Concrete Repair Guide

Reported by ACI Committee 546

Jay H. Paul
Chair

Paul E. Gaudette
Secretary

James P. Barlow	Yelena S. Golod	Kevin A. Michols	Joe Solomon
Paul D. Carter	Harald G. Greve	Richard Montani	Michael M. Sprinkel
Michael M. Chehab	Robert F. Joyce	Myles A. Murray	Ronald R. Stankie
Marwan A. Daye	Lawrence F. Kahn	Thomas J. Pasko	George I. Taylor
Floyd E. Dimmick	Brian F. Keane	Don T. Pyle	Alexander Vaysburd
Peter H. Emmons	Benjamin Lavon	Richard C. Reed	D. Gerald Walters
Michael J. Garlich	Kenneth M. Lozen	Johan L. Silfwerbrand	Patrick M. Watson
Steven H. Gebler	James E. McDonald	W. Glenn Smoak	Mark V. Ziegler
I. Leon Glassgold			

This document provides guidance on the selection and application of materials and methods for the repair, protection, and strengthening of concrete structures. An overview of materials and methods is presented as a guide for making a selection for a particular application. References are provided for obtaining in-depth information on the selected materials or methods.

Keywords: anchorage; cementitious; coating; concrete; joint sealant; placement; polymer; reinforcement; repair.

CONTENTS

ACI 546R-04 supersedes ACI 546R-96 and became effective September 23, 2004.

CHAPTER 1—INTRODUCTION

1.1—Use of this document

This document provides guidance on selection of materials and application methods for the repair, protection, and strengthening of concrete structures. The information is applicable to repairing damaged or deteriorated concrete structures, correcting design or construction deficiencies, or upgrading a structure for new uses or to meet more restrictive building codes.

This guide summarizes current practices in concrete repair and provides sufficient information for the initial planning of repair work and for selecting suitable repair materials and application methods for specific conditions. Many of the topics covered in this guide are more extensively covered in other ACI committee documents. Readers of this guide should refer to the appropriate documents of other ACI committees and other industry resources for additional information.

1.2—Definitions

corrosion—destruction of metal by chemical, electrochemical, or electrolytic reaction within its environment.

dampproofing—treatment of concrete or mortar to retard the passage or absorption of water, or water vapor, either by application of a suitable admixture or treated cement, or by use of preformed film, such as polyethylene sheets, placed on grade before placing a slab.

excavation—steps taken to remove deteriorated concrete, sound concrete, or both, designated for removal.

nonstructural repair—a repair that addresses local deterioration and is not intended to affect the structural capacity of a member.

protection—the procedure of shielding the concrete structure from environmental and other damage for the purpose of preserving the structure or prolonging its useful life.

repair—to replace or correct deteriorated, damaged, or faulty materials, components, or elements of a structure.

repair systems—the combination of materials and techniques used in the repair of a structure.

strengthening—the process of restoring the capacity of damaged components of structural concrete to its original design capacity, or increasing the strength of structural concrete.

structural repair—a repair that re-establishes or enhances the structural capacity of a member.

surface preparation—steps taken after removal of deteriorated concrete, including conditioning of the surface of substrate at bond line and the cleaning of existing reinforcing steel.

waterproofing—prevention of the passage of water, in liquid form, under hydrostatic pressure.

1.3—Repair methodology

A basic understanding of the causes of concrete deficiencies is essential to perform meaningful evaluations and successful repairs. If the cause of a deficiency is understood, it is much more likely that an appropriate repair system will be selected and, consequently, that the repair will be successful and the maximum life of the repair will be obtained.

Symptoms or observations of a deficiency should be differentiated from the actual cause of the deficiency, and it is imperative that causes and not symptoms be dealt with wherever possible or practical. For example, cracking can be a symptom of distress that may have a variety of causes such as drying shrinkage, thermal cycling, accidental overloading, corrosion of embedded metal, or inadequate design or construction. Only after the cause or causes of deficiency are determined can rational decisions be made regarding the selection of a proper repair system and the implementation of the repair process (Fig. 1.1).

1.3.1 *Condition evaluation*—The first step in concrete repair is to evaluate the current condition of the concrete structure. This evaluation may include a review of available design and construction documents, structural analysis of the structure in its deteriorated condition, a review of available test data, a review of records of any previous repair work, review of maintenance records, a visual inspection of the structure, an evaluation of corrosion activity, destructive and nondestructive testing, and review of laboratory results from chemical and petrographic analysis of concrete samples. Upon completion of this evaluation step, the personnel responsible for the evaluation should have a thorough understanding of the condition of the concrete structure and be able to provide insights into the causes of the observed deterioration or distress. Additional information on conducting surveys can be found in ACI 201.1R, 222R, 224.1R, 228.2R, 364.1R, and 437R.

1.3.2 *Determination of causes of deterioration or distress*—After the condition evaluation of a structure has been completed, the deterioration mechanism that caused the

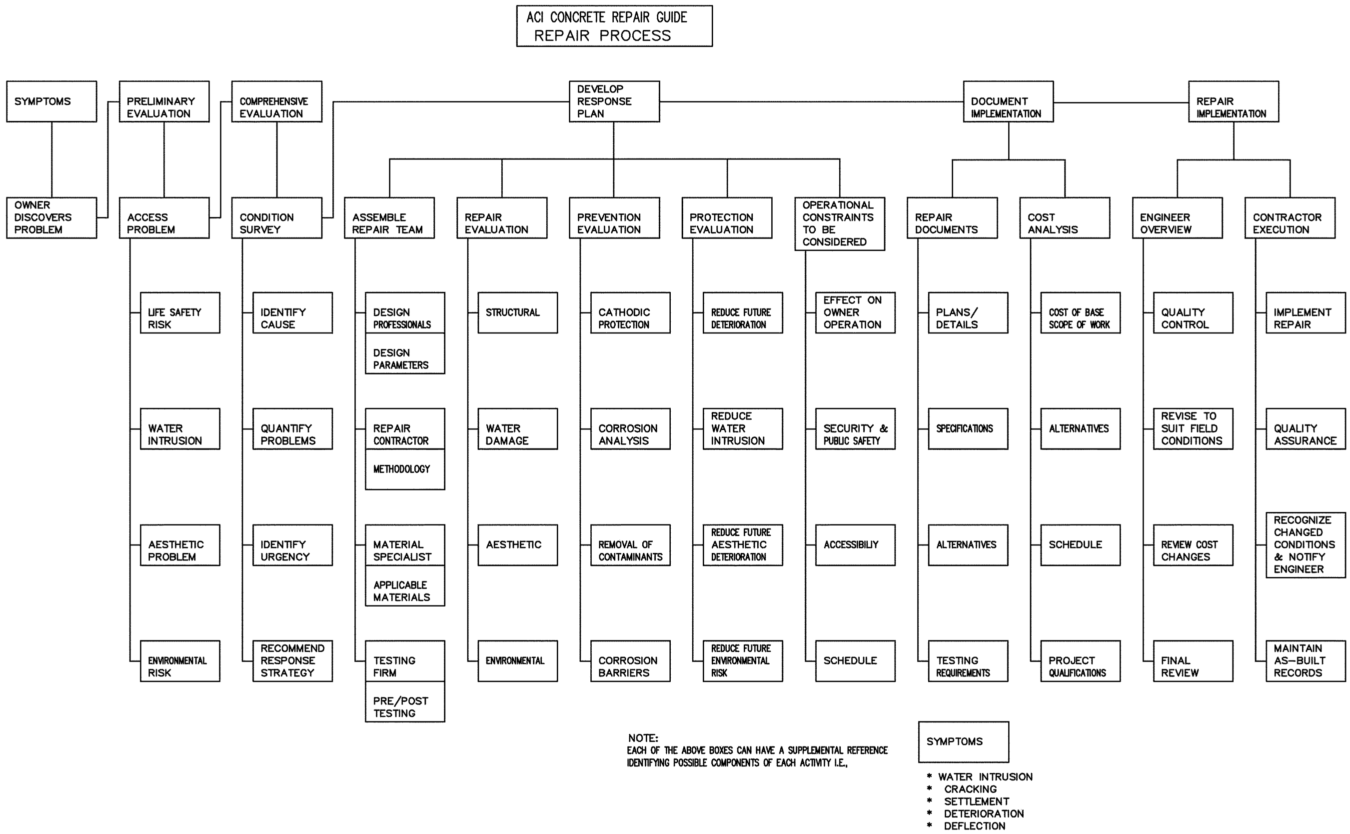

Fig. 1.1—Repair process.

deficiency should be determined. Because many deficiencies are caused by more than one mechanism, a basic understanding of the causes of concrete deterioration is essential to determine what has actually happened to a particular concrete structure and why. Only after the evaluation is complete can a suitable remedial action plan be developed, repair materials selected, and drawings and specifications prepared.

1.3.3 *Selecting repair methods and materials*—After the cause or causes of the damage or deterioration have been determined, appropriate repair materials and methods can be selected considering the following recommendations.

1.3.3.1—The engineer can incorporate adjustments or required modifications to remedy the cause of the deterioration, such as changing the water drainage pattern, eliminating sources of cavitation damage, providing for differential movements, or eliminating exposure to deleterious substances. The engineer should have a thorough understanding of factors affecting the durability of concrete and the causes of different types of cracking (ACI 201.2R, ACI 222R, and ACI 224.1R). It is not always possible to correct causes of deterioration; for example, it may not be possible to change the environment with which the concrete is exposed. In such cases, every reasonable effort should be made to mitigate the problem.

1.3.3.2—The engineer should consider outside constraints, such as limited access to the structure; the operating schedule of the structure; any limitations imposed by the owner of the structure, including the cost; the required useful life of the repaired structure; and the weather implications.

1.3.3.3—Inherent problems that cannot be corrected, such as continued exposure to chlorides in deicing salts or salt water and continued exposure to deleterious chemicals, should also be considered. In these cases, the repair and protection system extends the life of the structure but does not eliminate the causes of deterioration.

1.3.3.4—On some projects, environmental and occupational safety requirements may have a major impact on the selection of materials, methods, or both. Airborne vapor or particles that might result from the use of certain membranes, sealers, and coatings, and from abrasive blasting of silica aggregate contained in concrete is a significant concern. Noise and hazardous waste may be considerations as well. Environmental and occupational safety constraints are governed by specific requirements and policies of owners; the Environmental Protection Agency (EPA); the Occupational, Safety, and Heath Administration (OSHA); and local regulations.

1.3.3.5—The engineer should be aware of advantages and disadvantages of making permanent versus temporary repairs, and select the materials and methods that match the intended life of the repaired structure. Occasionally, there may be situations where it is more appropriate to address the symptoms and not the causes of the problem; for example, it may be more cost effective to temporarily shore the structure.

1.3.3.6—Structural safety before, during, and after the repair should be considered. Repair work often involves the removal of concrete and reinforcing steel that reduces the shear, bending, tensile, and compression capacities, or stability of the structure. Structural analysis, if necessary, should be performed for live and dead loads and the effects of volume changes resulting from temperature changes. Areas of special concern include negative moment areas in slabs and beams, cantilever beams, joint and connection details, precast spandrel beams, columns, and live loads imposed by repair equipment and material storage. Also, any requirements for temporary supports, shoring, and strengthening should be determined.

1.3.3.7—Availability of repair materials and methods, cost effectiveness, and the technical feasibility of using them should be considered. Manufacturers or suppliers can provide assistance in the selection of repair materials and application techniques. When selecting the appropriate repair material, one should keep in mind that the technical data presented in manufacturer's literature may not be sufficient because the tests performed may not be representative of the use of the material in a particular application.

1.3.3.8—The engineer should try to determine the extent of reinforcement deterioration and if there is a need for temporary shoring. If the steel reinforcement becomes partially or totally ineffective, has corroded, or has been cut during concrete removal at critical locations, such as in columns, at the ends of beams, or on the tension sides of beams, the redistribution of loading stress in the element's cross section takes place. In most cases, unless the dead and live loads are removed before repair work, the repaired portion of the element is not effective in supporting existing loads. In short, the repair is nonstructural. In those cases where restoration of load-carrying capacity is a primary concern, existing loads need to be removed and members need to be jacked-up and shored to support dead loads.

1.3.3.9—The capabilities of potential contractors to use specialized repair materials and techniques and to successfully execute repair procedures should be evaluated, considering both initial and long-term costs. Contractors should be capable of demonstrating their expertise in this type of construction.

1.3.4 *Preparation of drawings and specifications*—The next step in the repair process is the preparation of project drawings, specifications, or both. Because the full extent of concrete damage may not be completely known until concrete removal begins, drawings and specifications for repair projects should be prepared with as much flexibility as possible regarding work items such as concrete removal, surface preparation, reinforcement replacement, and quantities of repair materials. A comprehensive condition survey performed as close as possible to the time that the repair work is executed may help to minimize variations in estimated quantities of actual repair work.

When deterioration is particularly severe or where extensive concrete removal is anticipated, provisions for temporary structural support should be included in the project documents. Protection of the repair site and adjacent areas may present unique problems during the execution of a repair project. Special attention should be given to shoring and bracing of structural components under repair and adjacent framing. Shoring may be particularly necessary for slab and beam repairs and, in some cases, for column repairs. Redistribution of loading during repair is important for continuous slabs, beams, or girder systems, and is especially critical for unbonded prestressed structures. Contingency provisions should be included in the drawings and specifications for addressing potential increases in the scope of work.

Effective repair specifications should be clear and concise. Repair specifications should include the quantities of the repair units, the scope of the work, the materials requirements, surface preparation, the application considerations, and the performance testing standards with reference to specific requirements such as tensile strength, surface profiling, compressive strength, bond strength, and related support documents. Details of the concrete repair should be provided, parameters for concrete removal should be defined and, if possible, boundaries of concrete removal and replacement should be identified along with any special features of repair system installation that are necessary. Special attention to the details of reinforcement repair or replacement areas and the preparation of existing concrete before surface protection system application is required. The contract documents should provide sufficient information to the prospective contractors on concrete condition that were found during the investigation stage.

1.3.5 *Bid and negotiation process*

1.3.5.1 *Selection of a contractor*—One of the most important aspects of a repair project is the selection of a qualified contractor or the preparation of a list of qualified bidders. No repair contractors are proficient in all phases of all types of repair work. If possible, select contractors who have shown evidence of expertise and successful completion in each type of repair work included in the project.

1.3.5.2 *Prebid conference*—Before bids are submitted, the engineer, owner, and potential contractors should have a prebid conference to answer any questions regarding the project documents as well as defining the intent of the document, expectations of the owner, or both. If possible, this meeting should include a visit to the repair site. This meeting can go a long way toward eliminating disputes and claims during the construction process.

1.3.5.3 *Preconstruction conference*—Before starting a project, it is important to have the owner and engineer meet the project manager and superintendent. At this meeting, the contractor should be prepared to present the schedule to determine its acceptability to the owner and determine if there are going to be any conflicts with daily operations that should be resolved. Defining the frequency and types of reports helps all parties concerned with ensuring the means and methods that are used to communicate progress, discovered items, and construction problems.

1.3.6 *Execution of the work*—Repair work is executed in accordance with project drawings and specifications. The repair process generally consists of deteriorated or damaged concrete removal, surface preparation, the installation of repair materials, and the implementation of specified repair techniques. Protection may also be incorporated into the repair process.

1.3.6.1 *General*—Experience shows that concrete repair work requires much greater attention to details and good practice than new construction. The contractor should follow specifications and use proper procedures and techniques to achieve the desired results. On any repair project, unexpected or concealed conditions may be revealed as work progresses.

1.3.6.2 *Shoring/work sequences*—The repair process, including concrete removal and reinforcing repair, may alter the force and load distribution within and between reinforced concrete members. It is essential that the contractor, in conjunction with the engineer, determine the extent and sequence of shoring and repair that satisfies structural safety and force distribution.

1.3.6.3 *Concrete removal*—The removal of deteriorated, damaged, or defective concrete requires special attention to the identification and extent of unsound and sound concrete to be removed, the type of removal techniques, and the protection of existing reinforcing during removal. These factors should be addressed to maintain structural integrity and avoid damage to the structure while implementing the repairs.

1.3.6.4 *Surface preparation*—The success of the repair is highly dependent on adequate surface preparation before repair material installation. The surface preparation required depends on the concrete removal technique and the type of repair being undertaken. Surface preparation includes the repair, replacement, and installation of supplemental reinforcement.

1.3.6.5 *Repair techniques and material installation*—The proper repair techniques and material selection and installation are essential for a successful project that is capable of providing the serviceability and durability of any repair.

1.3.6.6 *Protective systems*—Protective systems, such as coatings, sealers, and membranes, may be incorporated into the repair process to extend the service life of the repairs and reduce and minimize further deterioration in the structure. Protective systems provide varying types of protection to the repair and structure and should be selected to satisfy project serviceability requirements.

1.3.7 *Quality control*— All of the best efforts in designing a repair and specifying the optimum materials are at risk if the implementation lacks the appropriate quality control measures to ensure a successful installation. ACI 311.4R provides guidance on setup of quality control, inspection, and testing procedures. A written quality plan should be prepared and presented to the engineer for approval. ACI 311.1R (SP-2) and ACI 311.5R provide guidance on inspection and testing.

1.3.7.1 *Performance objectives*—A clear understanding forms the basis for a successful repair. The repair documents should define the structural, environmental, aesthetic, and protection objectives to be achieved. Mockups are recommended where appropriate, such as in those instances where the color or texture must meet the surrounding concrete (ACI 311.1R, 311.4R, and 311.5R).

1.3.7.2 *Quality control procedures during the repair*—The successful completion of the repair procedure can be verified by continuing quality control measures during the course of the work.

- Such measures include the appropriate safety measures to ensure the safety of all personnel and the structure being repaired throughout the course of the work;
- Periodic reviews by the engineer and implementation of all testing and inspection requirements are essential to meeting the performance objectives established by the engineer;
- Adequate supervision by the contractor should include daily monitoring of temperature, humidity, and weather conditions that may affect the repair procedure;
- The contractor should pay attention to the specification requirements for appropriate shoring, repair preparation, material installation, and subsequent curing; and
- Accurate record keeping should include repair locations, repair modifications, quantities installed, site visits by the engineer, weather conditions, changes to the engineering documents, and periodic progress reports.

1.3.7.3 *Testing or inspection agency qualifications*—This agency should meet the requirements of ASTM E 329. Personnel should be ACI certified as technicians or inspectors as appropriate.

1.3.8 *Maintenance after completion of repairs*—Lack of adequate maintenance may result in premature failure of the repair or surrounding areas. Therefore, appropriate maintenance procedures after completion of a concrete repair project is recommended, including the following:

- Warranty documentation should be obtained, when applicable, upon completion of the repair project;
- Warranty conditions should be adhered to ensure warranty coverage is maintained, such as periodic inspections or recoating of membranes or sealers;
- All plan and specification documents from the repair project along with the original building plans and specifications (when available) should be maintained;
- Performance of repairs should be monitored by documenting subsequent changes of the concrete repairs or surface applied systems;
- Proactive monitoring systems, such as acoustic monitoring for post-tensioned structures, should be activated and maintained where applicable; and
- Continued evaluation of the structure by periodic engineering reviews should be programmed.

1.4—Design considerations

Although the contractor is fully responsible for construction safety, when designing a concrete repair, strengthening system, or both, the engineer should consider the safety and serviceability of the structure during the construction and the performance of the structure upon completion. The rehabilitated structure may be required to provide equivalent safety to that of a new structure; typically, the rehabilitated structure will have to be brought to current building code standards if an area greater than some percentage of the total is being rehabilitated. Generally, designed repairs fall under equivalent safety provisions and the engineer should judge how the repair affects the safety of the structural components and the overall structure. Often, code equations, like those in ACI 318, provide only guidance. The engineer should apply basic principles of structural mechanics and have an understanding of material behavior to evaluate and design a structural repair, strengthening procedure, or both. Some design considerations follow and are discussed throughout this guide.

1.4.1 *Current load distribution*—In a deteriorated state, a structural member or system distributes dead and live loads differently than first assumed when the structure was new. Cracking, deteriorated concrete and corroded reinforcement alter the stiffness of members, which leads to changes in shear, moment, and axial load distribution. As concrete and reinforcement are removed and replaced during the repair operation, these redistributed forces are further modified. To understand the final behavior of the structural system, the engineer should evaluate the redistribution of the forces. To fully re-establish the original load distribution, a member should be relieved of the load by jacking or other means. The repaired member and the repair itself supports the loads differently than would be assumed in the original or a new structure.

1.4.2 *Compatibility of materials*—If a repair and the member have the same stiffness—for example, modulus of elasticity—the analysis of the repaired member may be the same as a new section. If the stiffness varies, however, then the composite nature of the repaired system should be considered. A mismatch of other material characteristics further exacerbates the effects of thermal changes, vibrations, and long-term creep and shrinkage effects. Different coefficients of thermal expansion of the repair and original material results in different dimensional changes. The engineer should design for the different movements, or the repair system should be similar to the thermal and dimensional characteristics of the original material.

1.4.3 *Creep, shrinkage, or both*—Reduction in length, area, or volume of both the repair and original materials due to creep, shrinkage, or both, affect the structures serviceability and durability. As an example, compared with the original material, high creep or shrinkage of repair materials results in loss of stiffness of the repair, redistributed forces, and increased deformations. The engineer should consider these effects in the design.

1.4.4 *Vibration*—When the installed repair material is in a plastic state or until adequate strength has been developed, vibration of a structure can result in reduced bonding of the repair material. Isolating the repairs or eliminating the vibration may be a design consideration.

1.4.5 *Water and vapor migration*—Water or vapor migration through a concrete structure can degrade a repair. Understanding the cause of the migration and controlling it should be part of a repair design consideration.

1.4.6 *Safety*— The contractor is responsible for construction safety. Nevertheless, as the engineer considers a repair design, which may involve substantial concrete removal, steel reinforcing cutting, or both, he or she should notify the contractor of the need and extent of shoring and bracing. The local repair of one small section can affect a much larger area, of which the contractor may not be aware.

1.4.7 *Material behavior characteristics*—When new and innovative materials and systems are used for repair and strengthening, the structural behavior of the repaired section can differ substantially from the behavior of the original section. For example, if a beam's steel reinforcement has corroded extensively and lost part of its load-carrying capacity, the steel reinforcement may be replaced by carbon fiber-reinforced polymer (CFRP) applied to the external bottom face of the beam. The original yielding behavior of the steel bar is replaced by FRP that is stronger, but has a more elastic and brittle behavior. The behavior assumptions of codes like ACI 318 are no longer valid. The engineer should consider the behavior and performance of the new repair under the actual service and ultimate load, and design the repair to provide at least an equivalent level of safety to the original design. Such a design is outside the scope of ACI 318.

1.5—Format and organization

Chapter 2 discusses removal of deteriorated concrete, preparation of surfaces to receive repair materials, general methods for concrete repair, and repair techniques for reinforcing and prestressing steel. Chapter 3 discusses various types of repair materials that may be used. The reader is urged to use Chapters 2 and 3 in combination when selecting the repair material and method for a given situation. Chapter 4 describes materials and systems that may be used to protect repaired or unrepaired concrete from deterioration. Chapter 5 provides methods for strengthening an existing structure when repairing deficiencies, improving load-carrying capabilities, or both. Chapter 6 provides references, including other appropriate ACI documents and industry resources.

CHAPTER 2—CONCRETE REMOVAL, PREPARATION, AND REPAIR TECHNIQUES

2.1—Introduction and general considerations

This chapter covers removal, excavation, or demolition of existing deteriorated concrete, preparation of the concrete surface to receive new material, preparation and repair of reinforcement, methods for anchoring repair materials to the existing concrete, and various methods that are available to place repair materials. The care that is exercised during the removal and preparation phases of a repair project can be the most important factor in determining the longevity of the repair, regardless of the material or technique used.

Specific attention should be given to the removal of concrete around prestress strands, both bonded and unbonded. The high-energy-impact tools, such as chipping hammers, should avoid contact with the strand because this will reduce the strands' load-carrying capacity and may cause the wire(s) to rupture, which may lead to strand failure.

2.2—Concrete removal

A repair project usually involves removal of deteriorated, damaged, or defective concrete. In most concrete repair projects, the zones of damaged concrete are not well defined. Most references state that all damaged or deteriorated material should be removed, but it is not always easy to determine when all such material has been removed or when too much good material has been removed. A common recommendation is to remove sound concrete for a defined distance beyond the delaminated area; thereby, exposing the reinforcing steel beyond the point of corroded steel.

Removal of concrete using explosives or other aggressive methods can damage the concrete that is intended to remain in place. For example, blasting with explosives or the use of some impact tools heavier than 12 kg (30 lb) can result in additional delamination or cracking. Delaminated areas can be identified by using a hammer to take soundings. In most cases, such delaminations should be removed before repair materials are placed.

Removal of concrete using impact tools may result in small-scale microcracking damage (termed bruising) to the surface of the concrete left in place. Unless this damaged layer is removed, a weakened plane may occur in the parent concrete below the repair material bondline. This condition can result in a low tensile rupture (bond) strength between the parent concrete and repair material. Thus, a perfectly sound and acceptable replacement material may fail due to improper surface preparation. All damaged or delaminated concrete, including bruising, at the interface of the repair and the parent concrete should be removed before placing the repair material. This may require one type of aggressive removal for gross removal followed by another type of removal for bruising.

In all cases in which concrete has been removed from a structure by primary means such as blasting or aggressive impact methods, the concrete left in place should be prepared by using a secondary method, such as chipping, abrasive blasting, or high-pressure water jetting, to remove any remaining damaged surface material. Careful visual inspections of the prepared surfaces should be conducted before placing repair materials. Wetting the surface may help to identify the presence of cracking. Determination of the tensile strength (ACI 503R, Appendix A) by pull-off testing is advisable on prepared surfaces to determine the suitability of the surface to receive repair material.

