} + (1000)\overset{0.3769}{(P/F,5\%,20)}\right]\right\}$$

$$= \$1{,}802/yr.$$

E. Comparison of Rehabilitation Methods
Since \$1,801/yr < \$2,266/yr, contract repair should be chosen.

F. Comparison of Rehabilitation vs. Replacement

$$VM = \$3{,}424/yr - \$1{,}802/yr = + \$1{,}622/yr$$

Therefore, structure should be rehabilitated (by contract)

Annual Savings	Capitalized Savings
\$1,622/yr	\$1,622/0.05 = \$32,440

Example 2. The cash flows associated with replacement and rehabilitation (by force account and by contract) for a certain bridge are shown in Tables A.5 and A.6. Determine the most cost-effective approach.

A. Assume that the interest rate is based on the technique that takes into account the rates of inflation and funding. As previously discussed, the historical rates for the period 1970-1979 are:

Inflation rate for highway construction costs: 9.4%
Inflation rate for highway maintenance costs: 7.4%
Increase in funding for highway maintenance and construction: 4.8%

Since it is not practical to use two interest rates in the same analysis, assume that the combined inflation rate is the average of the rates for construction and maintenance costs:

$$f = \frac{9.4 + 7.4}{2} = 8.4\%$$

Assume that the prevailing interest rate for long-term public financing during the period = 10%.

Therefore, the true interest rate is

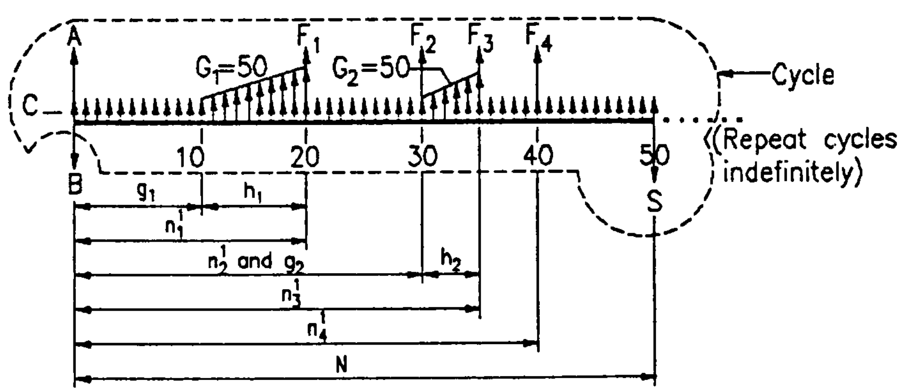
$$i^* = \frac{(1 + i)(1 + q)}{(1 + f)} - 1$$

$$= \frac{(1 + 0.10)\ (1 + 0.048)}{(1 + 0.084)} - 1$$

$$= 0.063 = 6.3\%$$

1. Replacement Structure

Using the Replacement Model:



$$\text{EUAC}_{\text{Replace}} = \underset{(A/P,6.3\%,50)}{0.06612}(44{,}000 - 4{,}000)$$

$$+ \left[(50)\underset{(P/G,6.3\%,10)}{29.020}\underset{(P/F,6.3\%,10)}{0.5428} + (50)\underset{(P/G,6.3\%,6)}{11.312}\underset{(P/F,6.3\%,29)}{0.1700}\right]$$

$$+ \left[(8{,}800)\underset{(P/F,6.3\%20)}{0.2947} + (1{,}030)\underset{(P/F,6.3\%,30)}{0.1600} + (10{,}000)\underset{(P/F,6.3\%,35)}{0.1179}\right.$$

$$\left. + (900)\underset{(P/F,6.3\%,40)}{0.0868}\right] + (4{,}000 - 2{,}000)(0.063) + 500 = \$3{,}595/\text{yr.}$$

Table A.4 Cash Flow Table for Replacement of Present Structure, Example 2

End of Year	Costs ($)	Symbols*	Items
0	44,000; 2,000	A, B	Replacement bridge minus salvage beams of present bridge
1-11	500	C	Annual maintenance and cleaning
12	500; 50	C, G_1	Annual maintenance and cleaning + deck patching
13	500; 100	C, G_1	Annual maintenance and cleaning + deck patching
14	500; 150	C, $2G_1$	Annual maintenance and cleaning + deck patching
15	500; 200	C, $4G_1$	Annual maintenance and cleaning + deck patching
16	500; 250	C, $5G_1$	Annual maintenance and cleaning + deck patching
17	500; 300	C, $6G_1$	Annual maintenance and cleaning + deck patching
18	500; 350	C, $7G_1$	Annual maintenance and cleaning + deck patching
19	500; 400	C, $8G_1$	Annual maintenance and cleaning + deck patching
20	500; 450; 8,800	C, $9G_1$, F_1	Annual maintenance and cleaning + deck patching + deck overlay
21-29	500	C	Annual maintenance and cleaning
30	500; 1,030	C, F_2	Annual maintenance and cleaning + underpin abutment and clean channel
31	500; 50	C, G_2	Annual maintenance and cleaning + deck patching
32	500; 100	C, $2G_2$	Annual maintenance and cleaning + deck patching
33	500; 150	C, $3G_2$	Annual maintenance and cleaning + deck patching
34	500; 200	C, $4G_2$	Annual maintenance and cleaning + deck patching
35	500; 250; 10,000	C, $5G_2$, F_3	Annual maintenance and cleaning + deck patching + deck overlay
36-39	500	C	Annual maintenance and cleaning
40	500; 900	C, F_4	Annual maintenance and cleaning + repair bearing areas
41-49	500	C	Annual maintenance and cleaning
50	500; 4,000	C, S	Annual maintenance and cleaning minus salvage value of beams and railings

Bridge is replaced after 50 years.

*For explanation of symbols, see figure A.1.

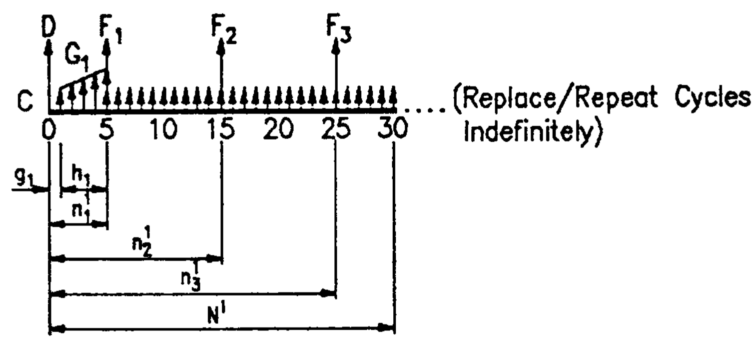
Table A.5 Cash Flow Table for Rehabilitation of Present Structure, Example 2

End of Year	Costs ($) Force Account	Contract	Symbols*	Items
0	20,000	26,000	D	Underpin and pressure point abutments
1	600	600	C	Annual maintenance and cleaning
2	600; 50	600; 50	C, G_1	Annual maintenance and cleaning + deck patching
3	600; 100	600; 100	C, $2G_1$	Annual maintenance and cleaning + deck patching
4	600; 150	600; 150	C, $3G_1$	Annual maintenance and cleaning + deck patching
5	600; 200; 28,000	600; 200; 25,000	C, $4G_1$, F_1	Deck replacement and add more stringers
6-14	600	600	C	Annual maintenance and cleaning
15	600; 3,000	600; 3,500	C, F_2	Annual maintenance and cleaning + painting
16-24	600	600	C	Annual maintenance and cleaning
25	600; 3,000	600; 3,500	C, F_3	Annual maintenance and cleaning + painting
26-30	600	600	C	Annual maintenance and cleaning

Bridge is replaced after 30 years.

*For explanation of symbols, see figures A.1 and A.2.

2. Rehabilitate Structure



Rehabilitated Structure

(Note: The same cash flow diagram applies to force account and contract rehabilitation in this case; only the values of factors are different.)

(a) Force Account
Using the Rehabilitation Model:

$$EUAC_{Rehab.} = (3595)\overset{0.1600}{(P/F,6.3\%,30)} + 0.063 \Bigg\{ 20{,}000 + (600)\overset{13.3340}{(P/A,6.3\%,30)}$$

$$+ (50)\overset{7.847}{(P/G,6.3\%,5)}\ \overset{1.0000}{(P/F,6.3\%,0)} + \Bigg[(28{,}000)\overset{0.7368}{(P/F,6.3\%,5)}$$

$$+ (3{,}000)\overset{0.4000}{(P/F,6.3\%,15)} + (3{,}000)\overset{0.2171}{(P/F,6.3\%,25)}\Bigg]\Bigg\} = \$3{,}780/\text{yr.}$$

(b) Contract
Using the Rehabilitation Model:

$$EUAC_{Rehab.} = (3595)\overset{0.1600}{(P/F,6.3\%,30)} + 0.063 \Bigg\{ 26{,}000 + (600)\overset{13.3340}{(P/A,6.3\%,30)}$$

$$+ (50)\overset{7.847}{(P/G,6.3\%,5)}\ \overset{1.0000}{(P/F,6.3\%,0)} + \Bigg[(25{,}000)\overset{0.7368}{(P/F,6.3\%,5)} + (3{,}500)\overset{0.4000}{(P/F,6.3\%,15)}$$

$$+ (3{,}500)\overset{0.2171}{(P/F,6.3\%,25)}\Bigg]\Bigg\} = \$4{,}038/\text{yr.}$$

3. Comparisons

Rehabilitate:	Force Account	\$3,780/yr
	Contract	\$4,038/yr
Replace:		\$3,595/yr

Choose replacement.