Removal of limited areas of concrete in a slab, wall, or column surface requires saw-cutting the perimeter of the removal area, providing an adequate minimum thickness of repair material at the edge of the repaired area, and mitigating the advancement of undetected incipient cracking. Feathering of repair materials generally should be avoided. The preparation for shotcreting is an exception. ACI 506R recommends tapered edges around the perimeter of such patches. Saw cutting can also improve the appearance of the repaired area. The general shape of the repaired areas should be as symmetrical as possible (ICRI 03730). Reentrant corners should be avoided. Large variations in the depth of removal in short distances should also be avoided. The texture of the prepared surface should be appropriate for the intended repair material (ICRI 03732).

Every precaution should be made to avoid cutting underlying reinforcement. Reviewing design drawings and using a covermeter or similar device provides data as to the location and depth of reinforcement. In addition, the removal of small areas of concrete is commonly used to confirm the location and depth of bars before saw cutting.

Sections 2.2.1 through 2.2.18 present descriptions of many of the commonly used concrete removal techniques to help in the selection process.

2.2.1 *General considerations*—Concrete removal addresses deteriorated and damaged material. Some sound concrete, however, may be removed to permit structural modifications and to ensure that all unsound material is removed. The effectiveness of various removal techniques can differ for deteriorated and sound concrete. Some techniques may be more effective in sound concrete, while others may work better for deteriorated concrete.

Concrete removal techniques selected should be effective, safe, economical, environmentally friendly, and minimize damage to the concrete left in place. The removal technique chosen may have a significant effect on the length of time that a structure will be out of service. Some techniques permit a significant portion of the work to be accomplished without removing the structure from service. The same removal technique, however, may not be suitable for all portions of a given structure. In some instances, a combination of removal techniques may be more appropriate to speed removal and limit damage to the remaining sound concrete. Trial field testing various removal techniques can help confirm the best procedures.

In general, the engineer responsible for the design of the repair should specify the objectives to be achieved by the concrete removal, and the repair contractor should be allowed to select the most economical removal method, subject to the engineer's acceptance. In special circumstances, the engineer may also need to specify the removal techniques to be used and those that are prohibited.

The mechanical properties of the concrete and the type and size of aggregate to be removed provide important information to determine the method and cost of concrete removal. The mechanical properties include compressive and tensile strengths. This information is also necessary for the engineer to specify the prepared surface condition and select the repair material, and it should be made available to contractors for bidding purposes.

2.2.2 *Monitoring and shoring during removal operations*—It is essential to evaluate the removal operations to limit the extent of damage to the structure and to the concrete that remains. Structural elements may require shoring, removal of applied loads, or both, before concrete removal to prevent structural deformations, possible collapse, buckling, or slippage of reinforcement. Care should be used during removal of concrete to avoid cutting and damaging reinforcing steel. Because reinforcement is often misplaced, unanticipated damage may occur when saw cutting, impacting, or removing concrete.

Careful monitoring is required throughout the concrete removal operation. This can be accomplished by visual inspection, sounding, use of a covermeter, or other means to locate reinforcement. The project specifications should assign responsibilities for the inspection of the prepared concrete.

Sounding is an excellent means to detect delaminated concrete adjacent to the outermost layers of reinforcing steel. Subsurface cracks, the extent of deterioration, or other internal defects, however, may not be identified by this method alone. Other means of evaluation should be used to properly identify the extent of concrete to be removed. In addition, sounding usually does not indicate near-surface microcracking or bruising. Only microscopic examination or bond testing may disclose near-surface damage.

Subsurface evaluation (examination of the substrate) can provide valuable information about the condition of the concrete. This information may be obtained by the following methods before, during, or after concrete removal (ACI 228.2R):

a) Taking cores for visual examination, microscopic examination, compressive strength tests, and splitting-tensile strength tests;

b) Pulse-velocity tests;

c) Impact-echo tests;

d) Bond tests (pull-off testing, ACI 503R Appendix A);

e) Covermeter or similar equipment to locate reinforcement and determine its depth below the surface;

f) Infrared thermography; and

g) Ground-penetrating radar (GPR).

Many of these methods are discussed in ACI 228.2R.

2.2.3 *Quantity of concrete to be removed*—In most repair projects, all damaged or deteriorated concrete should be removed; however, the quantity of concrete to be removed is directly related to the elapsed time between preparation of the estimate and actual removal. Substantial overruns are common. Estimating inaccuracies can be minimized by a thorough condition survey as close as possible to the time the repair work is executed. Potential quantity overruns, based on field-measured quantities, should be taken into account. When, by necessity, the condition survey is done far in advance of the repair work, the estimated quantities should be increased to account for continued deterioration. Because most concrete repair projects are based on unit prices, repair areas should be accurately measured before forms are installed. This is usually done jointly by the engineer and the contractor. It is not uncommon for estimated quantities to increase significantly between the detectable quantities and the actual quantity removed. ICRI 03735 provides guidelines for methods of measurement for concrete repair work.

2.2.4 *Classification of concrete removal methods*—Removal and excavation methods can be categorized by the way in which the process acts on the concrete. These categories are blasting, cutting, impacting, milling, hydrodemolition, presplitting, and abrading. Table 2.1 provides a general description of these categories, lists the specific removal techniques within each category, and provides a summary of information on each technique. The techniques are discussed in detail in the following sections.

2.2.5 *Blasting methods*—Blasting methods generally employ rapidly expanding gas confined within a series of bore holes to produce controlled fracture and removal of the concrete. The only blasting method addressed in this report is explosive blasting.

Explosive blasting is the most cost-effective and expedient means for removing large quantities of concrete—for example, portions of large mass concrete foundations or walls. This method involves drilling bore holes, placing an explosive in each hole, and detonating the explosive. Controlled-blasting techniques minimize damage to the material that remains after blasting. One such technique, cushion blasting, involves drilling a line of 75 mm (3 in.) diameter or smaller bore holes parallel to the removal face, loading each hole with light charges of explosive (usually detonating cord) distributed along its length, cushioning the charges by stemming each hole completely or in the collar with wet sand, and detonating the explosive with electric blasting caps. The uniform distribution and cushioning of the light charges produce a relatively sound surface with little overbreak.

Blasting machines and electrical blasting-cap delay series are also used for controlled demolition and employ proper timing sequences to provide greater control in reducing ground vibration. Controlled blasting has been used successfully on numerous repair projects. The selection of proper charge weight, borehole diameter, and borehole spacing for a repair project depends on the location of the structure, the acceptable degree of vibration and damage, and the quantity and quality of concrete to be removed. If at all possible, a pilot test program should be implemented to determine the optimum parameters. Because of the inherent dangers in the handling and usage of explosives, all phases of the blasting project should be performed by qualified, appropriately licensed personnel with proven experience and ability.

2.2.6 *Cutting methods*—Cutting methods generally employ mechanical sawing, intense heat, or high-pressure water jets to cut around the perimeter of concrete sections to permit their removal. The size of the sections that are cut free is governed by the available lifting and transporting equipment. The cutting methods include high-pressure water jets, saw cutting, diamond wire cutting, mechanical shearing, stitch drilling, and thermal cutting.

a) *High-pressure water jet (without abrasives)*—A high-pressure water jet uses a small jet of water driven at high velocities, commonly producing pressures of 69 to 310 MPa (10,000 to 45,000 psi). A number of different types of water jets are currently being used. The most promising are the ultra high-pressure jet and the cavitating jet. Section 2.2.10 describes using a water jet as a primary removal method. Water jets used with abrasives are described in Section 2.2.11.

b) *Saw cutting*—Diamond or carbide saws are available in sizes ranging from small (capable of being hand-held) to large (capable of cutting depths of up to 1.3 m [52 in.]).

c) *Diamond wire cutting*—Diamond wire cutting is accomplished with a wire containing nodules impregnated with diamonds. The wire is wrapped around the concrete mass to be cut and reconnected with the power pack to form a continuous loop. The loop is spun in the plane of the cut while being drawn through the concrete member. This system can be used to cut a structure of any size as long as the wire can be wrapped around the concrete. The limits of the power source determines the size of the concrete structure that can be cut. This system provides an efficient method for cutting up and dismantling large or small concrete structures.

d) *Mechanical shearing*—The mechanical shearing method employs hydraulically powered jaws to cut concrete and reinforcing steel. This method is applicable for making cutouts through slabs, decks, and other thin concrete

Table 2.1— Summary of features and considerations/limitations for concrete removal methods

Category	Features	Considerations/Limitations
2.2.5 *Blasting* Uses rapidly expanding gas confined within a series of boreholes to produce controlled fracture and removal of concrete.	*Explosives* Most expedient method for removing large volumes where concrete section is 10 in. (250 mm) thick or more. Produces good fragmentation of concrete debris for easy removal.	Requires highly skilled personnel for design and execution. Stringent safety regulations must be complied with regarding the transportation, storage, and use of explosives due to their inherent dangers. Blast energy must be controlled to avoid damage to surrounding improvements resulting from air blast pressure, ground vibration, and flying debris.
2.2.6 *Cutting* Uses perimeter cuts to remove large pieces of concrete.	*High-pressure water jet (without abrasives)* Applicable for making cutouts through slabs, decks, and other thin concrete members. Cuts irregular and curved shapes. Makes cutouts without overcutting corners. Cuts flush with intersecting surfaces. No heat, vibration, or dust is produced. Handling of debris is efficient because bulk of concrete is removed in large pieces.	Cutouts for removal limited to thin sections. Cutting is typically slower and more costly than diamond blade sawing. Moderate levels of noise may be produced. Controlling flow of waste water may be required. Additional safety precautions are required due to the high water pressure produced by the system.
2.2.6 *Cutting* (continued)	*Diamond saw* Applicable for making cutouts through slabs, decks, and other thin concrete members. Makes precision cuts. No dust or vibration is produced. Handling of debris is efficient because bulk of concrete is removed in large pieces.	Cutouts for removal limited to thin sections. Performance is affected by type of diamonds and the diamond-to-metal bond in blade segments (segment selection is based on aggregate hardness). The higher the percentage of steel reinforcement in cuts, the slower and more costly the cutting. The harder the aggregate, the slower and more costly the cutting. Controlling flow of waste water may be required.
2.2.6 *Cutting* (continued)	*Diamond wire cutting* Applicable for cutting large and/or thick pieces of concrete. The diamond wire chain can be infinitely long. No dust or vibration is produced. Large blocks of concrete can be easily lifted out by a crane or other mechanical methods. The cutting operation can be equally efficient in any direction.	The cutting chain must be continuous. Access to drill holes through the concrete must be available. Water must be available to the chain. Controlling the flow of waste water may be required. The harder the aggregate and/or concrete, the slower and more costly the cutting. Performance is affected by the quality, type, and number of diamonds as well as the diamond-to-metal bond in the chain.
2.2.6 *Cutting* (continued)	*Mechanical shearing* Applicable for making cutouts through slabs, decks, and other thin concrete members. Steel reinforcement can be cut. Limited noise and vibration are produced. Handling of debris is efficient because bulk of concrete is removed in large pieces.	Limited to thin sections where an edge is available or a hole can be made to start the cut. Exposed reinforcing steel is damaged beyond reuse. Remaining concrete is damaged. Extremely rugged profile is produced at the cut edge. Ragged feather edges remain after removal.
2.2.6 *Cutting* (continued)	*Stitch drilling* Applicable for making cutouts through concrete members where access to only one face is feasible. Handling of debris is more efficient because bulk of concrete is removed in large pieces.	Rotary-percussion drilling is significantly more expedient and economical than diamond core drilling; however, it results in more damage to the concrete that remains, especially at the point of exit from the concrete. Depth of cuts is dependent on accuracy of drilling equipment in maintaining overlap between holes with depth and diameter of the boreholes drilled. The deeper the cut, the greater borehole diameter required to maintain overlap between adjacent holes and the greater the cost. Uncut portions between adjacent boreholes will hamper or prevent the removal. Cutting reinforced concrete increases the cutting time and increases the cost. Aggregate toughness for percussion drilling and aggregate hardness for diamond coring will affect cutting cost and rate. Personnel must wear hearing protection due to high noise levels.

members. It is especially applicable where total demolition of the member is desired. The major limitation of this method is that cuts should be started from free edges or from holes made by hand-held breakers or other means.

e) *Stitch drilling*—The stitch-drilling method employs the use of overlapping boreholes along the removal perimeter to cut out sections for removal. This method is applicable for making cutouts through concrete members where access to only one face is possible, and the depth of cut is greater than can be economically cut by the diamond-blade method. The primary drawback of stitch drilling is the potential for costly removal complications if the cutting depth exceeds the accuracy of the drilling equipment, so that uncut concrete remains between adjacent holes.

f) *Thermal cutting*—This method requires powder torch, thermal lance, and powder lance, which develop intense heat generated by the reaction between oxygen and powdered metals to melt a slot into concrete. The thermal device's ability for removing concrete from structures mainly depends on the rate at which the resulting slag can flow from the slot. These devices use intense heat and are especially effective for cutting reinforced concrete; however, they are considered slow, relatively expensive, and are not widely used.

Table 2.1 (cont.)—Summary of features and considerations/limitations for concrete removal methods

Category	Features	Considerations/Limitations
2.2.6 *Cutting* (continued)	*Thermal cutting* Applicable for making cutouts through heavily reinforced decks, beams, walls, and other thin to medium concrete members. An effective means of cutting reinforced concrete. Cuts irregular shapes. Produces minimal noise, vibrations, and dust.	Limited availability commercially. Not applicable for cuts where slag flow is restricted. Remaining concrete has thermal damage with more extensive damage occurring around steel reinforcement. Produces smoke and fumes. Personnel must be protected from heat and hot slag produced by cutting operation.
2.2.7 *Impacting* Uses repeated striking of the surface with a mass to fracture and spall the concrete.	*Hand-held breakers* Applicable for limited volumes of concrete removal. Applicable where blow energy must be limited. Widely available commercially. Can be used in areas of limited work space. Produces relatively small and easily handled debris.	Performance is a function of concrete soundness and aggregate toughness. Significant loss of productivity occurs when breaking action is other than downward. Removal boundaries will likely require saw cutting to avoid feathered edges. Concrete that remains may be damaged (microcracking). Produces high levels of noise, dust, and vibration.
	Boom-mounted breakers Applicable for full-depth removal from slabs, decks, and other thin concrete members and for surface removal from more massive concrete structures. Can be used for vertical and overhead surfaces. Widely available commercially. Produces easily handled debris.	Blow energy delivered to the concrete may have to be limited to protect the structure being repaired and the surrounding structures from damage due to high cyclic energy generated. Performance is a function of concrete soundness and aggregate toughness. Damages remaining concrete. Damages reinforcing steel. Produces feathered edges. Produces high level of noise and dust.
2.2.7 *Impacting* (continued)	*Scabblers* Low initial cost. Can be operated by unskilled labor. Can be used in areas of limited work space. Removes deteriorated concrete from wall or floor surfaces efficiently. Readily available commercially.	High cyclic energy applied to a structure will produce fractures in the remaining concrete surface area. Produces high level of noise and dust. Limited depth removal.
2.2.8 *Milling* Uses scarifiers to remove concrete surfaces.	*Scarifier* Applicable for removing deteriorated concrete surfaces from slabs, decks, and mass concrete. Boom-mounted cutters are applicable for removal from wall and ceiling surfaces. Removal profile can be controlled. Method produces relatively small and easily handled debris.	Removal is limited to concrete without steel reinforcement. Sound concrete significantly reduces the rate of removal. Can damage concrete that remains (microcracking). Noise, vibration, and dust are produced.
2.2.9 *Hydrodemolition* Uses high-pressure water to remove concrete.	Applicable for removal of deteriorated concrete from surfaces of bridges and parking decks and other deteriorated surfaces where removal depth is 6 in. (150 mm) or less. Does not damage the concrete that remains. Steel reinforcing is left clean and undamaged for reuse. Method produces easily handled, aggregate-sized debris.	Productivity is significantly reduced when sound concrete is being removed. Removal profile will vary with changes in depth of deterioration. Method requires large source of potable water to meet water demand. Waste water may have to be controlled. An environmental impact statement may be required if waste water is to enter a waterway. Personnel must wear hearing protection due to the high level of noise produced. Flying debris is produced. Additional safety requirements are required due to the high pressures produced by these systems.

2.2.7 *Impacting methods*—Impacting methods are the most commonly used concrete removal systems. They repeatedly strike a concrete surface with a high-energy tool or a large mass to fracture and spall the concrete. The use of these methods in partial-depth concrete removal can result in microcracking on the surface of the concrete left in place. Extensive microcracking results in a weakened plane below the bond line. Currently, the committee is unable to provide definitive guidelines to prevent such damage when using impact methods; however, factors such as the weight and size of the equipment should be considered to minimize microcracking. Determination of the tensile strength by pull-off testing is recommended to determine the suitability of the surface to receive repair materials. Additionally, after impacting secondary methods, such as sandblasting, abrasive blasting, and water blasting, may be required to remove excessive microcracking.

a) *Hand-held breakers*—The hand-held breaker or chipping hammer is probably the best known of all concrete removal devices. Hand-held breakers are available in various sizes with different levels of energy and efficiency. These tools are generally defined by weight and vary in size from 3.5 to 41 kg (8 to 90 lb). (Note: the larger the hammer, such as 14 kg [30 lb] and larger, the greater the potential for microcracking.) The smaller hand-held breakers, such as 7 kg (15 lb) and smaller, are used in partial removal of unsound concrete or concrete around reinforcing steel because they do little damage to surrounding concrete. Larger breakers are used for complete removal of large volumes of concrete or delaminations. Care should be exercised when selecting the size of

Table 2.1 (cont.)—Summary of features and considerations/limitations for concrete removal methods

Category	Features	Considerations/Limitations
2.2.10 *Presplitting* Uses hydraulic jacks, water pulses, or expansive agents in a pattern of boreholes to presplit and fracture the concrete to facilitate removal. Large sections can be presplit for removal, thereby making handling of debris more efficient. Development of presplitting plane in direction of boreholes depth is limited. Development of presplitting plane is significantly decreased by presence of reinforcing steel normal to presplitting plane. Presplit opening must be wide enough to allow cutting of steel reinforcement. Secondary means of breakage may be required to complete removal. Loss of control of presplitting plane can result if boreholes are too far apart or if holes are located in severely deteriorated concrete.	*Hydraulic splitter* Applicable for presplitting slabs, decks, walls, and other thin to medium concrete members. Usually less costly than cutting members. Direction of presplitting can be controlled by orientation of wedges and drill hole layout. Can be used in areas of limited access. Limited skills required by operator. No vibration, noise, or fly rock is produced except by the drilling of boreholes and secondary breakage method.	Development of presplitting plane is significantly decreased by presence of reinforcing steel normal to presplitting plane. Presplit opening must be wide enough to allow cutting of steel reinforcement. Secondary means of breakage may be required to complete removal. Loss of control of presplitting plane can result if boreholes are too far apart or if holes are located in severely deteriorated concrete.
2.2.10 *Presplitting* (continued)	*Water pulse splitter* Economical, portable, rugged, easy to use and maintain. Devices have self-contained power sources. Negligible vibration. Unaffected by extreme temperatures.	Requires boreholes at close intervals to control crack propagation. Control of crack plane depth is limited. Not applicable to vertical surfaces. Produces some noise. Drill holes must hold water.
2.2.10 *Presplitting* (continued)	*Expansive agents* Applicable where 9 in. (230 mm) or more of a concrete face is to be removed. Can be used to produce vertical splitting planes of significant depth. No vibration, noise, or flying debris is produced other than that produced by the drilling of boreholes and secondary breakage method.	Best used in gravity-filled vertical or near-vertical holes. Agents of putty consistency are available for use in horizontal or overhead holes. Development of presplitting plane is significantly decreased by presence of reinforcing steel normal to presplitting plane.
2.2.11 *Abrasive blasting* Uses equipment that propels an abrasive medium at high velocity at the concrete to abrade the surface.	*Sandblasting* Efficient method for roughening the surface and exposing aggregate. Cleans reinforcing steel. Removes surface contamination.	Dry sandblasting procedure produces large volumes of dust. Wet sandblasting is slow and is difficult to operate within legal emission requirements.
	Shotblasting Efficient method for roughening the surface and exposing aggregate. Low dust emissions. Removes surface contaminants. Controlled depth of concrete removal. Readily available commercially.	Large units may produce high noise levels. High voltage power requirements.
2.2.11 *Abrasive blasting* (continued)	*High-pressure water blasting (with abrasives)* Selectively removes defective concrete. Removes large quantities of concrete efficiently. Precise control of removal process Cleans reinforcing steel while removing concrete. Produces minimal damage to remaining concrete. Produces no heat or dust. Abrasives enable jet to cut steel reinforcement and hard aggregates.	High initial investment. Additional protection and safety procedures are required due to high water pressure. Controlling flow of contaminated waste water may be required.

breakers if breakage and secondary damage are to be minimized and to avoid breaking through of floors and decks. Determination of the tensile rupture strength by pull-off testing is recommended on surfaces prepared by hand-held breakers to determine the suitability of the surface to receive repair materials.

b) *Boom-mounted breakers*—The boom-mounted breaker is similar to the hand-held breaker except that it is mechanically operated and considerably larger. Equipment-mounted breakers, however, differ from hand-held pneumatic tools in that they work on the principle of high energy and low frequency rather than low energy and high frequency. The mechanical tool is attached to compressed air or hydraulic pressure. The reach of the hydraulic arm enables the tool to be used on walls or overhead at a considerable distance above and below the level of the machine. The boom-mounted breaker is a highly productive means of removing concrete. The high-cycle impact energy delivered to a structure by the breaker, however, generates vibrations that may damage the remaining concrete and reinforcing steel and adversely affect the integrity of the structure.

c) *Scabblers*—Scabblers are best known for their ability to remove shallow depths of concrete from a surface. The scabbler heads are of various sizes and geometrical shapes, and varying numbers may be mounted on the cylinders. The size of the equipment depends on the number of cylinders. The equipment is normally operated by compressed air. The scabbler is designed to remove deteriorated or sound concrete from a surface to a predetermined depth. Scabblers usually leave a surface with microcracking. Determination of the tensile rupture strength by pull-off testing is recommended on surfaces prepared by scabblers to determine the suitability of the surface to receive repair materials.

2.2.8 *Milling methods*—Milling methods remove a specified amount of concrete from large areas of horizontal or vertical surfaces. The removal depth can vary from 3 to 100 mm

(0.1 to 4 in.). Milling operations usually leave a sound surface with less microfractures than impact methods (Virginia Transportation Research Council 2001).

2.2.9 *Scarifier*—A scarifier is a concrete cutting tool that employs the rotary action and mass of its cutter bits to rout cuts into concrete surfaces. It removes loose concrete fragments (scale) from freshly blasted surfaces and removes concrete that is cracked and weakened by an expansive agent. It also is the sole method of removing deteriorated and sound concrete in which some of the concrete contains form ties and wire mesh. Scarifiers are available in a range of sizes. The scarifier is an effective tool for removing deteriorated concrete on vertical and horizontal surfaces. Other advantages include well-defined limits of concrete removal, relatively small and easily handled concrete debris, and simplicity of operation.

2.2.10 *Hydrodemolition*—High-pressure water jetting (hydrodemolition) can be used to remove concrete to preserve and clean the steel reinforcement for reuse and to minimize microcracking to the remaining in-place concrete. The method also has a high efficiency. Hydrodemolition disintegrates concrete, returning it to sand and gravel-sized pieces. This process works on sound or deteriorated concrete and leaves a rough profile. Hydrodemolition punches through the full depth of slabs in small areas when the concrete is unsound or when full-depth patches are inadequately bonded to the side walls. Hydrodemolition should not be used in structures with unbonded tendons, except under the direct supervision of a structural engineer.

High-pressure water jets in the 70 MPa (10,000 psi) range require 130 to 150 L/min (35 to 40 gal./min). As the pressure increases to 100 to 140 MPa (15,000 to 20,000 psi), the water demand varies from 75 to 150 L/min (20 to 40 gal./min) (Nittenger 1997). The equipment manufacturer should be consulted to confirm the water demand. Ultra-high-pressure equipment operating at 170 to 240 MPa (25,000 to 35,000 psi) has the capability of milling concrete to depths of 3 to 150 mm (0.1 to 6 in.). Containment and subsequent disposal of the water are requirements of the hydrodemolition process. Many localities require this water to be filtered and then treated to reduce the alkalinity and particulates before the water can be released into a storm or wastewater system.

Water jet lances operating at pressures of 70 to 140 MPa (10,000 to 20,000 psi) and having a water demand of 75 to 150 L/min (20 to 40 gal./min) are available. They are capable of cutting sections of concrete or selectively removing surface concrete in areas that are difficult to reach with larger equipment (ICRI 03737).

2.2.11 *Presplitting methods*—Presplitting methods use hydraulic splitters, water pressure pulses, or expansive chemicals used in boreholes drilled at points along a predetermined line to induce a crack plane for the removal of concrete. The pattern, spacing, and depth of the boreholes affect the direction and extent of the crack planes that propagate. Presplitting is generally used in mass concrete structures or unreinforced concrete.

a) *Hydraulic splitter*—The hydraulic splitter is a wedging device that is used in predrilled boreholes to split concrete into sections. This method has potential as a primary means for removal of large volumes of material from mass concrete structures. Secondary means of separating and handling the concrete, however, may be required where reinforcing steel is involved.

b) *Water-pulse splitter*—The water-pressure pulse method requires that the boreholes be filled with water. A device, or devices, containing a very small explosive charge is placed into one or more holes, and the explosive is detonated. The explosion creates a high-pressure pulse that is transmitted through the water to the structure, cracking the concrete. Secondary means may be required to complete the removal of reinforced concrete. This method does not work if the concrete is so badly cracked or deteriorated that it does not hold water in the drill holes.

c) *Expansive product agents*—Commercially available cementitious expansive product agents, such as those containing aluminum powder, when correctly mixed with water, exhibit a large increase in volume over a short period of time. By placing the expansive agent in boreholes located in a predetermined pattern within a concrete structure, the concrete can be split in a controlled manner for removal. This technique has potential as a primary means of removing large volumes of material from concrete structures and is best suited for use in holes of significant depth. Secondary means may be required to complete the separation and removal of concrete from the reinforcement. A key advantage to the use of expansive agents is the relatively nonviolent nature of the process and the reduced tendency to disturb the adjacent concrete.

2.2.12 *Abrading blasting*—Abrading blasting removes concrete by propelling an abrasive medium at high velocity against the concrete surface to abrade it. Abrasive blasting is typically used to remove surface contaminants and as a final surface preparation. Commonly used methods include sand-blasting, shotblasting, and high-pressure water blasting.

2.2.13 *Sandblasting*—Sandblasting is the most commonly used method of cleaning concrete and reinforcing steel in the construction industry. The process uses common sands, silica sands, or metallic sands as the primary abrading tool. The process may be executed in one of three methods.

2.2.14 *Dry sandblasting*—Sands are projected at the concrete or steel in a stream of high-pressure air in the open atmosphere. The sand particles are usually angular and may range in size from passing a 212 to a 4.75 mm (No. 70 to a No. 4) sieve. The rougher the required surface condition, the larger the sand particle size.

The sand particles are propelled at the surface in a stream of compressed air at a minimum pressure of 860 kPa (125 psi). The compressor size varies, depending on the size of the sand-blasting pot. Finer sands are used for removing contaminants and laitance from the concrete and loose scale from reinforcing steel. Coarser sands are commonly used to expose fine and coarse aggregates in the concrete by removing the paste or tightly bonded corrosion products from reinforcing steel. Although sandblasting has the ability to cut quite deeply into concrete, it is not economically practical to remove more than 6 mm (0.25 in.) from the concrete surface.

2.2.15 *Wet sandblasting*—The free particulate rebound that results from the sand being projected at the concrete surface is confined within a circle of water. Although this process significantly reduces the amount of airborne particulates, some of the water intercepts the sand being projected at the concrete surface and reduces the efficiency of the sandblasting operation. This process is generally limited to cleaning a concrete surface or reinforcing steel.

2.2.16 *High-pressure wet sandblasting*—Sand is projected at the concrete surface or the reinforcing steel in a stream of water at pressures ranging from 10 to 20 MPa (1500 to 3000 psi). The water significantly reduces the efficiency of the sandblasting operation. Although this process eliminates any free airborne dust, it only can be used for cleaning concrete surfaces. The water may require treatment before being released into a storm sewer system.

2.2.17 *Shotblasting*—Shotblasting equipment cleans or removes concrete by projecting metal shot at the concrete surface at a high velocity. This equipment can remove finite amounts of sound or unsound concrete. The shot erodes the concrete from the surface. The shot rebounds with the pulverized concrete and is vacuumed into the body of the shotblasting machine. The concrete particulates are separated out and deposited into a holding container to be discarded later while the shot is reused. The shotblasting process is a self-contained operation that is highly efficient and environmentally sound.

Shotblasting uses shot of varying sizes. The final surface condition required determines the size of the shot and the speed at which the machine will travel. A surface cleaning operation is achieved by using a small-sized shot and setting the machine for maximum travel speed. Removal of as much as 6 mm (0.25 in.) in a single pass and leaving a surface with an amplitude of 3 mm (0.12 in.) can be achieved by increasing the size of the shot and by traveling at a low speed.

Shotblasting equipment is an effective and economical method for removing up to 20 mm (0.75 in.) of concrete. Shotblasting has removed up to 40 mm (1.5 in.) of concrete. The cost per unit of volume, however, increases significantly as depth of removal increases beyond 20 mm (0.75 in.).

2.2.18 *High-pressure water blasting (with abrasives)*—High-pressure water blasting with abrasives is a cleaning system using a stream of water at high pressure of 10 to 35 MPa (1500 to 5000 psi) with an abrasive, such as sand, aluminum oxide, or garnet, introduced into the stream. This equipment can remove dirt or other foreign particles and concrete laitance, thereby exposing the fine aggregate.

Abrasive water blasting eliminates the airborne particles that occur when using normal sandblasting procedures. The water used is collected and the abrasive removed before the water is discharged into a storm or waste water system.

Abrasive water blasting leaves the concrete surface clean and free of dust. The surface is prepared to receive the next operation such as sealer, coating, or overlay.

2.3—Surface preparation

One of the most important steps in the repair of a concrete structure is the surface preparation of the repair area. This involves removal of deteriorated or designated sound concrete, or both. Regardless of the nature, sophistication, or expense of the repair material, the repair is only as good as the surface preparation. To ensure the desired behavior in the structure, repairs to reinforced concrete should also include proper preparation of the reinforcing steel to develop a bond with the replacement concrete. This section examines the preparation of concrete and reinforcing steel as may be required. Concrete removal procedures, as presented in Section 2.2, should be followed.

2.3.1 *General conditions*—Surface preparation consists of the final steps necessary to prepare the concrete surface to receive the repair materials. The appropriate preparation of the concrete surface depends on preceding operations to remove concrete and on the type of repair undertaken.

Most of the methods described in Section 2.2 also can be used for surface preparation. An effective method for concrete removal, however, may not be effective or appropriate for a specific surface preparation. For example, some concrete removal methods leave the concrete surface too smooth, too rough, or too irregular for the subsequent repair. In these cases, removal methods or methods specifically intended for the final surface preparation may be needed. Some concrete removal methods damage or weaken the remaining concrete surface. This may be critical if structural bonding of a subsequent repair is important. For example, microcracking of the resultant surface can occur when impact methods are used, providing a weakened plane in the parent concrete below the repair material bondline. In this case, a less invasive and less aggressive method of final surface preparation, such as sand or water abrasion, may be better.

In many repair situations, the proposed repair requires only surface roughening, exposure of coarse or fine aggregate, removal of a thin layer of damaged concrete, or cleaning of the concrete surface. Within the limits described in the previous paragraph, several of the removal methods described in Section 2.2 can be used for this type of surface preparation. The referenced methods offer a wide range of possible surface characteristics and results. For example, the finished surface may vary from a light abrasive cleaning suitable for the application of a coating, to a deeper roughening needed for strong bond and reliable performance of a critical structural repair. The choice of suitable methods is extremely important because it has a strong influence on both the cost and the performance of the repair.

2.3.2 *Methods of surface preparation*—Typical methods of surface preparation are:

a) *Chemical cleaning*—In most cases, chemical cleaning methods of surface preparation are not appropriate for use with the concrete repair materials and methods presented in this guide. With certain coatings under certain conditions, however, it may be possible to use detergents, trisodium phosphate, and various proprietary concrete cleaners. All traces of the cleaning agent should be removed after the contaminating material is removed. Solvents should not be used to clean concrete because they dissolve the contaminant and carry it deeper into the concrete.

b) *Acid etching*—Acid etching of concrete surfaces has long been used to remove laitance. The acid removes enough cement paste to provide a roughened surface that improves the bonding surface for coatings. ACI 515.1R recommends that acid be used only when no alternative means of surface preparation can be used, and ACI 503R does not recommend the use of acid etching. Acids may penetrate the concrete surface through cracks and promote corrosion of underlying reinforcing steel in structural concrete. The acids weaken the remaining paste phase on the concrete surface.

c) *Mechanical preparation*—This technique consists of mechanically removing thin layers of surface concrete using equipment such as impacting tools (breakers and scabblers), grinders, and scarifiers. Depending on the equipment used, a variety of surfaces may be obtained. These methods should be used carefully or they could result in excessive microcracking, as discussed in Section 2.2.7.

d) *Abrasive preparation*—This technique removes thin layers of surface concrete using abrasive equipment such as sandblasters, shotblasters, or high-pressure water blasters.

Upon completion of the surface preparation, all residue left by the process should be removed. This may require further water or air blasting, vacuuming, or other techniques. ICRI 03732 provides a profile number system for determining surface roughness necessary for various overlay applications. Surface roughness defined by sandpaper grit size is also commonly used to characterize surface roughness.

2.4—Reinforcement repair

The most frequent cause of damage to reinforcing steel is corrosion. Other possible causes of damage are fire, chemical attack, and accidental cutting. The following basic preparation and repair procedures may be used for all of these causes of damage.

After the cause of the damage has been determined, it is necessary to expose the steel, evaluate its condition, and prepare the reinforcement for the repair techniques. Proper steps to prepare the reinforcement helps ensure that the repair method meets the requirements for the longevity of the repair solution.

An inexpensive (on a short-term basis) and common approach to repair of reinforcement corrosion is to replace concrete only where spalls or delaminations have occurred. Generally, this approach leaves chloride-contaminated concrete surrounding the repaired area that is highly conducive to continued corrosion. The repairs may actually aggravate corrosion in the area adjacent to them. This is known as the halo or anodic effect and is discussed more extensively in Section 4.3.5.

2.4.1 *Removal of concrete surrounding steel*—The first step in preparing reinforcing or prestressing steel for repair or cleaning is removing the deteriorated concrete surrounding the reinforcement. Care should be used to ensure that further damage to the reinforcing or prestressing steel is not caused by the process of removing the concrete. Impact breakers can heavily damage reinforcing or prestressing steel if the breaker is used without regard to the location of the reinforcement. For this reason, a covermeter or reinforcing bar locator, along with a copy of the structural drawings (if available), should be used to determine the depth, size, quantity, and approximate location of the reinforcement in the concrete.

Once the larger areas of unsound concrete have been removed, a smaller chipping hammer should be used to remove the concrete in the vicinity of the reinforcement. Care should be taken not to vibrate the reinforcement or otherwise cause damage to its bond to concrete adjacent to the repair area. Drawings and specifications should provide guidance on development length and lap splices. Specific language should be provided in the documents that every precaution be taken to avoid damaging or cutting existing reinforcement during the concrete removal process (Section 2.2.2). In addition, no reinforcing bar is to be cut or removed without the approval of the engineer of record. Additionally, the engineer should be aware that, in many instances, the reinforcement is not stressed to the same level after the completion of the repair, unless the structural component being repaired has the live and dead loads removed before and during the repair process. In all cases, shoring requirements should be reviewed before removing concrete or cutting reinforcement. The contractor is usually responsible for shoring. The contract documents should provide adequate information to the contractor, including warning, when appropriate, where the contractor should properly provide the required bracing and shoring.

a) *Quantity to remove*—All weak, damaged, and easily removable concrete should be chipped away. If the reinforcing bars are only partially exposed after all unsound concrete is removed, it may not be necessary to remove additional concrete to expose the full circumference of the reinforcement. When the exposed reinforcing steel has loose rust, corrosion products, or is not well bonded to the surrounding concrete, the concrete removal should continue to create a clear space behind the reinforcing steel of 6 mm (0.25 in.), plus the dimension of the maximum size aggregate of the repair material (ICRI 03730).

b) *Inspection of reinforcing steel*—After all deteriorated and some sound concrete have been removed, reinforcing steel should be cleaned and carefully inspected. The inspection should determine whether the reinforcing steel is capable of performing as intended by the designer. Damaged reinforcement may have to be replaced or supplemented, and the responsible engineer should be consulted. Project specifications should include criteria whereby decisions concerning repair or replacement can be made during the project as reinforcement is exposed, such as lap tables to establish splice lengths, alternate mechanical splice parameters, or both, on the project drawings.

c) *Cleaning reinforcing steel*—All exposed surfaces of the reinforcement should be thoroughly cleaned of all loose mortar, rust, oil, and other contaminants. The degree of cleaning required depends on the repair procedure and material selected. For limited areas, wire brushing or other hand methods of cleaning may be acceptable. Generally, sandblasting is the preferred method. When cleaning the steel and blowing loose particles out of the repair area after cleaning, neither the reinforcing steel nor the concrete substrate should be contaminated with oil from the air compressor. For this reason, either an oil-free compressor or one that has a good oil trap is recommended.

Freshly cleaned reinforcing steel may rust between the time it is cleaned and the time concrete is placed. If the rust that forms is tightly bonded to the steel such that it cannot be removed by wire brushing, no action is required. If the rust is loosely bonded so as to inhibit bond between the steel and the concrete, the reinforcing bars should be cleaned again before repair material is placed. A protective coating may be applied to the reinforcement after the initial cleaning has been completed.

2.4.2 *Repair of reinforcement*—Mild reinforcing steel and prestressing steel are used in concrete structures, and two different repair procedures are necessary. Depending on the condition of the exposed reinforcement, a decision for a repair alternative can be made.

2.4.2.1 *Mild reinforcing steel*—For reinforcing steel, one or two repair alternatives may be necessary: replacement of deteriorated bars or supplementing partially deteriorated bars. Which alternative to use is strictly an engineering decision based on the purpose of the reinforcement and the required structural capacity for the reinforced member.

a) *Replacement*—One method of replacing reinforcement is to cut out the damaged area and splice in replacement bars. The length of the lap should conform to the requirements of ACI 318. If welded splices are used, welding should be performed in accordance with ACI 318 and American Welding Society (AWS) D1.4. All welding and cutting should be performed by a welder AWS certified to the requirements of AWS D1.4. Butt welding should be avoided due to the high degree of skill required to perform a full penetration weld because the back side of a bar is not usually accessible. Welded splices for bars larger than 25 mm (No. 8) might present problems because the embedded bars may get hot enough to expand and crack the surrounding concrete. Special precautions are necessary when welding adjacent to unbonded or bonded prestressing steel. Reinforcing steel splicing can be used to mechanically butt splice the ends of reinforcement.

Another method of splicing bars is to use mechanical connections. ACI 439.3R describes commercially available proprietary mechanical connection devices. Mechanical connections should meet the requirements of ACI 318.

b) *Supplemental reinforcement*—This alternative is selected when the reinforcement has lost cross section, the original reinforcement was inadequate, or the existing member needs to be strengthened. The allowable loss of cross-sectional area of the existing reinforcing steel and the decision to add supplemental reinforcement should be evaluated on a case-by-case basis and is the responsibility of the engineer. The damaged reinforcing bar should be cleaned in accordance with the guidance in Section 2.4.1.c. The concrete should be chipped away to allow placement of the supplemental bar beside the old bar. The length of the supplemental bar should be equal to the length of the deteriorated segment of the existing bar plus a lap-splice length on each end equal to the lap-splice requirements for the smaller bar diameter, as specified in ACI 318. The supplemental bar may also be lap welded to the original bar in accordance with AWSD 1.4. When supplemental reinforcement is placed to strengthen a section, techniques discussed in Chapter 5 should be considered. Supplemental reinforcement may be used in some situations to provide additional anchorage of the concrete repair materials by providing mechanical anchorage if a bond failure should occur. This mechanical anchorage is important on repairs to building façades or overhead repairs.

c) *Coating of reinforcement*—New and existing bars that have been cleaned may be coated with epoxy, polymer-cement slurry, or a zinc-rich coating for protection against corrosion. The coating should be applied at a thickness less than 0.3 mm (12 mils) to minimize loss of bond development at the deformations. Reinforcing bars that have lost their original deformations as a result of corrosion and cleaning have less bond with most repair materials. Coating of these bars further reduces the bond with repair materials. Care should be taken during the coating process to avoid spillage on the parent concrete. Some materials, such as epoxy or zinc-rich coatings, could act as a bond breaker between the new repair material and the original concrete. Other coating materials, such as polymer-cement coatings, may contain corrosion inhibitors for the reinforcing steel and bonding agents for the new to old concrete.

2.4.3.2 *Prestressing steel*—Prestressing steel in structural members is of two basic types: bonded and unbonded. Deterioration or damage to the strands or bars can result from impact, design error, overload, corrosion, or fire. Fire may anneal cold-worked, high-strength prestressing steel.

Flexibility in repair of either type is limited. Unlike mild steel reinforcing bars, the unbonded high-strength strands may need to be detensioned before repair and retensioned after repair to restore the initial structural integrity of the member. Repair options for bonded strands are different from those for the unbonded strands.

a) *Bonded strands*—Because the prestressed strand is bonded, only the exposed and damaged section is restressed following repairs. The repair procedure requires replacing the damaged section with the new section of strand connected to the existing ends of the undamaged strands. The new strand section and the exposed lengths of the existing strand should be post-tensioned to match the stress level of the bonded strand. For other types of repair, refer to Chapter 5.

b) *Unbonded tendons*—Unbonded strands are installed inside sheathings (conduits) embedded in the concrete member. The strands are protected against corrosion by the sheathing, corrosion-inhibiting material (commonly grease), or both. Because the strands are not stressed until after the concrete has been placed, an annular space is created within the sheathing following the stressing operation. Corrosion of the end connections and the strand have been the primary cause of failure of unbonded tendons. Strand failure due to corrosion at the end anchorages and in the strands is a problem that increases with time.

Unbonded tendons can be tested to verify their ability to carry the design load. This can be done by attaching a chuck and coupler to the exposed end of the strand and performing a lift-off test. This usually requires at least 20 mm (0.75 in.) of free strand beyond the bulkhead. If there is excessive corrosion in the strand, failure occurs and the strand should be replaced or spliced. Shoring of the span being repaired and adjacent spans up to several bays away may be required before removing or retensioning unbonded prestressed strands. ICRI 03736 provides some guidance for evaluation of unbonded post-tensioned concrete structures.

A deteriorated portion of a strand can be exposed by excavating the concrete and cutting the sheathing. The strand is then cut on both sides of the deterioration where sound strand is evident. Caution should be exercised when cutting tensioned strands. The removed portion of the strand is replaced with a new section that is spliced to the existing strand at the location of the cuts. The repaired strand is then restressed.

Removal of an unbonded strand from the sheathing is possible, but is sometimes difficult. It is also difficult to install a replacement strand of the same diameter. When the intention is to replace a strand, it may be preferable to insert a smaller-diameter strand of higher-strength material that would be capable of providing a stressing force comparable to that in the original strand.

Carbon fiber or equivalent systems are available to supplement the reinforcement in prestressed, post-tensioned, and mild steel reinforced structures. This system is normally glued onto the exterior surface. Unless the component being reinforced is unloaded, however, it only provides reinforcement for future loadings. Fiber wrapping is commonly used for reinforcing columns, especially in earthquake zones.

Protective materials commonly dry and leave the unbonded strand unprotected and vulnerable to corrosion. There are systems available that reestablish the protective barrier within the sheathing. One system fills the void with a two-component urethane and the other system fills the void with a grease. These procedures provide extended protection against corrosion of the strand.

2.5—Anchorage methods and materials

Anchors are often used in conjunction with supplemental reinforcement to prevent new repairs from dislodging from parent concrete in the event of a bond failure. They are commonly used on the repairs of vertical surfaces, such as façades, to ensure that the façade repair does not dislodge from the structure. Noncorrosive anchors, such as stainless steel, are commonly specified for overhead repairs that might cause injury in the event of failure of the repair façades and for other corrosive conditions where inadequate cover is provided.

There are two general categories of anchoring systems: post-installed and cast-in-place (ACI 355.1R).

2.5.1 *Post-installed anchors*—Post-installed anchor systems are installed into a predrilled hole. They can be divided into two general types: bonded and expansion anchors.

Bonded anchors include both grouted (headed bolts or a variety of other shapes installed with a cementitious grout) and chemical anchors (usually threaded rods set with a two-part chemical compound that is available as glass capsules, plastic cartridges, tubes, or bulk). These anchor systems develop their holding capacities by the bonding of the adhesive to both the anchor and the concrete at the wall of the drilled hole. The different chemical systems (epoxies, polyesters, and vinyl esters) have different setting and performance characteristics that should be understood by the specifier and user. The engineer should exercise precautions when specifying chemical anchors in either an underwater or moist environment by verifying that the chemical adhesive has been tested in a similar environment.

Expansion anchor systems, sometimes called mechanical anchors, include torque-controlled, deformation-controlled, and undercut anchors. These anchors develop their strength from friction against the side of the drilled hole, from keying into a localized crushed zone of the concrete resulting from the setting operation, or from a combination of friction and keying. These types of anchors develop bursting forces in the concrete. Edge distances and the spacing between anchors should be sufficient to prevent cracking. For the undercut anchors, strength is derived from keying into an undercut at the bottom of the drilled hole.

2.5.2 *Cast-in-place anchors*—Cast-in-place anchor systems include embedded nonadjustable anchors of various types and shapes, bolted connections, and adjustable anchors that are set in place before placing concrete.

2.5.3 *Anchor strength*—Anchor strength and long-term performance are dependent on a variety of factors that should be evaluated for the anchor to be used. Some factors to be considered include material strength (yield and ultimate), hole diameter and drilling system used, embedment length, annular gap between the anchor and the drilled hole for post-installed anchors, concrete strength and condition, type and direction of load application (static, dynamic, tension, shear, bending, or combined loading), spacing to other anchors and edges, temperature (for chemical anchors), hole cleaning, mode of failure of the anchor system (concrete breakage, steel breakage, slip, or pullout), environmental conditions for moisture and corrosion resistance, and creep.

Site testing for verification of performance is recommended for critical applications. For chemical anchors, tests should be performed to determine the long-term creep performance at the highest expected service load and temperature. For all anchor systems, installation instructions should be followed to ensure proper anchor performance.

2.6—Materials placement for various repair techniques

Many techniques are available to place the repair materials and depend on the constraints and limitations of a project (Emmons 1994). In addition, the past experience of the contractor to successfully perform the placement technique is also important. ICRI 03731 provides guidance for selecting application techniques for repair.

2.6.1 *Cast-in-place concrete, modified concrete, and proprietary concrete or mortars*—Repair by conventional concrete placement is the replacement of defective concrete with new concrete that is conventionally placed. This method is the most frequently used repair technique, and it is usually the most economical.

Repair by conventional concrete placement is applicable to a wide range of situations, including repair of deterioration due to defects caused by poor construction practices. Replacement with conventionally placed concrete should not be used where aggressive exposure conditions caused deterioration of the concrete, unless a protection system can mitigate the factors that triggered the deterioration. For example, if the deterioration was caused by acid attack, aggressive water attack, or even abrasion-erosion, a repair made with conventional concrete may deteriorate again for the same reasons. Portland-cement concrete (PCC) modified with silica fume, acrylics, styrene-butadiene latex, or epoxy, however, have been successful in extending service life.

2.6.2 *Shotcrete*—Shotcrete is concrete or mortar that is pneumatically conveyed at high velocity through a hose onto a surface. The high velocity of the material striking the surface provides the compactive effort necessary to consolidate the material and develop a bond to the substrate surface. The shotcrete process is capable of placing repair materials in vertical and overhead applications without the use of forms, and it can routinely place material several hundred feet from the point of delivery.

There are two basic shotcrete processes. In wet-mix shotcrete, cement, aggregate, and water are mixed and pumped through a hose to a nozzle where air is added to propel the material onto the surface. Dry-mix shotcreting uses cement and aggregate that are premixed and pneumatically pumped through a hose, then water is added at the nozzle as the material is projected at high velocity onto a surface.

Either method places suitable repair materials for normal construction requirements. ACI 506R provides detailed information on the two shotcrete processes and their proper application.

In addition to placing conventional PCC and mortar, the shotcrete process is also used for placing polymer-cement concrete, fiber-reinforced concrete using both steel and synthetic fibers, and concrete containing silica fume and other pozzolans.

The application of repair materials by the shotcrete process should be considered wherever access to the site is difficult, the elimination of formwork provides economy, and significant areas of overhead or vertical repairs exist. Shotcrete is frequently used for repairing deteriorated concrete or masonry on bridge substructures, piers, sewers, dams, and building structures. It is also used for reinforcing structures by encasing additional reinforcing steel added to beams, placing bonded structural linings on masonry walls, and placing additional concrete cover on existing concrete structures (refer to Chapter 5).

The shotcrete nozzle operator's skill determines the in-place quality of the repair material. ACI 506.3R provides a basis for determining the qualifications of a nozzle operator. The nozzle operator should be certified by ACI. ACI 506.2 provides a basic specification for the application and inspection of shotcrete.

2.6.3 *Preplaced-aggregate concrete*—Preplaced-aggregate concrete is produced by filling the repair area with gap-graded coarse aggregate, then filling the voids in the aggregate by pumping in a cementitious or resinous grout. As the grout is pumped into the forms, it fills the voids (displacing any water that is present), and forms a concrete mass. The worker should avoid the entrapment of air that result in voids. This method is used for partial-depth repairs or for replacement of whole members. This method reduces drying shrinkage because the aggregate particles are in point-to-point contact before and after grouting.

Generally, the same requirements for materials and procedures that apply to preplaced-aggregate concrete in new construction also apply to repair. Preplaced-aggregate concrete is covered in detail in ACI 304R and ACI 304.1R.

2.6.4 *Formed and pumped concrete and mortar*—Formed and pumped repair is a method of replacing damaged deteriorated concrete by filling a formed cavity with a repair mortar or concrete under pump pressure. This method can be used for vertical and overhead repairs. Formwork should be constructed to a strength sufficient to handle the pressure induced by hydrostatic pressure and the additional pump pressure required to consolidate the repair material. The cavity and formwork design should provide for venting the air. Venting can be accomplished by the removal profile of the prepared concrete, vent tubes, or drilled holes in the existing concrete. Pumping the cavity is started at the lowest point in vertical repairs or at an extremity in overhead repairs. Pumping continues until the material flows from an adjacent port in the formwork. Pumping continues until the cavity is completely full (Emmons 1994). During the final pressurization, the repair material is consolidated around the reinforcing steel and driven into the crevices of the prepared substrate to improve bond.

2.6.5 *Troweling and dry packing*—

a) *Troweling*—Repairs applied by hand-troweling can be used for shallow or limited areas of repair. These repairs can be made using: portland-cement mortars; proprietary products such as cementitious prepackaged materials; polymer-cement grouts; polymer grouts; and mortars. Trowel-applied systems are not recommended when reinforcing steel is exposed and undercut due to the difficulty of consolidation of repair material around and behind the reinforcing steel.