B. Same Situation, Ignoring Inflation
Ignoring the effect of inflation means that $f = q$, and therefore $i^* = i = 10\%$.

1. Replacement Structure
Using the Replacement Model:

$$EUAC_{Replace} = \overset{0.10086}{(A/P,10\%,50)} \{(44{,}000 - 4{,}000)$$

$$+ \left[(50)\overset{22.891}{(P/G,10\%,10)}\ \overset{0.3855}{(P/F,10\%,10)} + (50)\overset{9.684}{P/G,10\%,6)}\ \overset{0.0630}{(P/F,10\%,29)}\right]$$

$$+ \left[(8{,}800)\overset{0.1486}{(P/F,10\%,20)} + (1{,}030)\overset{0.0573}{(P/F,10\%,30)} + (10{,}000)\overset{0.0356}{(P/F,10\%,35)}\right.$$

$$\left. + (900)\overset{0.0221}{(P/F,10\%,40)}\right]\Bigg\} + (4{,}000 - 2{,}000)(0.10) + 500 = \$4{,}958/yr.$$

2. Rehabilitate Structure
   (a) Force Account
   Using the Rehabilitation Model:

$$EUAC_{Rehab.} = (4958)\overset{0.0573}{(P/F,10\%,30)} + 0.100\Big\{(20.000) + (600)\overset{9.4269}{(P/A,10\%,30)}$$

$$+ (50)\overset{6.862}{(P/G,10\%,5)}\overset{1.000}{(P/F,10\%,0)}$$

$$+ \left[(28{,}000)\overset{0.6209}{(P/F,10\%,5)} + (3{,}000)\overset{0.2394}{(P/F,10\%,15)} + (3{,}000)\overset{0.0923}{(P/F,10\%,25)}\right]\Bigg\} = \$4{,}722/yr.$$

   (b) Contract Rehabilitation
   Using the Rehabilitation Model:

$$EUAC_{Rehab.} = (4958)\overset{0.0573}{(P/F,10\%,30)} + 0.100\Big\{(26{,}000) + (600)\overset{9.4269}{(P/A,10\%,30)}$$

$$+ (50)\overset{6.862}{(P/F,10\%,5)}\overset{1.000}{(P/F,10\%,0)}$$

$$+ \left[(25{,}000)\overset{0.6209}{(P/F,10\%,5)} + (3{,}500)\overset{0.2394}{(P/F,10\%,15)} + (3{,}500)\overset{0.0923}{(P/F,10\%,25)}\right]\Bigg\} = \$5{,}152/yr.$$

3. Comparisons

<u>Rehabilitate</u>: Force Account: $4,722/yr

Contract: $5,152/yr

<u>Replace</u>: $4,958/yr

Choose rehabilitation by force account.

C. Effect of Ignoring Inflation in this Case.

1. Choice of alternative:

Changes choice of alternative from replacement to rehabilitative (by force account).

2. Magnitude of effect:

Since the relationship used in accounting for inflation is based on the real present value of inflated future costs, comparisons will have to be based on present worth.

The equivalent uniform annual costs (EUAC) computed using the model are for perpetual service. The present worth for perpetual service, also called "capitalized cost," is EUAC divided by the interest rate (expressed as a decimal).

Therefore, the capitalized cost for the choice when inflation is taken into account (replacement) =

$$\frac{2595}{i^*} = \frac{3595}{0.063} = \$57{,}063$$

and the capitalized cost for the choice when inflation is ignored (rehabilitation by force account) =

$$\frac{4722}{i} = \frac{4722}{0.100} = \$47{,}220$$

Therefore, if inflation is ignored in this case, the "real" present value for the least-cost alternative is understated by nearly $10,000 (17.2%).

A cost-effective methodology has been presented that, when used as a management tool, should optimize the use of limited funds. The mathematical models for rehabilitation and replacement were developed from generalized cash flow diagrams. The models present a least-cost solution to bridge work based on the service life of the bridge and considering the time value of money. The effects of inflation can also be taken into account. The solutions of the mathematical models are used in a value management (VM) term. The magnitude of the VM term represents the dollars saved, and the sign of the VM term indicates whether the structure should be rehabilitated (positive) or replaced (negative). A microcomputer program is available to solve the mathematical models (Richard E. Weyers, Virginia Polytechnic Institute and State University, Department of Civil Engineering, 200 Patton Hall, Blacksburg, VA 24061-0105).

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