The paste of the repair material should be used as the bonding medium. The repair material should be applied to the grouted surface before the grout or paste sets. Where multiple layers are needed to build up the total thickness of the repair, the surfaces should be roughened to help bond subsequent layers. For most applications, the surface of the cavity should be saturated and surface dry at the time of application of the material.

There are a wide variety of proprietary repair mortars and concretes that have been modified with chemicals and thixotropic agents. The placement techniques recommended by the manufacturers do not always conform to accepted placement techniques for portland-cement mortar and concrete. This is particularly true for thin sections, such as 3 mm (0.1 in.), and vertical and overhead applications. The addition of chemicals and thixotropic agents that permit deviations in placement techniques may compromise some of the performance properties. Examples of some of the properties that may be affected are shrinkage, bond strength, and coefficient of thermal expansion. The specifier and contractor should consult with the manufacturer and make sure that the materials performance capability and limitations meets the project criteria before using these materials. Refer to Chapter 3.

Successful use of trowel-applied repairs is dependent on the surface preparation and the skill of the mason. Masons should be experienced, and close field observation of the work should be made. Proper troweling technique should be used to prevent the entrapment of air at the bonding surface, which can cause reduced bond strength. Proper curing of portland-cement mortar so that patch material does not dry before hydration is complete is important. Special curing provisions may be advisable for some proprietary repair materials or where accessibility is difficult (ACI 308R and 308.1).

b) *Dry packing*—Dry packing is the hand placement of a low-water portland-cement mortar and the subsequent tamping or ramming of the mortar into place. Because of the low water-cementitious material ratio (*w/cm*), these repairs, when compacted properly, have good strength, durability, and water tightness.

Dry packing can be used for repair of form-ties, cone-bolts, and other holes and small areas with relatively high depth-to-surface-areas ratios. Because of the labor-intensive nature of this technique, it is not often used for large repairs.

2.6.6 *Injection grouting*—Grouting is the common method for filling cracks, open joints, honeycomb, and interior voids with a cement grout or other material that cures in place to produce a desired result. Materials other than cement grout include polymer-cement slurry, epoxy, urethane, and high-molecular-weight methacrylate (HMWM). Grouting can strengthen a structure, arrest water movement, or both. Before designing a grouting repair program, the objectives of grouting should be defined and the proper material selected to meet those objectives. Where appropriate, quality control measures should include taking cores to verify that proper penetration and bond has been achieved.

2.6.6.1 *Cement grouting*—Cement grout is a mixture of cementitious material, normally portland cement or microfine cement, and water, with or without fine aggregate or admixtures, proportioned to produce a pumpable consistency without excessive segregation of constituents. Grout can be injected into an opening from the surface of a structure or through holes drilled to intersect the opening in the interior.

a) *Grouting from the surface*—When grout is to be injected from the surface, short entry holes (ports), a minimum of 25 mm (1 in.) in diameter and a minimum of 50 mm (2 in.) deep, are drilled into the opening. The surface of the opening is sealed between ports with a portland cement or resinous mortar. Whether or not short pipe nipples are cemented into the holes for grout hose connections depends largely on anticipated grouting pressures. If the ports are drilled after sealing the openings, a hand-held, cone-shaped fitting on the grout hose may be adequate for pressure under 350 kPa (50 psi). Where cracks or openings extend through a structure, such as a wall, the opening is usually sealed and ported on the far side as well.

Where appearance is not a factor, openings may often be sealed by caulking with cloth or fabric that pass water or air yet retain solids. Paper and materials that remain plastic are not suitable for this purpose.

The spacing of the entry ports is largely a matter of judgment based on the nature of the work. As a general rule, however, ports should be farther apart than the desired depth of grout penetration.

Before grouting, the openings should be flushed with clean water, subject to the procedure that is used in grouting. Flushing is done for several reasons: to wet the interior surfaces for better grout flow and penetration; to check the effectiveness of the surface sealing and port system; to provide information on probable grout flow patterns and internal interconnections, some possibly unexpected; and to familiarize the grouting crew with the situation.

Grouting is started at one end of a horizontal opening or at the bottom of a vertical opening and continues until grout shows at the second port away from the one being pumped. When this occurs, the grouting operation moves to the next port and continues until grout again shows at the second distant hole. The valve should be shut or plugged at each flowing port before moving to the next injection location. Progress should be monitored on the far side of the structure, if accessible, and close ports or valves as necessary.

Grouting is usually started with a relatively thin grout, thickened as quickly as possible to the heaviest consistency that can be pumped without blockage, as determined at the grouting operation.

b) *Interior grouting*—Grouting of cracks, joints, and voids from the interior is done through 25 mm (1 in.) or larger diameter holes drilled at an angle to intersect the opening at desired depth from the surface or, as near as possible, to the bottom of the void.

Drilling is done with diamond core bits, rotary carbide bits, or percussion drills. Diamond or rotary bit drilling is preferred, especially when the openings to be grouted are relatively narrow, to minimize the debris that would choke the crack. Applying a vacuum to the drill stem further reduces the possibility of drill cuttings getting into the crack. For wider openings—for example, 12 mm (0.5 in.) or more—drill cuttings are less of a problem, but in any event, all holes should be thoroughly washed and water circulated through the system before grouting.

2.6.6.2 *Chemical grouting*—Chemical grout, as defined in this guide, is any fluid material not dependent on suspended solids for reaction. The grout should harden without adversely affecting any metals or the concrete boundaries of the opening or void into which it has been injected. From the standpoint of the user, chemical grouts are usually two-component systems requiring blending at or near the point of injection, or the blending of the injected chemical with moisture or water existing in the crack or placed there by the grouter. Chemical grouts may contain various inert fillers to modify physical properties, such as consistency and heat generation, and to increase volume. Chapter 3 describes materials for chemical grouting.

Chemical grout should be injected from the surface or the interior in the same general manner as cement grouts with the exception that, for unfilled grouts, the port sizes may be 3 or 6 mm (0.1 or 0.2 in.) in diameter, and the port devices may be mechanically anchored or cemented into place (ACI 224.1R, 503.4, and 503.6R).

2.6.6.3 *Selection of type of grout*—Questions that may be considered when the type of grout is to be selected for a given repair include:

1. Is it necessary to transmit compression, tension, shear, or a combination across the crack?
2. Is the crack active (moving) and subject to future tensile forces that may exceed the tensile or shear strength of the concrete in the vicinity of the repaired crack, or exceed the elongation and creep characteristics of the grout?
3. Is preventing water or air movement through the crack all or part of the requirement?
4. Is the crack width sufficient to accept the type of grout selected?
5. Does the required internal grout pressure exceed the resistance of the structure or of the surface sealer?
6. Is the rate of grout stiffening slow enough to permit the grout to reach its destination and fast enough to minimize leakage from the blind side?
7. Is the exothermal heat liberation of some chemical grouts, especially epoxy types, excessive?
8. Is the grout cost effective in relation to desired or required results?
9. Are the shrinkage, creep, and moisture absorption characteristics of the grout compatible with the project conditions?
10. Is the viscosity low enough and the pot life long enough to ensure full penetration of the crack, particularly small cracks in a large concrete mass?
11. Are the wetability characteristics of the resin capable of ensuring an acceptable bond?
12. Will the epoxy adhesive cure in the presence of moisture without absorbing moisture and compromising the bond?

2.6.6.4 *Parameters of cement and chemical grouts*—

a) *Cement grouts*—Cement and other grouts containing solids in suspension can be used only where the width of the opening is sufficient to accept the solid particles. For the reliable penetration of neat grouts (hydraulic cement mixed with a latex with or without pozzolans and other admixtures) mixed with approximately 83 L of water to 100 kg (10 gal. per 100 lb) of solids (water-to-solids ratio of approximately 0.8), minimum crack width at the point of introduction should be approximately 3 mm (0.1 in.). With flow started in the opening, such grout penetrates through cracks 0.25 mm (0.01 in.) wide. As crack widths increase to 6 mm (0.25 in.) or more, the mixing water may be reduced to 42 to 50 L/100 kg (5 to 6 gal. per 100 lb) of solids (water-to-solids ratio of approximately 0.4 to 0.5), especially when water-reducing admixtures are used. For openings of 12 mm (0.5 in.) or more and for interior voids, grouting sand or masonry sand ranging from one to two times the mass or volume of the cementing material can be included. Fine aggregate meeting the requirements of ASTM C 33 can also be used when filling large voids.

Finely ground specialty cements and silica fume moves into finer openings more readily than normal hydraulic cements, but definitive information on the penetrability of these materials into cracks and joints is limited.

Hydraulic cement grouts are excellent for reintegrating and stabilizing cracked structures, such as bridge piers, tunnel linings, or walls, where reestablishing compression and shear strength is the main goal. Cement grouts also provide some tensile bond, but tensile strength is difficult to predict. Expansive cement grouts are widely used to prevent water movement (ACI 223).

b) *Chemical grout*—Chemical grouts should be considered under two categories according to whether they harden to a rigid condition or to a flexible gel or foam. Epoxies and acrylates are examples of rigid types; polyurethane is an example of a gel.

Rigid chemical grouts bond exceedingly well to dry substrate and some bond to wet concrete. These grouts can prevent all movement at an opening and restore the full strength of a cracked concrete member. If tensile or shear stresses exceeding the capability of the concrete recur after grouting, however, new cracks appear in the concrete near, but generally not at, the grouted crack. Rigid grouts can penetrate cracks somewhat finer than 0.05 mm (0.002 in.), the penetration being dependent on viscosity, injection pressure, temperature, and grout set time.

The principal use for gel-type chemical grouts is to shut off or greatly reduce water movement. Gel grouts do not restore strength to a structure, but they generally maintain water tightness despite minimal movement across a crack. Most gel grouts are water solutions and therefore exhibit shrinkage if allowed to dry, but they do recover when rewetted. Some gel grouts can be formulated at consistencies so near that of water that they can be injected into any opening through which water flows. Others can be made to yield a foam that can be used in openings approximately 100 mm (4 in.) wide.

2.6.7 *Underwater placement*—Placing concrete directly underwater by means of a tremie or pump is a frequently used repair method. In general, the same requirements for material and procedures that apply to new construction also apply to repair placements underwater. Underwater repairs are presented in ACI 546.2R. Placing concrete under water by tremie and by pump is covered in detail in ACI 304R.

Preplaced-aggregate concrete is frequently used on underwater repair projects. The concrete mortar is pumped from the bottom of the placement, displacing the water as it rises. The use of preplaced-aggregate concrete is covered in ACI 304.1R and ACI 546.2R.

2.7—Bonding methods

Repair materials may or may not require a separate bonding agent. In either case, the success of the repair depends on achieving intimate and continuous contact between the repair material and the substrate. Intimate contact can be achieved by vibration, pneumatic application, high fluidity, and troweling pressure in conjunction with attaining an optimum surface profile.

In cases where a separate bonding agent is to be used, application of the bonding agent to the prepared substrate should be done with care and should be timed to the placement of the repair material. Bonding agents applied to substrates may begin setting or curing prematurely, creating a bond breaker with the new repair material. Cement-based bonding agents are sprayed or broomed, whereas epoxy- and latex-based systems are rolled, broomed, or sprayed.

Whether the repair material is self-bonding or a separate bonding agent is used, tests should be conducted to ensure that bonding is taking place. In-place tensile pull-off tests are recommended to evaluate that bonding of repair materials is adequate. This testing provides for the evaluation of the adequacy of surface preparation, provides a thickness and consolidation assessment, evaluates the tensile (bond) strength, and assesses the failure mode. Such in-place testing should use the method described in ACI 503R, Appendix A.

CHAPTER 3—REPAIR MATERIALS

3.1—Introduction

This chapter contains descriptions of the various categories of materials that are available for repair or rehabilitation of concrete structures. Typical properties, advantages, disadvantages or limitations, typical applications, and applicable standards and references are discussed for each repair material. Also, general guidance on selection of repair materials is provided.

3.2—Cementitious materials

To match the properties of the concrete being repaired as closely as possible, PCC and mortar or other cementitious compositions using similar proportions of ingredients are the best choices for repair materials. The new cementitious repair material should be compatible with the existing concrete substrate.

3.2.1 *Conventional concrete*—Conventional concrete composed of portland cement, aggregates, and water is often used as a repair material. Admixtures are used to entrain air, accelerate or retard hydration, improve workability, reduce mixing water requirements, increase strength, or alter other properties of the concrete. Pozzolanic materials, such as fly ash or silica fume, may be used in conjunction with portland cement for economy, or to provide specific properties such as reduced early heat of hydration, improved later-age strength development, reduced permeability, or increased resistance to alkali-aggregate reaction and sulfate attack.

Concrete proportions should provide workability, density, strength, and durability necessary for the particular application (ACI 211.1). To minimize shrinkage cracking, the repair concrete should have a *w/cm* as low as possible and a coarse aggregate content as high as possible. According to ACI 201.2R, frost-resistant normalweight concrete should have a *w/c* not to exceed 0.45 for thin sections, such as 40 to 75 mm (1.5 to 3 in.), and 0.50 for all other structures. Mixing, transporting, and placing of conventional concrete should follow the guidance given in ACI 304R, ACI 304.1R, ACI 304.2R, ACI 304.5R, and ACI 304.6R.

a) *Advantages*—Conventional concrete is readily available, well understood, economical, has similar properties to the parent concrete, and is relatively easy to produce, place, finish, and cure. Generally, concrete mixtures can be proportioned to match the properties of the underlying concrete; therefore, conventional concrete is applicable to a wide range of repairs.

Conventional concrete can be easily placed under water using a number of well-recognized techniques and precautions to ensure the integrity of the concrete after placement (ACI 304R and 546.2R). Concrete is typically placed under water using a tremie or a pump.

b) *Limitations*—Conventional concrete should not be used in repairs where the aggressive environment that caused the original concrete to deteriorate has not been eliminated, unless a reduced service life is acceptable. For example, if the original deterioration was caused by acid attack, aggressive water attack, or even abrasion-erosion, repair with conventional concrete may not be successful unless the cause of deterioration is removed.

When used as a bonded overlay, the shrinkage properties of the repair material are critical because the new material is being placed on a material that has exhibited essentially all of the shrinkage that it will experience. Full consideration of the shrinkage properties and the curing procedure should be addressed in the specification for the repair.

Concrete that is mixed, transported, and placed under hot weather conditions of high temperature, low humidity, or wind requires measures to be taken to eliminate or minimize undesirable effects (ACI 305R). There are special requirements for producing and placing satisfactory concrete during cold weather (ACI 306R).

c) *Applications*—Conventional concrete is often used in repairs involving relatively thick sections and large volumes of repair material. Typically, conventional concrete is appropriate for partial- and full-depth repairs and resurfacing overlays where the minimum thickness is greater than 50 mm (2 in.) or the overlay extends beyond the reinforcement, or when the repair area is large. Conventional concrete is most commonly used for repairs of slabs, walls, columns, piers, and hydraulic structures (McDonald 1987), as well as for full-depth repairs or overlays on bridge or parking structure decks.

Conventional concrete is particularly suitable for repairs in marine environments because the typically high humidity in such environments minimizes the potential for shrinkage (Troxell, Raphael, and Davis 1958).

d) *Standards*—ASTM C 94 covers ready-mixed concrete manufactured and delivered to a purchaser in a freshly mixed and unhardened state. Properties such as shrinkage and bond are not included in this specification, and they should be specified separately if they merit special consideration in a given repair.

3.2.2 *Conventional mortar*—Conventional mortar is a mixture of portland cement, fine aggregate, and water. Water-reducing admixtures, expansive agents, and other modifiers are often used with conventional mortar to minimize shrinkage.

a) *Advantages*—The advantages of conventional mortar are similar to those of conventional concrete. In addition, mortar can be placed in thinner sections. A wide variety of prepackaged mortars is available. They are particularly appropriate for small repairs.

b) *Limitations*—Mortars generally exhibit increased drying shrinkage compared to concrete because of their higher water volume, higher unit cement content, higher paste-aggregate ratio, and the absence of coarse aggregate.

High air contents are often required to provide adequate resistance to freezing and thawing and scale resistance; however, high air content does reduce strength.

c) *Applications*—Conventional mortar can be used wherever thin repair sections are required. These applications should be reviewed when installed in the path of vehicular traffic that is subject to repetitive loading. Testing under actual field conditions is recommended to ensure the satisfactory performance of the material and installation.

d) *Standards*—ASTM C 387 covers the production, properties, packaging, and testing of packaged, dry, combined materials for concrete and mortar. Special consideration should be given to properties not covered in this specification, such as shrinkage and durability.

3.2.3 *Dry-pack mortar*—Dry-pack mortar consists of one part cement, two and one-half to three parts sand or prepackaged proprietary materials, and only enough water so the mortar sticks together when molded into a ball by slight pressure of the hands and does not exude water but leave the hands damp. Curing is critical because of the low initial water content of dry-pack mortar.

a) *Advantages*—Because of its low w/c, dry pack exhibits very little shrinkage. Therefore, the repair remains tight and is of good quality with respect to durability, strength, and water tightness. If the patch should match the color of the surrounding concrete, a blend of gray and white portland cement may be used. Normally, about 1/3 white cement is adequate, but the precise proportions can only be determined by trial.

b) *Limitations*—Dry pack is not well suited for patching shallow depressions or for patching areas requiring filling behind exposed reinforcement, or for patching holes extending entirely through concrete sections. Without adequate curing, dry pack repairs are subject to failure.

c) *Applications*—Dry pack can be used for filling large or small cavities, form tie holes, or any cavity that allows for adequate compaction. Such repairs can be accomplished on vertical and overhead surfaces without forms. Dry pack can also be used for filling narrow slots cut for the repair of dormant cracks; however, it is not recommended for filling or repairing active cracks. Dry pack is commonly used at load transfer points and contact areas.

d) *Standards*—Currently, there are no standards for dry-pack mortar.

3.2.4 *Proprietary repair mortars*—Proprietary repair mortars are prepackaged mortars that are a blend of portland cement or specialty cement, admixtures, plasticizers, expansive agents, densifiers, accelerators, polymers, thixotropic agents, or fine aggregate.

a) *Advantages*—The advantages of proprietary repair materials are their convenience of use in the field and the broad range of products with different physical and mechanical properties to meet the different field conditions, such as partial-depth repairs on vertical and overhead surfaces without the need of forms. They can have faster times of setting and shorter cure times.

b) *Limitations*—Proprietary repair mortars have different mechanical properties than the concrete being repaired. Because they may be cement-rich and extended with any one or more modifiers subsequently, they can have a propensity for high shrinkage compared with conventional concrete. There are no standards addressing bond strength of these mortars when applied to vertical or overhead applications.

c) *Applications*—Some proprietary mortars are recommended for repairs as thin as 3 mm (1/8 in.). These applications should be reviewed when installed in the path of vehicular traffic or subjected to repetitive loadings. Testing under actual field conditions is recommended to ensure the satisfactory performance of the material and the installation.

d) *Standards*—The existing standard, ASTM C 928, allows the inclusion of 1% calcium chloride by weight of cement, without advising the specifier or the contractor. The allowable shrinkage is high and may compromise the integrity of the repair. This standard is not all-inclusive and compliance to ASTM C 928 is no assurance of the desired quality.

3.2.5 *Ferrocement*—Ferrocement is a form of reinforced concrete that differs from conventional reinforced or prestressed concrete primarily by the manner in which the reinforcing elements are dispersed and arranged (ACI 549R). Ferrocement is commonly constructed of hydraulic cement mortar reinforced with closely spaced layers of continuous and relatively small-diameter wire mesh. The wire mesh may be made of steel, stainless steel, or other suitable materials.

a) *Advantages*—Ferrocement has a high tensile strength- to-weight ratio, increased durability, and superior cracking behavior in comparison to reinforced concrete.

b) *Limitations*—The use of ferrocement in a repair situation is limited by the nature of the repair and the cost.

c) *Applications*—Because no formwork is required, ferrocement is especially suitable for repair of structures with curved surfaces, such as shells, and other free-form shapes.

d) *Standards*—There are currently no standards for ferrocement. Additional information is available in ACI 549R and 549.1R.

3.2.6 *Fiber-reinforced concrete*—Fiber-reinforced concrete is conventional concrete with either metallic or polymeric fibers added to achieve greater resistance to plastic shrinkage, drying shrinkage, and service-related cracking. In most applications, fiber reinforcement is not intended as primary reinforcement. Fibers can be steel, glass, synthetic, or natural (ACI 544.1R). Fiber-reinforced concrete has been used for repairs using conventional and shotcrete placement methods. Information on fiber-reinforced concrete or shotcrete can be obtained from ACI 544.3R, ACI 544.4R, and ACI 506.1R.

a) *Advantages*—The fibers are added during concrete production and are in the concrete when it is placed. These fibers can be used to provide reinforcement in thin overlays that are not thick enough to include reinforcing bars. The addition of fibers can increase durability and reduce plastic shrinkage cracking of repair materials. Areas subject to shock or vibration loading, where plastic shrinkage cracking is a problem, or where blast resistance is required, could benefit from the addition of fiber reinforcement.

b) *Limitations*—The addition of fibers reduces the slump and can cause workability problems for inexperienced workers. Rust stains may occur at the surface of steel fiber-reinforced concrete due to corrosion of fibers at the surface.

c) *Applications*—Fiber-reinforced concrete has been used for slabs-on-ground, overlays of concrete pavements, slope stabilization, and reinforcement of structures, such as arches or domes. Reinforced concrete structures have been repaired with fiber-reinforced shotcrete.

d) *Standards*—ASTM C 1116 covers the materials proportions, batching, delivery, and testing of fiber-reinforced concrete and shotcrete.

3.2.7 *Grouts*—Grouts appropriate for concrete repair are categorized as either hydraulic cement or chemical.

3.2.7.1 *Cement grouts*—Cement grouts are mixtures of hydraulic cement, fine aggregate, and admixtures that, when mixed with water, produce a plastic, flowable, or fluid consistency without segregating the constituents. Admixtures are frequently included in the grout to accelerate or retard time of setting, minimize shrinkage, improve pumpability or workability, or improve the durability of the grout. Fly ash may be used to enhance performance or for reasons of economy when substantial quantities of grout are required. Silica fume may be used to increase chemical resistance, density, durability, and strength, and decrease permeability.

a) *Advantages*—Cement grouts are economical, readily available, easy to install, and fairly compatible with concrete. Admixtures can modify cement grouts to meet specific job requirements. Admixtures to minimize shrinkage are available on the market.

b) *Limitations*—Cement grouts can be used for repairs by injection only where the width of the opening is sufficient to accept the solid particles suspended in the grout. Normally, the minimum crack width at the point of introduction should be approximately 3 mm (0.1 in.).

c) *Applications*—Typical applications of hydraulic-cement grout may vary from grout slurries for bonding old concrete to new concrete, to filling of large dormant cracks, or to filling of voids around or under a concrete structure. Nonshrink cement grouts can be used to repair spalled or honeycombed concrete or to install anchor bolts in hardened concrete.

d) *Standards*—ASTM C 1107 covers three grades of packaged, dry, hydraulic-cement grouts (nonshrinkable) intended for use under applied load, such as to support a structure or a machine, where change in thickness below initial placement thickness is to be minimized.

3.2.7.2 *Chemical grouts*—Chemical grouts consist of solutions of chemicals that react to form either a gel, foam, or a solid precipitate as opposed to cement grouts that consist of suspensions of solid particles in a fluid. The reaction in the solution may involve only the constituents of the solution or it may include the interaction of the constituents of the solution with other substances, such as water, encountered in the use of the grout. The reaction causes a decrease in fluidity and a tendency to solidify and fill voids in the material into which the grout has been injected.

a) *Advantages*—The advantages of chemical grouts include their applicability in moist environments, their wide ranges of gel or setting time, and their wide range of viscosities. Cracks in concrete as narrow as 0.05 mm (0.002 in.) have been filled with chemical grout.

Rigid chemical grouts, such as epoxies, exhibit excellent bond to clean, dry substrates, and some bond to wet concrete. These grouts can restore the full strength of a cracked concrete member.

Gel-type or foam chemical grouts, such as sodium silicate, acrylates, acrylamides, and polyurethanes, are particularly suited for use in control of water flow through cracks and joints. Some gel grouts can be formulated at viscosities near that of water so they can be injected into almost any opening that water flows through.

b) *Limitations*—Chemical grouts have different mechanical properties and are more expensive than cement grout. Also, a high degree of skill is needed for satisfactory use of chemical grouts. Some epoxy systems do not bond in the presence of moisture.

Chemical bonding agents, such as epoxies, have relatively short pot lives and working times at high ambient temperatures.

Gel or foam grouts should not be used to restore strength to a structural member. Most gel or foam grouts are water solutions and exhibit shrinkage if allowed to dry in service.

c) *Applications*—Repair of fine cracks, either to prevent moisture migration along the crack or to restore the integrity of a structural member, are two of the most frequent applications of chemical grout.

d) *Standards*—ASTM C 881 covers two-component, epoxy-resin bonding systems for application to PCC that are able to cure under humid conditions and bond to damp surfaces.

3.2.8 *Low-slump dense concrete*—Low-slump dense concrete (LSDC) is a special form of conventional concrete. It has a moderate-to-high cement factor, a *w/c* less than 0.40, and exhibits working slumps of 50 mm (2 in.) or less. LSDC gains strength rapidly and is distinctive because of its high density and reduced permeability.

a) *Advantages*—Overlays of LSDC with a minimum thickness of only 40 mm (1.5 in.) have provided up to 20 years of service when properly installed. LSDC can be placed using conventional equipment with slight modifications. Compared to structural grade concrete, LSDC provides increased resistance to chloride-ion penetration when tested according to ASTM C 1202 (Ozyildirim 1993).

b) *Limitations*—LSDCs require maximum consolidation effort to achieve optimum density or the use of a high-range water-reducing admixture (HRWRA) to improve workability of the concrete and reduce the compaction effort needed to provide bond to the reinforcing steel and underlying concrete.

These low *w/c* concretes require at least 7 days of continuous moist curing to obtain adequate hydration.

LSDC permits galvanic corrosion, even with a 0.32 *w/c* and 25 mm (1.0 in.) cover (Pfeifer, Landgren, and Zoob 1987).

Drying-shrinkage cracks, depending on crack width and depth, can increase chloride-ion intrusion, resulting in corrosion of the reinforcing steel in bridge deck overlays (Babei and Hawkins 1988). Because there are no standards, the specifying engineer should review appropriate application and compatibility issues.

c) *Applications*—LSDC is frequently used as an overlay or final wearing course in a composite repair to obtain a quality, abrasion-resistant, protective layer with a durable concrete surface.

d) *Standards*—The same standards apply to LSDC as apply to conventional concrete.

3.2.9 *Magnesium phosphate concrete and mortar*—Magnesium phosphate concrete and mortar (MPC) is based on a hydraulic cement system that is different from portland cement. Unlike portland and some polymer-cement concrete, which require moist curing for optimum property development, these systems produce their best properties upon air curing, similar to epoxy concrete. Rapid strength development and heat are produced, although retarded versions are available that produce less heat. These materials have been used in repairs to concrete since the mid 1970s.

a) *Advantages*—Setting times of 10 to 20 min are typically encountered at room temperatures, and early strength development of 14 MPa (2000 psi) within 2 h is regularly obtained. Retarded versions with extended setting times of 45 to 60 min at room temperature are also available. Scale resistance is similar to air-entrained, portland-cement-based concrete materials. When extended with aggregates, abrasion resistance of MPC is similar to conventional concrete of similar strength, when tested in accordance with ASTM C 672. Neat magnesium phosphate cement naturally has lower abrasion resistance, similar to portland-cement mortar of equal strength.

Magnesium phosphate mortar and concrete can be placed at temperatures as low as 32 °C (0 °F), or lower if the mixing water and material are heated. Magnesium phosphate mortars typically achieves 40 MPa (6000 psi) compressive strength within 48 h and does not increase very much beyond that level. The material has good bond strength to portland cement and low permeability. Thin patches with magnesium phosphate mortar perform better than portland cement patches because they do not require a moist cure.

b) *Limitations*—MPC should be extended only with noncalcareous aggregates such as silica, basalt, granite, trap rock, and other hard rocks. Reaction of carbonated surfaces with the early-forming phosphoric acid produces carbon dioxide (CO_2) and weakens the paste aggregate bond. Because of its acid-base reaction, MPC should be used only on well-prepared concrete substrates that have had the carbonation layer removed by mechanical or chemical means. MPC reacts chemically with the dust of the fracture or carbonated zone and can cause a reduction in bond strength at the bond interface.

Careful attention should be paid to surface preparation and drying times of MPC if impermeable membranes are applied over it. The MPC and membrane manufacturers should be contacted for detailed recommendations.

Because of the small interval between initial and final setting times, MPC generally is not hard troweled.

The field technician does not have the flexibility of varying the water content, and therefore, MPC cannot be placed in a dry-pack consistency. The mixing water typically has a tolerance of only ±10%. Any variation of the water content from that specified by the manufacturer reduces both the strength and the durability of the MPC mortar.

The neat or extended magnesium phosphate mortar develops a very rapid exothermic reaction that can produce relatively high temperatures. The mortar should not be placed at temperatures above 27 °C (80 °F) or in the sunlight within the temperature ranges of 15 to 27 °C (60 to 80 °F). Hot weather formulas are available for use in warm ambient conditions. The manufacturer should be consulted when placing the material in thickness greater than 100 mm (4 in.) or in insulated cavities where the heat generated cannot escape. Extending the mixture with aggregates or cool water can slow down the exothermic reaction.

In a hardened state, MPC quickly produces high strength and high modulus of elasticity. Therefore, it is not flexible and does not have the toughness that is typically found with organic-modified mortars and is susceptible to fracturing from impact loads.

With the normal setting formulations of MPC, high heat peaks are encountered. With sustained exposure to temperatures in excess of 80 °C (180 °F) in service, strength reductions can develop, but its compressive strength can be expected to exceed that of normal concrete.

c) *Applications*—Patching applications are the most common use of MPC, and are cost effective for rapid repairs where a short down time is important. Common uses are in highway, bridge deck, airport, tunnel, and industrial repairs.

Repairs in a cold-weather environment are important applications. Due to the exothermic nature of the reaction, heating the materials and the substrates is not usually necessary, unless the temperature is below freezing. MPC is useful for cold-weather embedments and anchoring because of its high bond strength and low shrinkage rate.

d) *Standards*—Currently, no standards are available for MPC.

3.2.10 *Preplaced-aggregate concrete*—Preplaced-aggregate concrete is produced by placing coarse aggregate in a form and later injecting a portland cement-sand grout, usually with admixtures or a resinous material, to fill the voids. Preplaced-aggregate concrete differs from conventional concrete in that it contains a higher percentage of coarse aggregate. ACI 304R and ACI 304.1R gives guidance on mixing and placing preplaced-aggregate concrete.

a) *Advantages*—Because of the point-to-point contact of the coarse aggregate, drying shrinkage of preplaced-aggregate concrete is approximately 1/2 that of conventional concrete. Because the aggregate is preplaced and the grout pumped under pressure, segregation is not a problem, and virtually all substrate voids are filled with mortar. The ability of the grout to displace water from the voids between aggregate particles during injection makes this material particularly suitable for underwater repairs (ACI 304R, ACI 304.1R, and ACI 546.2R).

In underwater construction, higher placing rates at lower cost have been achieved with preplaced-aggregate concrete compared to conventional placing methods (ACI 304R and ACI 304.1R).

Preplaced-aggregate epoxy concrete—Preplaced-aggregate epoxy concrete, based on field experience, performs in a manner similar to PCC under load.

b) *Applications*—Preplaced-aggregate concrete is used on large repair projects, particularly where underwater concrete placement is required or when conventional placing of concrete would be difficult. These factors make preplaced-aggregate concrete an ideal material for applications where considerable congestion of reinforcement or other embedments exists, or where access is difficult. Preplaced-aggregate concrete provides a solution for structural repairs requiring immediate load transfer capability because of its low shrinkage. Typical applications include underwater repair of stilling basins, dams, bridges, abutments, walls, beams, columns, and footings. Preplaced-aggregate concrete has also been used to repair beams and columns in industrial plants, water tanks, and other similar facilities, as well as caissons for underpinning existing structures.

c) *Limitations*—Formwork costs for preplaced-aggregate concrete are approximately the same as for conventional concrete; however, additional work may be required in the installation of forming because of the need to preplace the aggregate and prevent leaks. Because of the relatively high water content commonly used to yield pumpable cementitious mortars, the permeability to gas or vapor of the mortar fraction of preplaced-aggregate concrete can be somewhat greater than that of normal concrete, which is an important factor to consider where it is to be used in extreme environments. Inclusion of silica fume in the grout may mitigate this limitation (Watson 1996).

Because preplaced-aggregate concrete construction is specialized, repairs using this procedure should be conducted by qualified personnel experienced in this method of construction.

d) *Standards*—ASTM C 937 covers fluidifier for grout used for preplaced-aggregate concrete.

ASTM C 938 describes the practice for selecting proportions for grout mixtures required in the production of preplaced-aggregate concrete.

3.2.11 *Rapid-setting cements*—Rapid-setting cementitious materials are characterized by short setting times. Some may exhibit rapid strength development with compressive strengths in excess of 17 MPa (2400 psi) within 3 h. Type III portland cement has been used for the patching of concrete for a long time and has been more widely used than most other materials in full-depth sections (Transportation Research Board 1977).

a) *Advantages*—Rapid-setting cements provide accelerated strength development that allows the repair to be placed into service more quickly than conventional repair materials. This advantage is important in repairing highways, bridges, airport runways, and industrial plants because of the reduced protection times, lower traffic-control costs, and improved safety.

b) *Limitations*—Although most rapid-setting materials are as durable as concrete, some, due to their constituents, may not perform well in a specific service environment.

Some rapid-setting materials obtain their strength development and expansion from the formation of ettringite. If the level of expansion is high and the time to attain the maximum levels of expansion is long, strength retrogression may occur. The potential for delayed expansion resulting from insufficient initial curing followed by rewetting should be recognized.

Because some of these materials may contain abnormally high levels of alkali or aluminate to provide expansion, their exposure to sulfates and reactive aggregates should be limited. The specifying engineer should review appropriate application and compatibility issues.

c) *Applications*—Rapid-setting cements are especially useful in repair situations where an early return to traffic is required, such as repair of pavements, bridge decks, airport runways, and industrial plants.

d) *Standards*—ASTM C 928 covers packaged, dry, cementitious mortar or concrete materials for rapid repairs to hardened hydraulic-cement concrete pavements and structures. ASTM C 928 does not provide bond strength or freezing-and-thawing resistance requirements. The specification permits up to 1% calcium chloride without acknowledgement to the specifier or user. Also, a current footnote cautions the user to check on exposure conditions (sulfate exposure and alkali reactivity) that are not covered in the specification. Therefore, additional testing should be performed at anticipated application temperatures to verify if properties not covered in the specification are important for a given project. Substantial advances in the compounding of rapid setting cements have taken place in recent years.

3.2.12 *Shotcrete*—Shotcrete is a mixture of portland cement, sand, and water that is projected into place by compressed air. In addition to these materials, shotcrete can also contain coarse aggregate, fibers, and admixtures. Properly applied shotcrete is a structurally adequate and durable repair material that is capable of excellent bond with existing concrete or other construction materials.

a) *Advantages*—In repair projects where thin sections less than 150 mm (6 in.) in depth and large or small surface areas with irregular contours or shapes are involved, shotcrete may be more economical than conventional concrete because of the savings in forming costs.

Shotcrete can be applied overhead in normal applications, and materials can be mixed and transported several hundred feet to the nozzle operator at project sites with restricted access. Mechanical equipment is also available for remote placement of shotcrete. For shotcrete dry-mix, refer to ACI 506R.

b) *Limitations*—The successful application of shotcrete is dependent on the training, skill, and experience of the nozzle operator. The nozzle operator should demonstrate his or her skill by placing a test panel that reflects the site conditions.

The nozzle operator's performance should be evaluated and approved before he or she is allowed on the job. The nozzle operator should be ACI certified as a nozzle operator.

Dust and rebound require special attention in indoor applications.

c) *Applications*—Shotcrete has been used to repair deteriorated concrete bridges, buildings, parking structures, lock walls, dams, and tunnels. The performance of shotcrete repair has generally been good; however, there are some instances of poor performance. Major causes of poor performance are inadequate preparation of the old surface, poor workmanship, and not accounting for the relatively impermeable nature of shotcrete. There have been some repair failures where shotcrete was used to resurface old hydraulic structures. The relatively impermeable shotcrete traps moisture migrating through the concrete substrate causing the nonair-entrained concrete near the repair interface to become critically saturated. Subsequent cycles of freezing and thawing resulted in failure of the repair (U.S. Army Corps of Engineers 1995). Satisfactory shotcrete repair is contingent upon proper surface treatment of old surfaces to which the shotcrete is being applied and the skill of the nozzle operator.

d) *Standards*—ACI 506R and ACI 506.2 provide guides and specifications for shotcrete construction and certification requirements for the nozzle operator.

ASTM C 1116 covers the materials proportions, batching, delivery, and testing of fiber-reinforced concrete and shotcrete.

3.2.13 *Shrinkage-compensating concrete*—Shrinkage-compensating concrete is an expansive-cement concrete that minimizes cracking caused by drying shrinkage. The basic materials and methods are similar to those necessary to produce high-quality PCC. Consequently, the characteristics of shrinkage-compensating concrete are, in most respects, similar to those of portland-cement concrete. Proprietary, prepackaged mortar that uses some form of shrinkage-compensating cement to counter potential shrinkage is available and has been used successfully for years. For definitions of expansive cement, cement types K, M, and S, refer to ACI 223.

a) *Advantages*—When properly restrained by reinforcement or external restraint, shrinkage-compensating concrete tries to expand an amount equal to or slightly greater than the anticipated drying shrinkage, but because expansion is limited by restraint, reinforcement, or both, the concrete initially develops compressive stresses. Subsequent drying shrinkage reduces these expansive strains, compressive stresses, or both, but ideally, a residual expansion remains in the concrete, thereby reducing shrinkage cracking. The joints used to control shrinkage cracking can be reduced or eliminated along with the normal provisions for waterstops and load-transfer mechanisms. Where a watertight condition is essential, however, the elimination of waterstops is not recommended.

b) *Limitations*—Although its characteristics are similar to those of portland-cement concrete, the materials, selection of proportions, placement, and curing should be such that sufficient expansion, compressive stresses, or both, are obtained to compensate for subsequent drying shrinkage. The criteria and practices necessary to ensure that expansion occurs at the time and in the amount required are given in ACI 223. Low curing temperatures can reduce expansion. Special cleaning of transit mixer drums may be required to prevent contamination.

Provisions should be made to allow for initial expansion of the material to provide positive strain on the internal steel restraint. Consequently, shrinkage-compensating concrete is not effective in bonded overlays on portland-cement concrete because the substrate provides too much external restraint. The potential for the forces created during the expansion process could push out walls or fail the forms along the perimeter of the pours.

c) *Applications*—Shrinkage-compensating concrete minimizes cracking caused by drying shrinkage in concrete slabs, pavements, bridge decks, and structures. Additionally, shrinkage-compensating can reduce warping tendencies where concrete is exposed to single face drying and carbonation shrinkage.

d) *Standards*—ASTM C 845 provides standards for expansive hydraulic cement and limits, including strength, setting time, and expansion of the cement. Mortar and concrete expansions are usually determined in accordance with ASTM C 806 and C 878, respectively. Adequacy of concrete should be checked and used as outlined in ACI 223.

3.2.14 *Silica-fume concrete*—Silica fume, a by-product in the manufacture of silicon and ferrosilicon alloys, is an efficient pozzolanic material. Adding silica fume and a HRWRA to a concrete mixture significantly increases compressive strength, decreases permeability, and thus improves durability (ACI 234R). Silica fume is normally added to concrete in quantities of 5 to 15% by mass of cement. Compressive strengths of 85 to 105 MPa (12,000 to 15,000 psi) can be attained with silica-fume concrete.

a) *Advantages*—The initial commercial interest in silica fume was for increased chemical resistance of concrete; however, it is being added to concrete today to increase density and strength, and, in some cases, as a cement replacement or property-enhancing material to improve quality and performance in a wide range of applications.

Silica-fume concrete requires no significant changes from the normal transporting, placing, and consolidating practices associated with conventional concrete, except HRWRAs are required. Further, any retempering of silica fume modified concrete should be done with a HRWRA rather than with water.

b) *Limitations*—Typically, as silica-fume dosage increases, the concrete becomes more cohesive, is more susceptible to plastic shrinkage cracking, and adds significant cost to the repair material. Placing and finishing crews, however, have been able to overcome these differences without any significant difficulties (Holland 1987). Silica-fume concrete has little or no bleed water, which makes it difficult to provide a steel trowel finish.

The minimum curing temperature should be 4 °C (40 °F). Also, wet curing should start immediately for a recommended minimum of 7 days.

c) *Applications*—The first major applications of silica-fume concrete in the United States were for repair of hydraulic structures subjected to abrasion-erosion damage (Holland and Gutschow 1987). The high strength of silica-fume concrete and the resulting abrasion-erosion resistance offer an economical solution to abrasion-erosion problems, particularly in areas where locally available aggregate may otherwise result in unacceptable concrete for abrasion/erosion resistance.

Silica-fume concrete has been used extensively in overlays on parking structures and bridge decks to reduce the intrusion of chloride ions into the concrete.

d) *Standards*—ASTM C 1240 is the specification covering silica fume.

3.3—Polymer materials

The improvement of properties of hardened concrete by the addition of polymers is well documented. A bibliography of major references covering polymers in concrete and a glossary of terms are included in ACI 548.1R. This guide presents information on various types of polymer materials and their storage, handling, and use, as well as on concrete formulations, equipment to be used, construction procedures, and applications.

Two basic types of concrete materials use polymers to form composites: polymer-cement concrete (PCC) and polymer concrete (PC).

3.3.1 *Polymer-cement concrete and mortar*—Polymer-cement concrete (PCC) has, at times, been called polymer-portland-cement concrete (PPCC) and latex-modified concrete (LMC). It is identified as portland cement and aggregate combined, at the time of mixing, with organic polymers that are dispersed or redispersed in water. This dispersion is called a latex, and the organic polymer is a substance composed of thousands of simple molecules combined into large molecules. The simple molecules are known as monomers, and the reaction that combines them is called polymerization. The polymer may be a homopolymer if it is made by the polymerization of one monomer, or a copolymer if two or more monomers are polymerized.

In this section, the use of PCC includes both mortar and concrete.

Polymer dispersions are added to the concrete mixture to improve the properties of the final product. These properties include improved bond strength to concrete substrates, increased flexibility and impact resistance, improved resistance to penetration by water and by dissolved salts, and improved resistance to freezing and thawing.

Of the wide variety of polymers investigated for use in PCC, polymers made by emulsion polymerization have been the most widely used and accepted (ACI 548.3R). Styrene butadiene and acrylic latexes have been the most effective and predictable for concrete restoration. Other latexes commonly used include polymers and copolymers of vinyl acetate.

When emulsified and mixed with concrete, epoxies provide excellent freezing-and-thawing resistance, significantly reduced permeability, and improved chemical resistance. Bond is excellent, and flexural, compressive, and tensile strengths are high. Epoxy emulsions, however, have had limited use in concrete.

Mixture proportions depend on the specific application and the type of polymer used in the PCC. Polymer levels of 10 to 20% polymer solids by mass of cement are required for most desired applications. Typical *w/c* for workable PCC mixtures used for repair range from 0.30 to 0.40 for mixtures containing latexes, and 0.25 to 0.35 for mixtures containing epoxies (ACI 548.1R).

a) *Advantages*—PCC overlays have exhibited excellent long-term performance. Properly installed overlays are highly resistant to freezing-and-thawing damage, and they exhibit minimal bond failure after many years of service. LMC overlays installed on severely deteriorated bridge decks, after proper surface preparation, continue to perform many years after installation. Reports of satisfactory long-term performance on structures of variable initial condition and harsh in-service exposure are common (Virginia 2001).

Mixing of PCC should be done in a concrete mobile mixer. Handling, placing, and finishing of PCC is limited to less than 30 min. To achieve proper curing, PCC requires one day to two days of moist curing followed by air drying. The PCC is placed in service when it has developed sufficient strength, which is dependent upon the hydration of the cement.

An advantage of PCC is its good workability and ease of application when compared to similar systems.

The bonding characteristics of PCC are excellent (Kuhlmann 1990), and PMC usually exhibits low permeability (Kuhlmann 1984).

Styrene-butadine PCC has excellent durability for exterior exposures or environments where moisture is present. Surface discoloration occurs when the concrete is exposed to UV light. Where such discoloration is not acceptable, acrylic polymers should be used.

b) *Limitations*—PCC should be placed and cured at 7 to 30 °C (45 to 85 °F) with special precautions taken when either extreme is reached (ACI 548.1R and ACI 306.1).

Mobile, continuous mixers, fitted with an additional storage tank for the latex should be used for large applications of PMC. When small batches are mixed in drum or mortar mixers, the mixing time should be limited to 3 min. Longer mixing times result in an increase in the total air content with subsequent reductions in compressive strength.

PMC has a tendency for plastic shrinkage cracking during field placement. Special precautions are necessary when the evaporation rate exceeds 0.5 $kg/m^2/h$ (0.1 $lb/ft^2/h$) (ACI 308R and ACI 548.4).

The modulus of elasticity is generally lower compared to conventional concrete; therefore, its use in vertical or axially loaded members should be carefully evaluated (Kuhlmann 1990).

Polyvinyl acetate should not be used in applications that may be exposed to moisture.

Epoxy emulsions are more expensive than most latexes, and some are susceptible to color change and deterioration from exposure to sunlight.

c) *Applications*—PCC applications include overlays of bridge decks, parking structures and floors, and patching of any concrete surfaces. Styrene butadiene latex is commonly used for repair of bridges, parking decks, and floors. Acrylic latexes are used for floor repair and patching and are particularly suitable in exterior white cement applications where color retention is important.

PCC is most commonly used for overlays and is normally applied in sections ranging from 40 to 100 mm (1.5 to 4 in.) thick for concrete and 20 to 40 mm (0.75 to 1.5 in.) for mortars. These systems restore lost sections and provide a new, high-strength wearing surface that is very durable against weathering.

Although used as overlay materials, PCC is an effective patching materials. Because most patches and repairs in which PCC is used are relatively shallow, mixture proportions similar to those shown in ACI 548.3R should be considered.

d) *Standards*—ASTM C 685 covers concrete made from materials continuously batched by volume and mixed in a continuous mixer. Tests and criteria for batching accuracy and mixing efficiency are specified. ACI 548.4 covers the placement of styrene-butadiene LMC overlays for bridge decks. ASTM C 1438 is the specification for latex and redispersible polymer powder modifiers, and ASTM C 1439 covers test methods for PCC.

3.3.2 *Polymer concrete*—PC is a composite material in which the aggregate is bound together in a dense matrix with a polymer binder. The composites do not contain a hydrated cement phase, although portland cement can be used as an aggregate or filler. The term PC should never suggest a single product but rather a family of products. Use of the term PC in this section also includes mortar.

PC has been made with a variety of resins and monomers including polyester, epoxy, furan, vinylester, HMWM, and styrene (ACI 548.1R). Polyester resins are attractive because of moderate cost, availability of a great variety of formulations, and moderately good PC properties. Furan resins are low cost and highly resistant to chemical attack. Epoxy resins are generally higher in cost, but may offer advantages, such as low shrinkage, while some formulations bond to damp surfaces that do not require a primer. Detailed information on the use of epoxy compounds with concrete is available (ACI 503R). A special form of PC is sulfur concrete, which is placed hot and is thermosetting. For further information, refer to ACI 548.2R.

The properties of PC are dependent on the properties and the amount of the polymer used, modified somewhat by the effects of the aggregate and the filler materials. Typically, PC mixtures exhibit rapid curing; high tensile, flexural, and compressive strengths; good adhesion to most surfaces; good freezing-and-thawing resistance; low permeability to water and aggressive solutions; and good chemical resistance.

a) *Advantages*—PC can provide a fast-curing, high-strength patching material that is suitable for repair of PCC structures. PC is mixed, placed, and consolidated in a manner that is similar to conventional concrete. With some harsh mixtures, external vibration is required.

A wide variety of prepackaged polymer mortars is available that can be used as mortars or added to selected blends of aggregates. Depending on the specific use, mortars may contain variable aggregate gradations intended to impart unique surface properties or aesthetic effects to the structure being repaired. Also, polymer mortars are available that are trowellable and specifically intended for overhead or vertical applications.

Epoxy mortars shrink less than polyester or acrylic mortars. Shrinkage of polyester and acrylic mortars can be reduced by using an optimum aggregate loading. The aggregate grading and the mixture proportions should be available from the polymer formulator.

Polymer mortars are ideally suited to thin patches and overlays less than 20 mm (0.75 in.).

b) *Limitations*—Organic solvents are required to clean equipment when using polyesters and epoxies. Manufacturers should be consulted before using proprietary materials to address the issue of volatile organic compound (VOC) restrictions.

Rapid curing generally means less time for placing and finishing operations. Working times for these materials are variable and, depending on ambient temperatures, may range from less than 15 min to more than 1 h. Also, high or low ambient and concrete temperatures may significantly affect polymer cure time or performance.

The coefficients of thermal expansion of polymer materials are variable from one product to another and are significantly higher than conventional concrete.

Shrinkage characteristics of PCs should be closely evaluated so that unnecessary shrinkage cracking is avoided.

The modulus of elasticity of PC is significantly lower than that of conventional concrete, especially at higher temperatures. Its use in load-carrying members should be carefully considered.

Only a limited number of polymer systems are appropriate for repair of wet concrete surfaces. The aggregates used in PC should be dry to obtain the highest strengths and ensure the integrity of the concrete or mortar.

High temperatures can adversely affect the physical properties of certain PC, causing softening. Service temperatures should be evaluated before selecting PC systems for such use. Epoxy systems may burn in fires where temperatures exceed 230 °C (450 °F) and can significantly soften at lower temperatures. Users of PC should consider its lack of fire resistance.

Conventional concrete generally does not bond to cured PC, unless the aggregate is exposed and free of any polymer.

c) *Applications*—PC patching materials are designed for the repair of a broad variety of conditions that allow closing of a repair area for only a few hours. PCs are not limited to that use, however, and can be formulated for a wide variety of applications.

PC is used in several types of applications: fast-curing, high-strength patching of structures, and thin 3 to 19 mm (0.1 to 0.75 in.) thick overlays for floors and bridge decks.

Polymer mortars have been used in a variety of repairs where only thin sections (patches and overlays) are required. Polymers with high elongation and low modulus of elasticity are particularly suited for bridge overlays.

PC overlays are especially well suited for use in areas where concrete is subject to chemical attack.

For exterior applications, PCs using a preplaced aggregate procedure have been successfully used in large volumes with cross sections greater than 0.1 m^2 (1 ft^2) and greater than 4.5 m (15 ft) in length (Murray et al. 1993).

d) *Standards*—ACI 503.4 is a specification for repair of defects in hardened PCC with a sand-filled mortar using an adhesive binder as defined in ASTM C 881. It includes requirements for adhesive labeling, storage, handling, mixing and application, surface evaluation and preparation, and inspection and quality control.

ASTM C 881 covers two-component, epoxy-resin bonding systems for application to PCC, which are able to cure under humid conditions and bond to damp surfaces.

3.4—Bonding materials

Bonding materials can be used to bond new repair materials to an existing prepared concrete substrate. Bonding materials are of three types: epoxy based, latex based, and cement based.

a) *Epoxy*—Epoxy systems are covered in ASTM C 881. Care should be taken when using these materials in hot weather. High temperatures may cause premature curing and creation of a bond breaker. Most epoxy-resin bonding materials create a moisture barrier between the existing substrate and the repair material. Under certain conditions, a moisture barrier could result in failure of the repair when moisture is trapped in the concrete directly behind the moisture barrier and freezing occurs in this zone.

b) *Latex*—Latex systems are covered in ASTM C 1059. Latex bonding agents are classified as Type I-Redispersible and Type II-Nonredispersible. Type I bonding agents can be applied to the bonding surface several days before placing the repair materials; however, the bond strength is less than that provided by Type II bonding agents. Type I bonding agents should not be used in areas subject to water, high humidity, or structural applications. Type II systems act as bond breakers once they have skimmed over or cured. Type II systems are best suited for bonding when mixed with cement and water to produce a slurry. These are most commonly used with PMC mixtures.

c) *Cement*—Cement-based systems have been successfully used for many years. Cement bonding systems use neat portland cement or a blend of portland cement and fine aggregate filler proportioned one to one by mass. Water is added to provide a uniformly creamy consistency. Cementitious slurries are commonly used with PCC.

3.5—Coatings on reinforcement

Some epoxy, zinc-rich, and latex-cement coatings have been proven in laboratory environments, but their performance capability, due to holidays in the coating when field applied, have been seriously challenged. In addition, once they are applied, corrosion activity can no longer be determined by standard testing procedures.

A thorough literature search discussing the pros and cons of these materials and consultation with the manufacturer should be conducted before using these materials.

3.6—Reinforcement

Structural concrete members typically require reinforcement when subject to tensile stresses due to flexure, shear, and axial loads. When the repair design requires reinforcement, several options are available, which include various corrosion-resistant materials and conventional reinforcing steel.

3.6.1 *Uncoated steel reinforcement*—ACI 318 specifies steel reinforcement as deformed bars, spiral bars, and welded-wire fabric, with grades conforming to ASTM A 615, A 616, A 617, or A 706 for bars; ASTM A 185 for plain wire fabric; and ASTM A 497 for deformed wire fabric.

ACI 301 and 318 further specify minimal cover for various environmental exposures, maximum levels of chlorides, recommended *w/c*, and other guidelines to improve concrete performance, all of which contribute to minimizing the potential for corrosion of conventional reinforcing materials.

3.6.2 *Fusion-bonded epoxy coating for steel reinforcement*—Epoxy coatings on conventional reinforcing steel (ASTM A 775) was introduced in the mid 1970s to act as a barrier to the elements that initiate corrosion, that is, oxygen, moisture, and chlorides. Though shown to be an effective method of reducing steel corrosion in concrete bridge decks, performance in the splash zones has shown that epoxy-coating performance is subject to the quality of the coating, damage to the coating during installation, extent of concrete cracking, depth of cover, loss of adhesion between coating and reinforcement, and level of chloride concentrations.

3.6.3 *Galvanized reinforcement*—Hot-dipped galvanized reinforcing steel is also available as a method for reducing corrosion in reinforced concrete. ASTM A 767 specifies the requirements for zinc-coated (galvanized) reinforcing bars with repair procedures identified by ASTM A 780. The galvanizing process is based on the zinc coating providing a sacrificial corrosion layer around the reinforcing steel bars.

3.6.4 *Stainless steel reinforcement*—Solid stainless steel reinforcement is very resistant to corrosion in concrete, with Grades 304 and 316 being the most commonly used. Grade 316, however, is reported to be slightly more resistant to chloride environments and unaffected by cracking in the vicinity of a common reinforcing bar, which acts as a cathode. Stainless steel reinforcement can also be field fabricated and is resistant to surface damage during handling and concrete placement. Cost is the primary disadvantage, restricting its use.

3.6.5 *Stainless-steel-clad reinforcement*—Stainless-steel-cladded reinforcement offers similar advantages as solid stainless steel bars but at substantially reduced costs. Field fabrication, including cutting, bending and welding, is practical. Repairs at cut ends, however, are usually specified to be coated.

3.6.6—*Composite nonmetallic reinforcement*

3.6.6.1 *Fiber-reinforced polymers*—FRPs are composite systems comprised of high-strength fibers and a resin matrix. They generally use an epoxy resin, although other resin systems exist, such as vinyl esters and polyesters. The typical fibers are carbon, glass, or aramid, with material properties, durability, and cost varying for each.

a) *Advantages*—FRP systems offer a high strength-weight ratio, allowing simple installation. The systems can be designed to perform similarly to conventional repair schemes such as steel plating and shotcreting. FRP materials are not subject to corrosion and are resistant to chemical environments.

Systems can be installed indoors with little mess and impact to occupants. Minimal equipment, access, lead time, and cure time are required for proper performance of FRP systems.

b) *Limitations*—Temperature and weather can influence the ability to install FRP systems. Extreme temperatures and moisture should be avoided during the installation of such materials. Typically, service temperature should stay below 77 °C (170 °F). (Note: Some materials marketed may be below this service level.) These systems should also be protected against ultraviolet light exposure or where fire protection issues are a concern.

The successful application of FRP systems is dependent on the proven performance of the individual systems; proper design methodology; environmental protection; and training, skill, and experience of the installer. Each manufacturer should demonstrate successful structural tests and material durability of its FRP system, providing the designer with the necessary data to perform a proper design consistent with accepted practices and allowing a quality control program to ensure proper system installation. Due to the inherent differences within FRP systems, designers should not rely on the test data of a manufacturer of another system.

c) *Applications*—FRP systems have been tested and used for numerous repair and strengthening projects. Columns, beams, slabs, and walls can be strengthened for increased seismic performance, additional load capacity, or damage repair.

FRP systems have successfully performed through earthquakes and have generally performed as designed in field applications. For bonded applications (beams, slabs, walls), the concrete should be sound and proper surface preparation of the concrete substrate requires and should follow provisions as set in Section 2.3.

d) *Standards*—ASTM (ACI 440.3R) covers the determination of material properties for individual material systems. ICBO AC125 defines acceptance criteria for the use of FRP systems, including validation testing requirements, design methods, durability tests, and installation quality control. A manufacturer's system should meet the requirements as set within this document.

3.7—Material selection

It is apparent from the preceding discussion that a wide variety of conventional and specialty materials are available for repairing concrete. While this large selection provides a greater opportunity to match material properties with specific project requirements, it also increases the potential for selection of an inappropriate material. Bond and compressive strengths are obviously important material properties in many repairs, and they are provided frequently by material suppliers. Some other material properties, however, can be of equal or greater importance (Warner 1984).

3.7.1 *Coefficient of thermal expansion*—It is important to use a repair material with a coefficient of expansion similar to that of the existing concrete. Thermal compatibility is particularly important in repairs subjected to wide variations in temperature, large repairs, and overlays. Thermal incompatibility could cause failure either at the interface or within the material of lower strength.

3.7.2 *Drying shrinkage*—Because most repairs are made on older PCC that does not undergo further shrinkage, the repair material should have minimal shrinkage to keep from losing bond. Performance criteria for dimensionally compatible, cement-based repair materials limits 28-day strength and ultimate drying shrinkage to 400 and 1000 millinonths, respectively (McDonald et al. 2002). Shrinkage of cementitious repair materials can be reduced by using mixtures with low *w/c*, the maximum practical size and volume of course aggregate, shrinkage-reducing admixtures, or using construction procedures that minimize the shrinkage potential. Examples include dry pack and preplaced-aggregate concrete. Shrinkage potential is much greater in thin patches, such as less than 40 mm (1.5 in.), with cementitious materials. Further, the shrinkage potential increases as the thickness decreases. Proper curing methods should be followed.

3.7.3 *Permeability*—Good-quality concrete is relatively impermeable to liquids, but when moisture evaporates at a surface, replacement liquid is pulled toward the dry surface by capillary action. If impermeable materials are used for large patches, overlays, or coatings, moisture that rises up through the base concrete can be trapped between the concrete and the impermeable repair material. The entrapped moisture can cause failure at the bond line or critically saturate the base concrete and cause failure in freezing and thawing if the concrete substrate does not contain a properly entrained air-void system (Headquarters 1995).

3.7.4 *Modulus of elasticity*—For repaired areas that are subjected to loading, the modulus of elasticity of the repair material should be similar to that of the existing concrete. In nonstructural or protective repairs, a lower modulus repair material is desirable to aid in the relaxation of tensile stresses induced by restrained drying shrinkage. A correlation of results from laboratory and field exposure tests indicates that the modulus of elasticity for cement-based repair materials should not exceed 24 GPa (3.5×10^6 psi) (McDonald et al. 2002).

3.7.5 *Chemical properties*—Special attention has been directed in recent years to problems involving corrosion of embedded reinforcement. A pH close to 12 (alkaline environment) of concrete provides corrosion protection to embedded reinforcement. Repair materials with moderate to low pH values, however, may provide little, if any, protection to embedded reinforcement. When such materials should be used, because of constraints such as cure time or strength requirements, additional protection for the existing reinforcement should be considered. This protection could include techniques such as cathodic protection or reinforcement coatings. Each system has its own cost-benefit ratio and should be evaluated for each specific project.

3.7.6 *Electrical properties*—The resistivity or electrical stability of a repair material can also affect the performance of corrosion-damaged concrete following repair. Materials that have high resistance tend to isolate repaired areas from adjacent undamaged areas. Differentials in electrical potential resulting from variations of permeability or chloride content between the repair material and the original concrete could increase corrosion activity around the perimeter of the repair area, resulting in premature failure. This is commonly referred to as the anodic ring or halo effect. Refer to Section 4.3.5 for further discussion.

The conditions under which the repair material are applied and its anticipated service conditions are also important considerations in material selection. Almost every repair project has unique conditions and special requirements, and only after these have been thoroughly examined can the final repair criteria be established. Once the criteria are known, often more than one material can be used with equally good results. Final selection of the material or combination of materials should then take into consideration the ease of application, cost, and available labor skills and equipment (Warner 1984). In areas where failure of a repair could create a safety hazard, such as repairs on a wall above a public area, the repair should be designed with adequate mechanical anchorage.

3.7.7 *Color and texture properties*—For repair of architectural concrete surfaces, color and texture of the repair material should not differ appreciably from the adjacent surface. Trials should be made on a job-site mockup before beginning actual production work.

CHAPTER 4—PROTECTIVE SYSTEMS

Protective systems consist of materials and methods that reduce corrosion of metals embedded in concrete, which reduces the deterioration of the concrete. Protective systems limit the intrusion of moisture, chloride ions, and other contaminants into the concrete by using surface treatments, applying electrical-chemical principles, or by modifying the PCC overlay. Protective systems also include materials and methods that increase surface abrasion or impact resistance, or improve the resistance to other deleterious influences.

4.1—Introduction and selection factors

Under most circumstances, concrete structures and their repairs have a finite life. The primary reason for providing a protection system is to extend the life of the structure or subsequent repairs, and to reduce the rate of deterioration of the concrete structure. A proper repair program should determine the reasons for the onset of the concrete deterioration or failures. In reality, it is often difficult, if not impossible, for a repair program to change those conditions. Thus, it is imperative that protection systems be designed to improve the performance of the repairs and the original concrete by moderating the underlying causes of concrete deterioration. This is often the only means to alleviate the affects of these conditions after the completion of a repair program.

There are a multitude of methods and products available to protect concrete. It is not sufficient to only evaluate the properties of the protective materials. To optimize the protection, a comprehensive repair and protective system should be evaluated and provided. The reader is referred to the information included in ACI 515.1R and documents from *NACE International* while at the same time concentrating on the issues related to concrete repairs.

4.1.1 *Economic factors*—Life-cycle costs should be evaluated for the various protection systems that are being considered. The protection system with the lowest initial cost may actually be the most expensive when the costs of future repairs are evaluated over the projected life of the structure.

4.1.2 *Service record*—Several new systems are available now or may soon be brought into the market that have almost no service record. Although test results might be encouraging, most engineers and owners might be reluctant to incorporate such systems on their projects. If an unproven system is used, the owner should be made fully aware of the risks, especially any limitations and potential problems (Paul 1998).

4.1.3 *Appearance*—Owners need to know how a protection system will look before installing it. Appearance could be an important factor in determining the selection of a system.

4.1.4 *Construction observation*—An appropriate inspection program should be established and implemented during the installation of the protection system. ACI 515.1R thoroughly discusses this topic.

4.1.5 *Environmental considerations*—The Environmental Protection Agency (EPA) or other regulatory agencies may have certain VOC requirements as well as others that should be considered when selecting some systems. In addition, the handling, use, and disposal of hazardous chemicals can be a factor when evaluating potential protection systems. Noise and dust levels can also be selection factors.

4.1.6 *Compatibility and bond*—The surface conditions should be reviewed when selecting a protective system. Factors that should be considered are the compatibility and the ability to bond to any existing coatings or repair materials. Consideration should also be given to the bond of any remaining existing coatings to the underlying substrate. Removal of existing coatings or sealers may influence the selection of a new system. Job-site testing, laboratory testing, or both, should be mandatory before installing a new protection system over existing coatings or sealers (ACI 515.1R).

4.1.7 *Durability and performance*—The anticipated life cycle of a system coupled with the owner's requirements should be evaluated before selecting a material. Among the factors to consider are environmental concerns, such as exposure to wind-driven rain, carbon dioxide exposure, ultraviolet exposure, temperature variations, acid rain, abrasion, or any other potentially destructive conditions.

4.1.8 *Safety requirements*—The environment where a system is installed can influence the selection of that system. Not only should performance characteristics be considered but so should safety issues. Many of the protection materials are toxic and could cause serious medical problems to some people during installation. An obvious concern is for the safety of the construction crews during the application, especially in poorly ventilated spaces. People not involved with the installation who are in direct proximity, spaces with open flames, potential for sparks from nearby electrical service, or any other potential hazards are issues that should be dealt with. The contractor should follow proper safety procedures at all times. The manufacturers should be part of the process and included when decisions are made regarding the safe use of their products. Material Safety Data Sheets (MSDS) should be furnished with all supplied materials and made available to the owner, if requested, before releasing the construction documents to bid.

4.2—Concrete surface preparation and installation requirements

Certain basic conditions should be met before selecting and installing a protection system. Surface preparation is one of the most important.

4.2.1 *Completion of concrete repairs*—Concrete repairs, before application of protective system, should be completed and cured (usually 28 days). (Note: Manufacturers of some products claim they can be installed on moist surfaces and in considerably less time than 28 days. These claims should be substantiated by test data before acceptance by the specifier or contractor.)

4.2.2 *Surface preparation*—The surface should be dry and sound and the preparation implemented in accordance with the manufacturer's recommendations and the engineer's requirements. All surface contamination, including existing coatings, that could result in poor adhesion of a coating or that may limit the penetration of sealers should be removed.

Accepted techniques include scarification, brushing or grinding, abrasive blasting, shotblasting, and flame cleaning. Dust and debris resulting from surface preparation should be removed before applying the surface treatment. Cady, Weyers, and Wilson (1984), Gaul (1984), Murray (1989), and Wyman and Gumina (1994) discuss surface preparation requirements and methods. ACI 515.1R and ICRI 03732 provide guidelines for the preparation of concrete substrates that could be used to define a standard of acceptability for some protection systems.

4.2.3 *Surface for liquid-applied membranes and other coatings*—Surfaces should be relatively smooth, usually in the range of concrete surface profile 3 to 5 (ICRI 03732), for applying most liquid-applied membranes and most other thin coatings. Most liquid-applied products are not intended to level off or hide imperfections, although trowel grade materials can sometimes be used for this purpose. This could be an issue for exposed membranes or coatings where aesthetics is a concern. Proper surface preparation, as defined by the manufacturer, should be implemented, including the repairs of shallow delaminations, surface scaling, aggregate popouts, the grinding of rough surfaces, or the treatment of any other surface defects as required to achieve the proper performance of the product.

4.2.3.1 *Cleanliness and quality of surface*—The quality of the concrete surface should be evaluated before installing the liquid-applied membrane. Not only should the surface be free of contaminants that could affect the performance of the protection system, it should have adequate strength, a pH that is compatible with the product to be installed, and the properties required for the protection system. Refer to ACI 515.1R for an overview of the tests commonly used to check the surface quality.

The membrane manufacturer should be consulted to establish criteria for acceptability of the prepared surface. To avoid any warranty issues, approval of surface preparation should be provided in writing by the manufacturer, through his or her representative, before application.

4.2.4 *Ventilation requirements*—Ventilation conditions and humidity should be considered when selecting products so that proper application and curing can occur safely and in accordance with manufacturer's requirements.

4.2.5 *Temperature limitations*—Most products have temperature installation limitations that should be considered when scheduling the work, and the products need to be installed within the manufacturer's recommended limitations.

4.3—Typical selection problems

High chloride content, misplaced reinforcing steel, poor-quality concrete, and carbonation are some of the factors that contribute to corrosion. Every attempt should be made to minimize the affect of these factors on a repair program. An appropriate protective system should be designed into the repair program. This minimizes the effect of any detrimental existing conditions and anticipated factors that could result in a failure of the repairs or contribute to other concrete deterioration. When developing the protection system for a project, all of the factors affecting the performance of the completed repairs should be evaluated. More than one potential problematic condition usually should be addressed before selecting the protection system. The following are some of the more common problems that should be considered in a repair program.

4.3.1 *Poor-quality or inadequate concrete cover*—The performance of any concrete structure is in part a function of the quality and cover of concrete in that structure. Good quality, durable, properly consolidated concrete with adequate cover placed with minimal honeycombing and cracks provides an environment that should protect the embedded reinforcing steel for years without any protection system before repairs, if ever, are required. Conversely, poor-quality concrete with deficiencies such as excessive internal cracking, internal voids, lack of consolidation, inadequate entrained air-void system, or otherwise substandard conditions, contribute to the corrosion of the reinforcing steel as well as to other forms of deterioration of the structure. It is best to remove any deficient concrete as part of a repair program, when and where possible. A properly selected protection system can improve the long-term durability of poor-quality concrete, enhance the performance of good concrete, and extend the life of any repair.

4.3.2 *Misplaced reinforcing steel*—Misplaced reinforcing steel is a major contributing factor to corrosion. This commonly occurs at reveals on walls or columns, in slabs with variable thickness at the edge bars along exposed slabs, and at hooked bars perpendicular to exposed concrete slab edges, if placed too close to the surface. The repair program should provide for the lack of cover on the reinforcement. This can be accomplished with buildouts, where appropriate. Barrier coatings on the reinforcing steel or concrete surface, probably in combination with other techniques, provides added protection against corrosion. Cathodic protection, chloride extraction, and corrosion-inhibitor additives in repair materials can also be useful to prevent or delay future corrosion.

4.3.3 *Water penetration*—Water may penetrate into concrete by hydrostatic pressure, moisture vapor pressure, capillary action, wind-driven rain, or any combination of these. Cracks, concrete density, porous concrete, lack of entrained air, structural defects, or improperly designed or functioning joints all contribute to the movement of moisture. Water penetration into concrete contributes to corrosion of reinforcement, freezing-and-thawing damage, leakage into the interior of the structure or occupied levels beneath decks, and possible structural damage. A properly designed protection system should address any or all of these issues.

4.3.4 *Carbonation*—Carbonation is the reduction of the protective alkalinity of concrete, caused by the absorption of carbon dioxide and moisture. In normal concrete, the reinforcing steel is protected by the naturally high alkalinity of the concrete around the reinforcement, usually a pH above 12. A passivating oxide layer is formed around the reinforcing steel that acts as a protective coating. The oxide coating helps prevent the reinforcing steel from corroding as long as the high alkalinity is maintained. When carbonation occurs the alkalinity falls and, once it goes below a pH of 10, the embedded reinforcing steel is subject to corrosion. Because carbonation occurs from the face of the concrete inwards, any bars close to the exterior surface are subject to the effects of carbonation and are not protected against corrosion. Barrier coatings may provide protection against future carbonation where concrete coverage is insufficient. Many of these barrier coatings are relatively new and do not have any field performance data. Any barrier coating should be carefully evaluated before use. The owner should be informed of these limitations and written consent should be obtained before use. Other systems available to reestablish the protection of the reinforcing steel in carbonated concrete include the use of a cathodic protection system or the realkalization of concrete.

4.3.5 *Anodic ring (halo effect)*—On many concrete repair projects, situations arise where existing reinforcement extends from the parent concrete into a repair mortar or new concrete. Quite frequently, failures occur due to accelerated corrosion of the reinforcement in the parent concrete, just beyond the edge of the repair. It is common to see delamination of concrete around the perimeter of new repair patches in spite of the fact that good-quality materials, workmanship, and methods were used.

This is commonly referred to as an anodic ring or halo effect. It occurs because the same bar extends into two distinctly different environments, setting up conditions that could result in an increase of the differences in electrical potential at the bond line between the new and the parent concrete. Corrosion occurs at the anode, usually in the parent concrete, as electrons are attracted to the cathodic portion of the reinforcement in the uncontaminated repair material. The build-up of rust results in spalling of concrete due to the large internal forces developed at the surface of the reinforcement. The presence of chlorides accelerates this process.

This is a difficult problem to solve. Sealers or barrier coatings help, to some extent, to slow down the process but do not stop it. Barrier coatings on the reinforcing steel, such as epoxies, latex slurries, or zinc-rich coatings, can inhibit the corrosion activity; however, there are field-application problems that can significantly reduce their effectiveness. Cathodic protection and chloride extraction are alternatives that should be considered where economically feasible. Recently, several highway departments have used galvanic anodes to reduce corrosion in and around patches and in concrete overlays (Section 4.7.4).

4.3.6 *Cracks*—All concrete repair programs and protection systems should address the proper treatment or repair of cracks. Intrusion of water into cracks can result in corrosion and freezing-and-thawing problems in cold climates.

Only after determining the reason for the occurrence of a crack can a proper repair technique be developed. Structural cracks requiring repair should be able to reestablish load transfer across the crack, usually coupled with epoxy injection to ensure a positive repair.

Active cracks, especially those due to thermal changes on exterior exposures, should be repaired to allow for future movements. Techniques involving caulking, chemical grouts, elastomeric coatings, and high elongation epoxies have proven to be useful in addressing moving cracks.

The repair of active cracks on exterior exposures can be difficult. Most of the materials used for crack repair are temperature-sensitive and cannot be installed much below 4 °C (40 °F). It is most common to repair cracks at temperatures above the manufacturer's recommended minimum. Although this facilitates the installation of the repair material, active cracks due to temperature variations tend to close in warm weather. Cracks that open in the winter are closed in the summer. Both the contractor and the engineer should be aware of this before commencing with the repair process. If feasible, an inspection of the structure should be conducted in cold weather to document the location of the cracks requiring repair (ACI 224.1R). It is also desirable to conduct repairs when the crack is near its maximum width, because most flexible materials used in repair of active cracks perform better in compression than in tension (Emmons 1994; ACI 504R).

4.3.7 *Chloride/chemical attack*—Penetration of chemical or salt solutions through concrete contribute to the corrosion of the embedded steel. In addition, chemical attack, including acids, alkalis, and sulfates, may have a detrimental effect on the concrete itself. Barrier protection systems are commonly used to minimize the intrusion of chemicals into concrete. This is thoroughly discussed in ACI 515.1R, and it provides a summary of the effects of a variety of materials on concrete. Refer also to Portland Cement Association's Publication IS001 (1997).

4.3.8 *Surface erosion*—Erosion of concrete at the surface is a major concern on dams, spillways, and other waterfront structures, as well as on bridge decks, ramps, parking decks, industrial floors, and other traffic-bearing structures. Usually to a lesser extent, it can also be a concern on buildings exposed to acid rain and severe weather conditions. Concrete overlays, surface hardeners, sealers, or other treatments are often used to protect these surfaces after the completion of any required repairs (ACI 210R and 210.1R).

4.4—Systems concept

When selecting a protection system, all factors affecting the performance of that system on the structure need to be considered in their entirety and not just the individual components. This approach is defined as a systems concept in ACI 515.1R. Some of the factors that should be considered include:

- Protective materials;
- Interface of protective materials to concrete surface;
- Concrete to a depth of 40 mm (1.5 in.) (surface concrete);
- Concrete to a depth greater then 40 mm (1.5 in.);
- Structural considerations;
- As-built construction considerations (placement of reinforcement);
- Exposure of concrete to elements;
- Stability of ground-supporting concrete (if applicable);
- Groundwater pressures (if applicable);
- Chemical exposures (if applicable); and
- Exposure to abrasion, erosion, and cavitation.

4.5—Surface treatments

Surface treatments include any material applied to the surface of concrete that is intended to provide protection. The materials and procedures discussed in this section are important components of a protection system but are not the only components that should be evaluated to achieve an adequate protection system for a repair program. All of the factors discussed in Section 4.4 should be evaluated and dealt with in a manner appropriate for the application. Most of the protection systems for repaired concrete require at least one, or in some cases several, of the surface treatment classifications discussed in Sections 4.5.2 and 4.6 to 4.9.

4.5.1 *Uses*—Surface treatments include horizontal or vertical applications. The techniques and materials selected should be consistent with the intended use. The objective is to limit corrosion by establishing conditions that reduce the existing moisture level in the concrete while preventing further moisture and chloride intrusion. Although results have varied, surface treatments have been effective in substantially slowing reinforcement corrosion in laboratory tests, and some have performed well in field applications. Quality materials and workmanship are essential.

4.5.1.1 *Precautions*—It is important to ensure compatibility between the intended treatment, repair materials, and existing concrete substrate. Compatibility of all materials in contact with each other should be evaluated to avoid bond failures or chemistry-related failures of the surface treatment materials or the underlying substrate. Appropriate surface preparation should be provided. Encapsulating concrete between nonbreathing surface treatments should be avoided. Manufacturer's recommendations and limitations should be followed.

In addition, other factors that could impact the effectiveness of the protection system should be evaluated. For example, adjacent construction not included in the scope of work for the surface treatment may be a potential source for water infiltration. This may include adjacent masonry, unrepaired or repaired concrete, window systems, intersecting walls or slabs, roofs, decks, intersecting or continuous waterstops, or any other construction that could reduce the performance of the waterproofing system.

4.5.1.2 *Properties*—The properties of surface treatments affecting their selection are discussed as follows and summarized in Table 4.1. The surface treatments are classified to facilitate this presentation, but the summary lists characteristics based on limited data from laboratory tests and field performance of specific formulations of proprietary products. A particular product should only be selected based on its field performance and results of standardized or comparative tests. Studies that directly measure performance with respect to water absorption, durability in northern and southern climates, and resistance to reinforcement corrosion are discussed later in this section. Table 4.2 gives references to ASTM standards for sealers and coatings.

4.5.1.3 *Installation requirements*—Most surface treatments should be applied to a clean, dry, and sound substrate at moderate temperature and humidity conditions in a well-ventilated space. A relatively smooth surface is needed for liquid-applied membranes. Because these conditions do not always prevail, the difficulty and cost in achieving the appropriate installation conditions may influence the choice of a system.

Before applying most surface treatments, all concrete repairs should be completed and allowed to cure. Most manufacturers require a minimum of 28 days, but this may vary. Product manufacturers should be consulted for the required curing time and application conditions for specific products.

Details of terminations at expansion and control joints, door and window openings, drains, and curbs should be reviewed and installed properly. Moisture in the slab at time of application, temperature, presence of contaminants, and other factors affect the success in applying a system.

It is imperative to ventilate any flammable and noxious fumes that may be produced. Manufacturer's specifications should include installation requirements. All governing VOC regulations for all products used on the project should be met.

4.5.1.4 *Performance characteristics*—Each protection system and the materials used in that system has its own characteristics. Although several of the available systems might provide adequate protection to the concrete and embedded reinforcing steel, there might be certain parameters for a given project that make one more attractive than another. Before evaluating the available protection systems, the specifier should determine the required system characteristics for that project. Once determined, a system should be selected that satisfies the project requirements. Below are many of the primary system characteristics that should be reviewed when selecting a protection system. Table 4.2 lists most of the ASTM tests commonly used to evaluate performance characteristics.

Table 4.1—Summary of surface treatments

Types	Generic classification(s)	Installation requirements	Durability characteristics	Performance characteristics
Sealers	Boiled linseed oil Sprayed Approximately 50 °F (10 °C) or above	Clean, dry and sound surface Poor resistance to UV radiation	Improves resistance to freezing and thawing Frequent applications required	Darkens concrete slightly Does not bridge cracks
	Alkyl-alkoxy-silane Siloxanes	Surface free of pretreatments Sprayed, brushed, or rolled Ventilation required	Improves resistance to freezing and thawing Reduces salt penetration Reduces rate of corrosion	Improved resistance to water absorption and reinforcement corrosion Does not bridge cracks
	High-molecular-weight methacrylate	Clean, dry, and sound surface Sprayed, brushed, rolled, or squeegeed	Variable UV radiation resistance Prevents moisture from penetrating cracks	Seals cracks
Coatings	Epoxy Urethane or neoprene membrane/ epoxy top coat system Rubberized asphaltic top coat system Urethane Membrane/urethane topcoat system	Clean. dry, and sound surface Sprayed, brushed, rolled or squeegeed Approximately 50 °F (10 °C) or above Ventilation required Level surface typically required	Generally improves resistance to freezing and thawing Fair to good abrasion resistance Variable UV radiation resistance	Generally good resistance to water absorption Unknown resistance to reinforcement corrosion Bridges small cracks
Overlays	Concrete Polymer concrete Polymer-modified concrete	Clean, sound, and roughened surface Hand or machine applied Generally above freezing Ventilation may be required	Improves resistance to freezing and thawing Excellent abrasion resistance	May add weight Architectural finish is possible Protects structural concrete and reinforcement May improve structural capacity

a) *Water permeability*—Resistance to water absorption is a critical factor in protective systems. Water permeability is a measurement of the quantity of water that can pass through a surface treatment over a period of time.

b) *Vapor permeability*—While surface treatments should resist water, they should allow concrete to dry, particularly if placed over new concrete repairs or on the inside face of foundation walls or on slabs-on-ground. Moisture vapor transmission is the quantity of water vapor that can pass through a protective system over a period of time.

c) *Resistance to reinforcement corrosion*—The water and vapor permeability characteristics described previously are indirect measures of the resistance of concrete to reinforcement corrosion. Penetration of chemical or salt solutions through concrete is likely to cause localized corrosion of reinforcing steel. A primary reason to provide surface treatments is to increase the resistance to reinforcement corrosion by providing a less-corrosive environment. Pfeifer and Scali (1981) and Pfeifer, Landgren, and Zoob (1987) have documented studies of different protective systems to resist corrosion.

d) *Crack bridging*—Moisture penetration through cracks can defeat the purpose of the surface treatment by allowing localized reinforcement corrosion. The protection system should include proper treatment of cracks that could potentially provide paths for the entry of contaminated water or other liquids into the concrete substrate. The surface-treatment products included in Section 4.5.2 have varying crack bridging ability. Some products have the ability to fill small dormant cracks but are not suitable to bridge moving cracks. Certain products are formulated to bridge small, moving cracks if detailed properly. The capacity of the selected products to bridge cracks should be carefully reviewed. If necessary, incorporate additional methods into the protection system to treat the cracks.

Other sections of this guide discuss crack repair techniques. The data from the studies cited above should be reviewed with caution because these studies were conducted primarily on uncracked samples that were coated before exposure simulating northern and southern climates.

e) *Elongation characteristics*—The capability of a material to elongate and bridge small cracks with changes in width of the cracks might be a criterion for selection.

f) *Skid and slip resistance*—Skid and slip resistance are measures of the frictional characteristics of the concrete surface. Penetrants do not affect the skid resistance and, because they are located at or beneath the surface, they are subject to wear to the same extent as the concrete surface. Surface sealers and high build coatings may make the surface less skid resistant, whereas membrane systems with embedded aggregate and overlays have the potential to make the surface more skid resistant. Refer to Table 4.1.

g) *Appearance*—Most surface treatments alter the appearance of the concrete. Some systems can mask the contrast between the original concrete and the repairs that can be advantageous.

h) *Carbonation resistance*—Carbonated concrete has lost its ability to protect the embedded reinforcement from corrosion.

i) *VOC characteristics*—Federal and state regulations have set allowable VOC levels.

4.5.2 *Surface treatment classifications*—The surface treatment classifications discussed in this section are those commonly used for concrete repair and protection against reinforcement corrosion. Because of the vast quantity of products or combination of products or techniques, it is impossible to cover all of them. In addition, new products and methods to protect concrete are constantly being developed and introduced into the market. Surface treatments in this section are grouped into the following general classifications: penetrating sealers, surface sealers, high-build coatings, membranes, and overlays.

The costs of the various surface treatments vary greatly. Accurate costs can only be obtained from the manufacturer and installer for a particular project.

4.5.2.1 *Penetrating sealers*—

a) *Description*—Penetrating sealers are materials that, after application, are generally within the substrate of the concrete. Depth of penetration varies by the product and with the properties of the concrete on which the sealer is applied.

Table 4.2—Testing of sealers and coatings for reinforced concrete

A. Protective properties	
1. Artificial or accelerated weathering	
ASTM D 1653	Test Method for Water Vapor Transmission of Organic Coating Films
ASTM E 96	Test Methods for Water Vapor Transmission of Materials
2. Water absorption	
ASTM C 67	Test Methods for Sampling and Testing Brick and Structural Clay Tile
ASTM C 642	Test Method for Density, Absorption, and Voids in Hardened Concrete
3. Chloride ingress	
AASHTO T 259	Resistance of Concrete to Chloride Ion Penetration
B. Durability	
1. Natural weathering	
ASTM D 4141	Practice for Conducting Black Box and Solar Concentrating Exposures
2. Artificial or accelerated weathering	
ASTM D 2565	Practice for Xenon Arc Exposure of Plastics Intended for Outdoor Applications
3. Salt spray resistance	
ASTM B 117	Practice for Operating Salt Spray (Fog) Apparatus
4. Freezing and thawing/scaling	
ASTM D 2247	Practice for Testing Water Resistance of Coatings in 100% Relative Humidity
5. Resistance to alkali	
ASTM D 1647	Test Methods for Resistance of Dried Films of Varnishes to Water and Alkali
6. Light stability	
ASTM D 333	Guide for Clear and Pigmented Lacquers
7. Chalking	
ASTM D 4214	Test Methods for Evaluating the Degree of Chalking of Exterior Paint Films
8. Water swelling	
ASTM D 2247	Practice for Testing Water Resistance of Coatings in 100% Relative Humidity
9. Resistance to wind-driven rain	
ASTM E 514	Test Method for Water Penetration and Leakage through Masonry
Fed. Spec. TT-C-555B	Coating, Textured (for Interior and Exterior Masonry Surfaces)
Fed. Spec. TT-P-1411A	Paint, Copolymer-Resins Cementitious (for Waterproofing Concrete and Masonry Walls
ASTM D 412	Test Methods for Vulcanized Rubber and Thermoplastic Elastomers—Tension
ASTM D 522	Test Methods for Mandrel Bend Test of Attached Organic Coatings
ASTM D 2370	Test Method for Tensile Properties of Organic Coatings
ASTM D 522	Test Methods for Mandrel Bend Test of Attached Organic Coatings
ASTM D 429	Test Method for Rubber Property—Adhesion to Rigid Substrates
ASTM D 2197	Test Methods for Adhesion of Organic Coatings by Scrape Adhesion
ASTM D 3359	Test Methods for Measuring Adhesion by Tape Test
ASTM D 4541	Test Method for Pull-Off Strength of Coatings Using Portable Adhesion Testers
ASTM D 2794	Test Method for Resistance of Organic Coatings to the Effects of Rapid Deformation (Impact)
10. Coefficient of friction	
ASTM D 2047	Test Method for Static Coefficient of Friction of Polish-Coated Floor Surfaces as Measured by the James Machine
B. Durability (continued)	
11. Chemical resistance	
ASTM C 267	Test Methods for Chemical Resistance of Mortars, Grouts, and Monolithic Surfacings and Polymer Concretes
12. Crack bridging	
ASTM C 836	Specification for High Solids Content, Cold Liquid-Applied Elastomeric Waterproofing Membrane for Use with Separate Wearing Course
C. Performance properties	
1. Elongation/elasticity	
D. Physical properties	
1. Viscosity	
ASTM D 2196	Test Methods for Rheological Properties of Non-Newtonian Materials by Rotational (Brookfield-Type) Viscometer
2. Hardness	
ASTM D 1474	Test Methods for Indentation Hardness of Organic Coatings
ASTM D 2134	Test Method for Determining the Hardness of Organic Coatings with a Sward-Type Hardness Rocker
ASTM D 2240	Test Method for Rubber Property—Durometer Hardness
ASTM D 3363	Test Method for Film Hardness by Pencil Test
3. Specific gravity	
4. Drying time	
ASTM D 1640	Test Methods for Drying, Curing, or Film Formation of Organic Coatings at Room Temperature
5. Nonvolatile content	
ASTM D 1353	Test Method for Nonvolatile Matter in Volatile Solvents for Use in Paint, Varnish, Lacquer, and Related Products
6. Flash point	
ASTM D 56	Test Method for Flash Point by Tag Closed Cup Tester
ASTM D 93	Test Methods for Flash-Point by Pensky-Martens Closed Cup Tester
ASTM D 3278	Test Methods for Flash Point of Liquids by Small Scale Closed-Cup Apparatus
7. Flame spread, smoke density	
ASTM E 84	Test Method for Surface Burning Characteristics of Building Materials
8. Gloss	
ASTM D 523	Test Method for Specular Gloss
9. Coating thickness	
ASTM D 4138	Test Method for Measurement of Dry Film Thickness of Protective Coating Systems by Destructive Means
10. Cure time	
11. Effect on potable water	
NSF Standard 61	Drinking Water Components—Health Effects
12. Microbiological attack	
ASTM D 3274	Test Method of Evaluating Degree of Surface Disfigurement of Paint Films by Microbial (Fungal or Algal) Growth or Soil and Dirt Accumulation
ASTM D 3456	Practice for Determining by Exterior Exposure Tests Susceptibility of Paint Films to Microbial Attack

Depth of penetration is determined by the size of the sealer molecule and the size of the pore structure in the concrete. While deep penetration may be desirable, especially for surfaces subjected to abrasion, it is not the most important criterion for sealer effectiveness (ACI 515.1R).

These sealers are formulated to provide protection as water repellents or as surface hardeners. Penetrants do not have crack-bridging capabilities. The hydrophobic properties imparted by some of these products may reduce the intrusion of moisture into narrow cracks. Products in this classification generally do not alter the appearance of the concrete surface, although some might change the color slightly. Repairs, cracks, and any other flaws on the surface will not be masked by penetrants.

Such sealers include, but are not limited to, boiled linseed oil, silanes, siloxanes, certain epoxies, magnesium and zinc fluorosilicates, and high molecular-weight methacrylates. Epoxies that are formulated for penetration penetrate only slightly into concrete, if at all.

b) *Applications and limitations*—Penetrating sealers may be applied by roller, squeegee, or spray to the concrete substrate. Proper surface preparation is important for successful application due to the sensitivity of penetrating sealers to contaminants and previously applied sealers in the substrate. UV, wear, and abrasion resistance are generally good when compared with coatings or membrane systems (Carter 1993).

Penetrating sealers do not bridge new or existing cracks. Some of these products are solvent-based and others are water-based. As such, the solvent-based products may have problems meeting some VOC regulations.

4.5.2.2 *Surface-applied corrosion inhibitors*—The use of topically applied corrosion inhibitors in concrete repair is recent, having been first introduced in the early 1990s. By definition, corrosion inhibitors reduce the rate of corrosion. The effectiveness and life expectancy of these materials is unknown and likely varies considerably with different types of inhibitors, the properties of the concrete, and site conditions. Users report a wide variation in their effectiveness, and this presents the question as to what parameters relate to successful use for any given formulation. Studies of their effectiveness (Berke et al. 1992; Jones, Lambert, and Wood 2000; Brown, Bryant, and Wyers 1999; Elsner 2000; Elsner et al. 1999; Mietz, Elsner, and Polder 1997; Elsner, Buchler, and Bohni 1997; Transportation Research Board 2001) have extended the question to repair materials and applications. Manufacturers should include in their literature the corrosion inhibitor type, recommended uses, application rates, constraints, anticipated effectiveness, and monitoring method.

4.5.2.3 *Surface sealers*—

a) *Description*—Surface sealers are products of 0.25 mm (10 mils) or less in thickness that generally lay on and adhere to the surface of concrete. The dry film thickness of surface sealers and paints range from 0.03 to 0.25 mm (1 to 10 mils). Although these products may be pigmented or naturally colored, some are transparent, which results in a wet or glossy appearance. They may or may not appreciably alter the texture of the surface, and most surface blemishes usually reflect through. Surface sealers do not have significant crack-bridging capabilities, although the hydrophobic nature of some of these products may reduce the intrusion of moisture into narrow cracks. In addition, some of these products may fill dormant cracks, reducing the penetration of moisture through those cracks.

Such products include epoxies, polyurethanes, HMWM, siloxanes, silanes, moisture-cured urethanes, and acrylic resins. Certain paints, whether oil-based or latex-based (such as styrene-butadiene, polyvinyl acetate, acrylic or blends of these with other polymers dispersed in water), would be included in this classification if less than 0.25 mm (10 mils) thick (Portland Cement Association 1995).

b) *Uses*—The products in this classification have the capability to reduce the intrusion of water, chlorides, and, in some cases, mild chemicals. They also may or may not permit the transmission of water vapor.

c) *Application and limitations*—The materials may be applied with brush, roller, squeegee, or spray. The chemical carriers within some of the products may cause application limitations. The manufacturer's safety recommendations should be followed. Surface sealers, in general, reduce skid resistance. They do not bridge moving cracks, but may be effective in filling (not bridging) small, nonmoving cracks. Many of these products are affected by UV and wear under surface abrasion.

Some of these products depend on solvents to work and may have problems meeting VOC regulations. Newer formulations that meet VOC regulations do not have many years of in-service usage.

4.5.2.4 *High-build coatings*—

a) *Description*—High-build coatings are materials with a dry thickness greater than 0.25 mm (10 mils) and less than 0.75 mm (30 mils) applied to the surface of the concrete. High-build coatings alter the appearance of the surface, may be pigmented, and partially mask blemishes in the concrete surface.

The base polymers of such products include, but are not limited to, acrylics, alkyds, styrene butadiene copolymers, vinyl esters, chlorinated rubbers, urethanes, silicones, polyesters, polyurethanes, polyurea, and epoxies.

b) *Uses*—These products are generally used for decorative or protective barrier systems. Some products in this classification may be suitable for use against wind-driven rain, salts, and mild chemicals.

c) *Applications and limitations*—These products may be applied with brush, roller, squeegee, or spray. For exterior environments, the coating should be resistant to oxidation and UV and infrared radiation exposure. On floors, resistance to abrasion and punctures and resistance to mild chemicals (salts, grease and oil, and detergents) are also important.

In addition to durability of the coating material, the bond between the coating and concrete substrate should remain intact. Even with proper surface preparation, the bond can break down. If the coating is highly impermeable to water vapor, water may condense at the concrete-coating interface and destroy the bond.

Epoxy resins are commonly used repair materials that generally have good bonding and durability characteristics. The resins can be mixed with fine aggregates to improve abrasion and skid resistance. When used in horizontal applications, some high-build coatings, including epoxies without aggregate, may result in a very slippery surface when wet and may not be suitable for pedestrian or vehicular traffic. Nonelastomeric high-build coatings generally do not bridge moving cracks, but may be effective in filling small, nonmoving cracks. These products have better wear characteristics than thinner systems.

A coating intended to reduce reinforcement corrosion in repair work may also be required to waterproof the structure, protect against chemical attack, or improve the appearance.

Breathability is often an important factor when selecting a protection material on exterior walls and slabs-on-ground. There are high-build coatings formulated for use in decorative systems that have high perm ratings while at the same time provide sufficient protection for moisture protection.

Some of these products are solvent-based. As such, they may have problems meeting VOC regulations, especially in their earlier formulations. Newer formulations of these products do not have many years of in-service usage.

4.5.2.5 *Membranes—*

a) *Description—*Membrane systems are surface treatments with thicknesses greater than 0.7 mm (30 mils) and less than 6 mm (250 mils) applied to the surface of the concrete. They may be bonded, partially bonded, or unbonded to the concrete surface. These products significantly alter the appearance and mask blemishes in the concrete surface. Elastomeric membranes are included in this group as well as some high-build coatings.

Elastomeric membrane systems generally have sufficient thickness and flexibility to bridge narrow, nonmoving cracks of various widths, even if the crack width fluctuates. Some systems require that cracks wider than 0.25 to 0.375 mm (10 to 15 mils) should be routed and sealed before application of the membrane. Elastomeric membranes are usually gray or black, but some manufacturers offer several other colors, which may have a tendency to fade, especially when exposed to UV. Refer to ACI 362.1R, 362.2R, and 350.1/350.1R.

Such products include, but are not limited to, urethanes, acrylics, epoxies, neoprenes, cement, polymer concrete, and asphaltic products.

b) *Uses—*The products in this classification are generally used as protective, waterproofing, or traffic or pedestrian systems. Some formulations are appropriate for use in damp-proofing systems.

c) *Application and limitations—*When in liquid form, these products may be applied by brush, squeegee, roller, trowel, or spray. Preformed sheets are sealed at the edges to form a continuous waterproofing membrane. Most of these membranes are resistant to water absorption and bridge small (less than 0.25 mm [0.01 in.]) moving or nonmoving cracks. Membranes with a rigid urethane mortar or epoxy-mortar top coat offer reasonable skid and abrasion resistance under traffic. Frequent maintenance of exposed membranes in parking structures may be required at steep ramps and at turning, starting, and stopping areas. The gritty surfaces of some membrane top coats provided for abrasion and skid resistance are difficult to keep clean and may wear off.

Tests that measure performance characteristics of elastomeric membranes, particularly traffic deck membrane systems, are quite limited. Manufacturers usually list a variety of different nonstandardized test results in their technical data sheets, which are conducted under their own laboratory conditions.

Standardized test results from independent laboratories for traffic-bearing membrane systems should be compared for key performance characteristics such as: permeability, elongation, tensile strength, tear strength, adhesion, modulus of elasticity, abrasion resistance, low temperature flexibility, and water-vapor transmission.

4.5.2.6 *Overlays: General—*

a) *Description—*Overlays are products of 6 mm (250 mils) or greater in thickness that can be bonded, partially bonded, or unbonded to the surface of the concrete. Such products include, but are not limited to, polymer concrete, portland-cement concrete, epoxies, polyesters, and polymer-modified concrete. Overlays change the appearance, texture, and elevation of the original concrete surface. The thickness of an overlay can be used to improve the drainage characteristics of the top surface of a concrete slab. Overlays may bridge nonmoving cracks; however, moving cracks may mirror through the overlay unless properly detailed. An overlay offers a choice of different colors and finishes.

b) *Uses—*Overlays are used as a wearing course and generally provide protection against the intrusion of water and chloride ions. Overlays also provide abrasion resistance and may be decorative (ACI 222R, 345R, 546.1R, 548.4, and 548.5R).

c) *Application and limitations—*Overlays may be placed, troweled, screeded, sprayed, or seeded in one or more layers onto the concrete surface. The overlay adds mass proportional to its thickness. Therefore, the additional dead load should be considered in the analysis of an existing structure. Overlays can be installed to act compositely with the existing structure. Additional reinforcement may be added, such as welded-wire fabric, and reinforcing steel or fibers.

For an overlay to perform properly, the surface to which it is bonded should be clean, sound, and appropriately roughened. Laitance, dust, and debris that result from the surface preparation operations should be removed.

Cracking has been a problem in some bonded overlays. Existing cracks can reflect through thin overlays; therefore, bonded overlays should not be used in situations in which there is active cracking or structural movement because the existing cracks reflect through the overlay. Partially bonded or unbonded overlays should be used in these situations.

Slabs-on-ground in freezing climates should never receive an overlay or coating that is a vapor barrier. An impervious barrier causes moisture passing from the subgrade or backfill to accumulate under or behind the barrier, leading to critical saturation and rapid deterioration of the concrete by cycles of freezing and thawing.

Thin, cementitious overlays are susceptible to plastic-shrinkage cracking because their high surface-to-volume ratio promotes rapid evaporation under drying conditions, such as low relative humidity and wind. Also, because these concretes usually have low *w/c*, there is little bleed water to replace evaporated water. Environmental conditions to be expected when placing overlays should be determined, and early morning or nighttime placements should be considered to avoid drying conditions.

Cracking in bonded overlays has at least one of four causes: tearing the surface due to late finishing operations; plastic shrinkage from excessive drying; differential movement between the deck and the overlay due to temperature differences or drying shrinkage; or existing cracks reflected through the overlay. Refer to ACI 224.1R for information on cracking.

Testing of the in-place concrete should be done before and after surface preparation to determine the modulus of rupture of the concrete and after the overlay has been completed to determine bond strength. The direct tensile test as described in ACI 503R, Appendix A should be specified.

4.5.2.6.1 *Common overlay systems—*

4.5.2.6.1.1 *Unbonded or partially bonded—* Unbonded or partially bonded overlays are used to provide wearing courses and for creating slopes for drainage over waterproofing membranes. They also can provide architectural treatments for exposed decks or driveways. Asphalt, asphalt concrete, or concrete are generally used for this purpose (ACI 546.1R).

Asphalt and asphaltic concrete overlays are porous, and by themselves, do not provide waterproofing. The absence of an effective waterproofing system below an asphaltic or concrete unbonded system can actually promote deck deterioration due to freezing and thawing.

These systems are used on bridge decks, plaza decks, parking garages, over office spaces, and over waterproofing.

4.5.2.6.1.2 *Bonded PCC overlay*—Bonded PCC overlays are simply layers of usually horizontal concrete placed on a properly prepared existing concrete surface to restore a spalled or disintegrated surface or to provide a protective barrier. Portland cement overlays are sometimes fortified with fibers and other additives (Sections 3.2.1 and 3.2.5). Low-slump concrete overlays are included in this group (Section 3.2.7). Bonded portland-cement-concrete overlays provide a protective barrier to deicing salts and sometimes increase the load-carrying capacity of the underlying concrete. The thickness of an overlay may range from 40 mm (1.5 in.) to almost any reasonable thickness, depending on the purpose it is intended to serve.

A portland-cement-based overlay may be suitable for a wide variety of applications, such as resurfacing spalled or cracked concrete surfaces on bridge decks or in parking structures, increasing cover over reinforcing steel, adding slip resistance, or leveling floors. Other applications of overlays include repair of concrete surfaces damaged by abrasion, freezing, or fire, and the repair of deteriorated pavements.

Low-slump concrete overlays are PCC overlays that have a modified mixture proportion to produce a denser, more durable concrete. They have good bond characteristics to a properly prepared substrate and increased durability because of lower *w/c*. These systems are less expensive than modified concrete systems.

The performance of low-slump concrete depends on the workmanship and use of conventional materials; generally the performance has been good. Local bond failures have been reported, but these have usually been attributed to inadequate surface preparation or construction deficiencies (refer to ACI 222R, 546.1R, and PCA IS144 Portland Cement Association 1996).

Low-slump concrete overlays and other portland-cement overlays are susceptible to cracking, especially on continuous span structures. Portland-cement-based overlays should not be used in any application in which the original damage was caused by chemical attack that would continue to attack the portland cement in the overlay (ACI 515.1R).

Portland cement overlays are used as a wearing course on parking garage and plaza decks to establish proper drainage in damp-proofing or decorative systems. They may also be used in conjunction with traffic or pedestrian elastomeric membrane systems.

4.5.2.6.1.3 *Latex-modified concrete (LMC) overlays—* LMC is similar to conventional concrete except that it contains a latex and less water. The rate of 15% latex solids to portland cement by mass has been most commonly used in bridge and parking deck overlays. The water in the latex constitutes part of the water required to hydrate the cement, and the latex provides supplementary binding properties to produce a concrete with a low *w/c*, good durability, good bonding characteristics, and a high degree of resistance to penetration by chloride ions.

There have been problems associated with finishing latex concrete overlays. Hot weather and wind cause rapid drying, which makes finishing difficult and promotes shrinkage cracking. Mixing, placing, and finishing should be completed within 30 min or less to accommodate the coalescing of the latex. Failure to do so may lead to tearing of the latex film that is forming within the concrete. Mixing the LMC should be done in a mobile mixer. Concrete mixed in a concrete truck or drum-type mixer should be limited to 3 min of mixing after the latex has been added. Longer periods of mixing entrap air and subsequently increase the total air content. The result can be significant reductions in compressive strength and abrasion resistance. Placement at night can help minimize these problems. Another problem is if the application of grooves (texturing) is applied too soon, the grooves collapse. If texturing is delayed until after the latex film forms, the surface tears and cracking often occurs. Refer to ACI 548.3R, ACI 548.4, ACI 222R, and Section 3.3.1.

LMC overlays are generally used as wearing and protective barriers on bridge and parking garage decks and in damp-proofing or decorative systems on plaza decks and parking garages.

4.5.2.6.1.4 *Polymer concrete overlays*—Polymer concrete is a class of composite materials that includes a wide group of mortar and concrete. The polymer families most commonly used in polymer concrete overlays are epoxies, polyesters, and vinyl esters. The compositions of each of these polymer binders are different. While more than one may be used for most applications, some are more suitable for specific situations. Refer to ACI 548.5R, and Section 3.3.2.

Overlays of polymer concrete are typically bonded to a prepared substrate and provide a durable and wear-resistant surface. They may be formulated to provide low permeability to water, chemicals, and chloride ions. This helps prevent deterioration of the concrete due to freezing-and-thawing cycles and chemical attack. These materials have good bonding characteristics to properly prepared substrates. The bond in tension and shear of the product selected should be at least equal to the tensile and shear strength of the substrate. The surface texture can be made to provide skid-resistant and hydroplaning-resistant surface characteristics that are within acceptable limits (ACI 548.1R and 548.5R).

An important advantage of polymer concrete as compared to other overlay systems is its rapid cure characteristics. This can minimize disruptions and traffic control costs and improve the scheduling of repairs. In addition, polymer concrete overlays can be installed without expensive equipment.

A disadvantage is that they should be applied to dry surfaces. The workability and curing rate are dependent on application temperatures (ACI 548.1R and ACI 548.5R).

Polymer-concrete overlays are generally used as a wearing course in dampproofing or decorative systems.

4.5.2.6.1.5 *Silica-fume overlays*—Silica-fume concrete is a normal portland-cement concrete that has been modified with a silica fume. Silica-fume concrete is used as an alternative to LMC or low-slump concrete. Silica-fume concrete overlays retard the intrusion of chlorides by providing a denser, less-permeable matrix. In addition, these overlays are more abrasion-resistant, have increased early and ultimate strength and improved bond to the substrate (Section 3.2.13).

Silica-fume overlays are used as a wearing course on bridge and parking garage decks and as a protective barrier to structural concrete in aggressive chemical environments

Silica-fume concrete is susceptible to cracking from drying and plastic shrinkage. If the silica fume is added to the concrete mixture at the end of the batch cycle, it may act as a retarding admixture. The addition of silica fume to the mixture makes it sticky, and thus more difficult to finish. Silica-fume concrete overlays are subject to reflective cracking from the substrate.

4.6—Joint sealants

Joint sealants in concrete minimize the intrusion of liquids, solids, or gases, and protect the concrete against damage. In certain applications, secondary functions improve thermal and acoustical insulation, dampen vibration, or prevent unwanted matter collecting in crevices (ACI 504R).

Protection systems of joints include the sealing of cracks, contraction (control) joints, expansion joints, and construction joints.

4.6.1 *Types of joints*—

a) *Cracks*—The reasons that cracks occur in concrete include shrinkage, thermal changes, structural-related stresses, and long-term strain shortening. Before selecting a sealant, the reason for the cracking should be determined. Moving cracks should be identified. In some instances, structural bonding of a crack may be required, whereas in other situations, restraint across the crack should be avoided.

b) *Contraction (control) joints*—Contraction joints are purposely installed joints designed to regulate cracking that might otherwise occur due to the contraction of concrete (ACI 224.3R). Often called control joints, they are intended to control crack locations. The necessary plane of weakness may be formed by reducing the concrete cross section by tooling or saw cutting a joint, usually within 24 h.

c) *Expansion (isolation) joints*—Expansion joints prevent crushing and distortion (including displacement, buckling, and warping) of abutting concrete structural units that might otherwise occur. Such crushing and distortion can be due to the transmission of compressive forces that may be developed by expansion, applied loads, or differential movements arising from the configuration of the structure or its settlement (ACI 504R). Expansion joints are made by providing a space over the entire cross section between abutting structural units.

d) *Construction joints*—Construction joints are made before and after interruptions in the placement of concrete or through the positioning of precast units. Locations are usually predetermined so as to limit the work that can be done at one time to a convenient size, with least impairment of the finished structure, though they may also be necessitated by unforeseen interruptions in concreting operations. Depending on the structural design, they may be required to function later as expansion or contraction joints, or they may be required to be soundly bonded together so as to maintain complete structural integrity. Construction joints may run horizontally or vertically depending on the placing sequence prescribed by the design of the structure (ACI 504R).

4.6.2 *Sealing methods*—Methods to seal joints include injection techniques, routing and caulking, bonding, installing premolded seals, or installing appropriate surface protection systems (such as elastomeric membranes) discussed in Section 4.5.2.4.

ACI 504R discusses different techniques and materials to seal joints, and ACI 503.4 discusses epoxy materials.

4.7—Cathodic protection

4.7.1 *Description*—Reinforcing steel in concrete is generally protected from corrosion by a passive oxide film created by the alkaline portland cement. When aggressive ions, such as chlorides, contaminate the concrete around the reinforcing steel, however, the passive oxide film is weakened or destroyed and corrosion of the reinforcing steel can occur.

Corrosion is an electrochemical process where anodic and cathodic areas are formed on the steel. When the anodic and cathodic areas are electrically continuous and in the same electrolyte, corrosion at the anodic areas occurs. The corrosion is created as an electrical current flow occurs through the corrosion cell, anodes, cathode, and electrolyte. Unless mitigated, the corrosion continues until failure occurs at the anodic area. ACI 222R contains additional information on corrosion of steel in concrete.

Cathodic protection is a proven procedure to control the corrosion of steel in contaminated concrete. The basic principle involved in cathodic protection is to make the embedded reinforcing steel cathodic, thereby preventing further corrosion of the steel. This can be accomplished by electrically connecting the reinforcing steel to another metal that becomes the anode with or without the application of an external power supply.

Cathodic protection systems in which an external power supply is not used are referred to as sacrificial passive systems. The metal that is used to protect the steel is "less noble" or more prone to corrosion than the steel—for example, zinc. As such, this metal corrodes in place of the steel and the structure is protected. Another type of cathodic system incorporates an external power supply to force a small amount of electric current through the reinforcing steel to counteract the flow of current caused by the corrosion process. A metal that corrodes at a very slow rate, such as platinum, typically serves as an anode. This method of controlling corrosion is known as impressed-current-cathodic protection. Refer to ACI 222.2R for additional information on this subject.

4.7.2 *Applications*—Cathodic protection can be used to protect almost any type of reinforced concrete structure, including:

a) *Horizontal slabs*—Parking garage decks, bridge decks, floors, roofs, and balconies.

b) *Vertical members*—Wall sections, towers, and similar structures.

c) *Structural members*—Beams, columns, foundations, and bridge substructures.

4.7.3 *Limitation*—Cathodic protection systems can mitigate corrosion on many types of structures; however, the following should be observed:

- Cathodic protection does not replace corroded steel.
- At the present time, cathodic protection using impressed-current systems is not recommended for general usage on prestressed concrete structures. Hydrogen embrittlement of the high-strength steels may occur.
- Post-tensioned structures may be cathodically protected using passive sacrificial systems after an analysis by a corrosion engineer.
- The reinforcing steel needs to be electrically continuous for cathodic protection to function. Electrical continuity of the reinforcing steel should be confirmed during system installation. In structures where the reinforcement is covered with an inorganic coating such as epoxy, electrical continuity of the reinforcement is likely to be problematic and should be confirmed before considering the use of cathodic protection.

The operation of a cathodic protection system requires the concrete to work as an ionically conductive media. As such, the concrete must have suitable resistivity and moisture content to be sufficiently conductive.

4.7.4 *Types of sacrificial passive cathodic systems*—

a) *Zinc hydrogel anode*—Zinc sheet anodes, precoated with a conductive hydrogel adhesive, are applied to the surface of the concrete. Various paints or coatings can be applied over the anode for aesthetics.

b) *Sprayed zinc or zinc alloys*—Sprayed zinc or zinc alloys are applied to the concrete using metallizing equipment such as arc spray or flame spray. Generally, enclosure during spraying is desired for toxicity and environmental concerns.

c) *Embedded galvanic anodes*—Embedded galvanic anodes provide localized galvanic protection. They are attached to the reinforcing steel and are embedded within the repair concrete. The anodes are primarily installed along the perimeter of concrete patch repairs to protect adjacent areas from corrosion due to the anodic-ring effect. The anodes can also be installed on a grid pattern to provide more overall protection. This method is new and does not have a long track record. Although initial results are encouraging, more service data need to be documented to determine the effectiveness of this method (Whitmore 2004).

4.7.5 *Types of impressed current cathodic protection systems*—All impressed current cathodic protection systems contain an anode, a DC power source, and connecting cables. In addition to the basic components, there may be reference electrodes or other monitoring and measuring devices. The primary difference between the types of systems is the anode installation and its application (Whitmore 2002).

a) *Surface-mounted anodes without overlays*—These anodes are mounted on the surface of the concrete and do not require a cementitious overlay. These anode systems are not usually used in high-wear applications.

b) *Conductive mastic anodes*—These consist of a conductive coating with embedded anodes on the surface of the concrete. They are used on vertical surfaces, ceilings, and columns.

c) *Plate-type anodes*—These consist of manufactured anode plates glued to the concrete surface.

d) *Surface-mounted anodes with overlays*—These anodes are generally used on horizontal surfaces and require a cementitious overlay of 13 mm (0.5 in.) minimum thickness.

e) *Mesh-type noble metal anodes*—A mesh of a noble metal anode is fixed to concrete with a multiplicity of pins and then covered with a cementitious material.

f) *Conductive polymer concrete strips*—A series of conductive polymer concrete strips containing a noble metal anode is fixed to the concrete surface and covered with a cementitious overlay.

g) *Embedded anodes*—The anode system is embedded in the surface of the concrete or at the level of the reinforcement in new construction.

h) *Saw slot anodes*—A series of small depth and width saw slots is made in the concrete surface. The slots are filled with a noble metal anode and a conductive polymer concrete.

i) *New construction*—Anodes can be placed at the level of the reinforcement during new construction; however, care should be taken to prevent contact between the anode and the reinforcing steel.

4.8—Chloride extraction

Chloride extraction can be accomplished by an electrochemical chloride extraction process.

4.8.1 *Description*—Electrochemical chloride extraction (ECE) is a treatment process where chloride ions are removed from chloride-contaminated concrete through ion migration. At the termination of the treatment process, the entire system is removed from the structure. Unlike a cathodic protection system, there are no permanent systems or components to be maintained or operated for the life of the structure.

The benefits of ECE include reduced corrosion activity and extended service life. ECE can be applied to both vertical and horizontal concrete surfaces. Minimal disruption of traffic on bridge or parking decks occurs if a temporary traffic-bearing surface is incorporated. Refer to ACI 222.2R for additional information on this subject.

4.8.2 *ECE application*—To remove chlorides from a reinforced concrete structure, an anode embedded in an electrolyte media is applied to the surface of the concrete. The anode and reinforcing steel in the concrete are connected to the two terminals of a DC power supply such that the anode is positively charged and the reinforcing bar is negative. Chloride ions, being negatively charged, migrate toward the anode, which is the positive electrode. Because this is external to the concrete, the chloride ions leave the concrete and accumulate in the electrolyte media around the anode. Thus, the chloride content of the concrete is reduced, particularly on and around the negatively charged reinforcing steel. At the end of the treatment period, the anode, and the electrolyte media that contains the chlorides, are removed from the surface of the concrete. While the chlorides are being removed, the electrolytic production of hydroxyl ions at the reinforcing steel surface results in a high pH around the steel. Therefore, when the process is terminated and the installation is removed, the reinforcing steel is situated in a chloride-free, highly alkaline concrete environment. The result is a strong repassivation of the embedded reinforcing steel. Corrosion on the reinforcing steel should cease.

ECE generally takes between 3 to 8 weeks to complete. The treatment duration is affected by a number of factors, including treatment density, quantity and distribution of chloride ions, reinforcing steel quantity, and concrete properties such as permeability and electrical resistivity. All required structural repairs need to be implemented either before or after the completion of the process (SHRP 1999).

4.8.3 *Limitations*—ECE can be used to mitigate corrosion on many types of structures; however, the following should be observed:

- ECE does not replace corroded steel;
- ECE is not recommended for usage on prestressed or post-tensioned concrete elements. Hydrogen embrittlement of the high-strength steels may occur;
- The reinforcing steel should be electrically continuous for ECE to function;
- The choice of a suitable power is important; and
- The effect of the removal may be different in areas close to reinforcing steel and between it.

4.9—Realkalization

Realkalization is a process used to increase the pH of carbonated concrete.

4.9.1 *Description*—Realkalization is an electrochemical treatment process whereby the pH of carbonated concrete is increased. At the termination of the treatment process the entire system is removed from the structure.

The benefits of realkalization include reduced corrosion activity, extended service life, and elimination of the need to remove and replace carbonated concrete. Realkalization can be applied to both vertical and horizontal concrete surfaces. Refer to ACI 222.2R for additional information on this subject.

4.9.2 *Application*—The realkalization process is essentially similar to ECE with two differences:

- The treatment time for realkalization is much shorter than ECE, typically 3 to 10 days; and
- Realkalization is completed using an alkaline electrolyte solution.

4.9.3 *Limitations*—Limitations for realkalization are similar to those for ECE.

CHAPTER 5—STRENGTHENING TECHNIQUES

5.1—General

Before the repair of structural members, it should be determined whether a structural analysis should be performed to determine if the members are overloaded or underdesigned for the service loads. The analysis should consider both serviceability and strength and should include consideration of the causes of the structural failure or degradation. Based on previous evaluations and analytical results, the engineer should decide whether repair only or repair plus strengthening is required.

Sections 5.2 through 5.7 outline several acceptable alternatives for strengthening structural members without completely replacing them. The sections concentrate on construction methods used for strengthening, such as placement of reinforcement within existing concrete or placement of new reinforcement exterior to the existing member. In all cases, the goal is to provide new reinforcement to resist tension caused by flexure, shear, torsion, and axial forces so that the strengthened structure meets the minimum requirements for strength and serviceability required by ACI 318 and other applicable building codes. Additional information can be found in ACI 224.1R.

5.2—Internal structural repair (restoration to original member strength)

5.2.1 *Description (repair of cracks)*—For cases requiring repair without increasing the member's structural strength capacity, epoxy injection is commonly used to restore the member. Epoxy injected into the crack restores the concrete section to its precrack condition. The epoxy tensile bond to the concrete substrate is stronger than the concrete's tensile strength, and subsequent loads applied to the concrete section consequently fail at the same load as that of the original uncracked member section. Therefore, resin injection by itself is not a strengthening method but is considered a restoration method that restores the section to its original member strength. Refer to ACI 503R and ICRI 03734.

Strengthening is provided by installing additional reinforcement across the failure plane in combination with the resin injection. Frequently, internal or external reinforcement is installed in combination with the epoxy injection for strengthening and restoration.

5.2.2 *Advantages and typical use*—Crack injection can be successfully performed on cracks as narrow as 0.013 mm (0.005 in.) in width with generally accepted epoxy injection resins. Cracks with less width can be injected with epoxy or other polymer systems having a low viscosity of 200 cps (ACI 503R and 224.1R).

5.2.3 *Disadvantages*—Special considerations should be made for bond strength for elevated temperatures. Epoxies and other resins lose strength when exposed to fire or

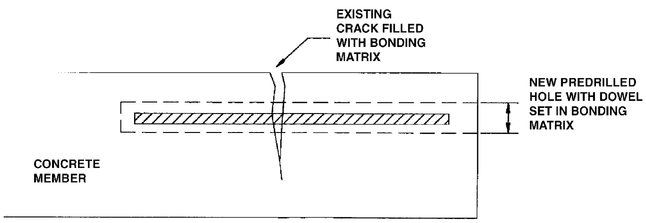

Fig. 5.1—Interior reinforcement provides increased tensile resistance to an existing crack when bonded in a predrilled hole or groove cut perpendicular to the crack.

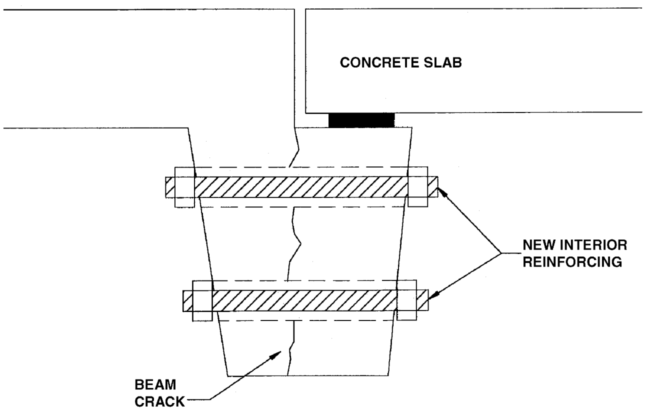

Fig. 5.2—Increasing flexural resistance with the addition of interior reinforcement bonded into predrilled hole.

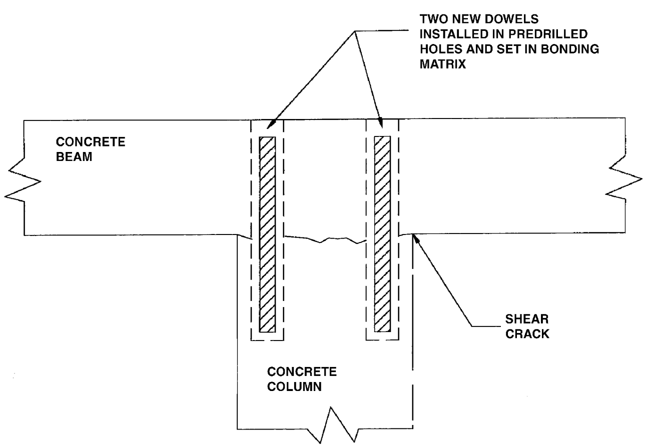

Fig. 5.3—Interior reinforcement for shear strengthening.

sustained elevated temperatures. Fireproofing protection is required for structural repairs using epoxies.

Epoxies that are labeled water-insensitive during curing may develop milky white bond lines if injected into wet or damp cracks. Verification that the epoxy is completely water-insensitive should be made by injecting the test epoxy into prewetted cracks, then evaluating the cored sample of the cured epoxy injected into the crack.

5.2.4 *Repair example*—Shrinkage cracking in slabs or walls that are restrained at support ends may develop into full-depth slab cracks. The cracking may develop due to the combination of volume changes during concrete curing or during extreme thermal fluctuations. Slab or wall cracking typically is caused by end restraints that do not allow slab or wall movements. The cause for the local overstress should be either eliminated or considered in the repair.

Concrete temperature should be considered in this repair. Variations in service thermal exposures can result in stress fluctuations in the epoxy bond line. Injecting the crack at the midrange temperature minimizes thermal stress fluctuations. Cyclic temperature changes causes internal stresses in the epoxy. The epoxy-cured properties such as creep resistance and modulus of elasticity should be considered.

5.3—Interior reinforcement

5.3.1 *Description (repair of cracks)*—A common method of providing additional reinforcement across cracked surfaces is to install new dowels into holes drilled perpendicular to the crack surfaces. The entire length of the dowel is fixed to the concrete by the use of a bonding matrix. Figure 5.1 to 5.4 show this repair method.

The structure should be shored and jacked if it is desired to relieve the member's dead load stresses so the new reinforcement resists the original dead load. At yield stress, however, the added reinforcement would normally be effective in resisting all loads.

Several bonding materials may be used. Portland-cement grouts, epoxy, epoxy mortar, latex-modified cement slurry, and other chemical adhesives have been successfully installed within the annular space between the dowel and sides of the predrilled hole. Creep, shear stress, tensile-bond strength, and other long-term changes of such resins and grouts should be considered when selecting the materials. In addition, properties such as heat generation and shear strength should be considered when sizing the hole diameter for dowels or anchors.

The dowels may be deformed reinforcement, smooth, or threaded steel or stainless steel bars, carbon fiber reinforcement, or bolts. Coating steel dowels with either zinc galvanizing or fused epoxy is acceptable if all components are chemically compatible with the bonding material. The protective coating of dowels should be considered when evaluating the bond strength between concrete and dowel.

Dowels for providing shear transfer between adjoining sections of moving pavement may be placed in slots cut from the top to mid-depth of the adjoining sections. In one section, the dowel is bonded; in the other, the dowel is unbonded using a sleeve or debonding agent.

5.3.2 *Advantages and typical uses*—Internal reinforcement can strengthen concrete cracked by flexural and shear stresses and restrained volume changes. The repair procedure is simple and uses commonly available equipment.

5.3.3 *Limitations*—Cutting or damaging existing embedded reinforcing bars or conduits during the drilling operation should be avoided. Nondestructive testing and design drawings can be used to determine the locations of embedded items. Heavily reinforced structural members may not permit drilling, and these members should be strengthened by external techniques.

Space constraints from the outside of the member may not permit drilling holes transverse to the crack.

Internal dowels in deteriorated concrete should not be installed if the bond strength cannot be developed. The concrete strength should be evaluated for each installation.

The drill holes of concrete should be cleaned of dust before the installation of reinforcement and bonding agent. If the hole is not thoroughly cleaned, the bonding matrix adheres to dust and limited bond strength to the concrete results.

5.3.4 *Installation details*—The predrilled holes should be drilled perpendicular to the crack or nearly so. A core bit or a carbide-tipped drill bit may be used, but core bits create a smooth surface inside the hole. The smooth surface is less effective for bonding to the concrete. The embedment length on both sides of the crack should be sufficient to develop the required stress in the bar by bond strength. For epoxy bond, 10 to 15 times the reinforcing bar diameter is usually sufficient, but development lengths should be based on calculated design loads and bond stresses or on tests. For cement mortars, the development lengths in ACI 318 are required. The final hole diameter should be from 3 to 6 mm (1/8 to 1/4 in.) larger than the dowel diameter if epoxy bonding agents are used. The hole diameter should be at least 50 mm (2 in.) larger than the bar for cement mortars. This provides an annular space that is adequate to allow for consolidation of the mortar. Epoxy viscosity, however, is critical in selecting hole diameter for bar installation. Follow the manufacturer's instructions and conduct trial installations to determine the best hole diameter and depth.

Hole cleanout can be performed by inserting a tube into the hole and blowing through the tube with oil-free compressed air. The jet of compressed air blows away the dust within the drill hole. Compressed air should be checked for the presence of moisture and oil in accordance with the ASTM D 4285 method before commencement of work and at least once every 4 h. A nylon brush used in combination with the air pressure cleans the inside of drill holes sufficiently. Carbide-tipped drills with internal vacuum ports are available to save time blowing out the hole after drilling. Sufficient bonding agent should be installed into the hole to fill the annular space. The epoxy bonding agent is displaced by the dowel and fills the hole opening, if installed properly. If the hole is too tight for the resin viscosity, then installation of the dowel may not be possible. The viscosity of the bonding agent should be fluid enough to permit the agent to flow between the dowel and the hole.

5.3.5 *Example*—The Kansas Department of Transportation strengthened 84 reinforced concrete girder bridges as shown in Fig. 5.4 (Stratton and Crumpton 1984; Stratton, Alexander, and Nolting 1978, 1982). The Kansas bridges had diagonal tension cracks in many of the girders of older two-lane, two-girder, reinforced concrete structures. Calculations and exploratory programs beginning around 1978 indicated that the load-carrying capability of the bridges could be significantly enhanced by adding shear reinforcement in holes drilled diagonally. This procedure not only repaired the existing cracks but it added diagonal tension strength to the members.

The procedure found most effective and used was:

1. Diagonal cracks on the surface of the girders were sealed with a silicone gel;

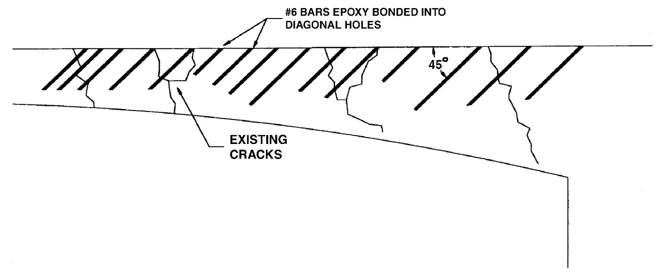

Fig. 5.4—Reinforced concrete girder of a Kansas bridge repaired and strengthened using reinforcing bars epoxy bonded into drilled holes (Stratton, Alexander, and Nolting 1978).

2. One in. (25 mm) diameter holes spaced approximately 14 in. (350 mm) apart along the length of the girder were drilled at a 45-degree angle on the centerline of each girder with a vacuum drill working from above;

3. Epoxy was pressure-injected into the holes to fill the diagonal cracks and to nearly fill each hole; and

4. A 19 mm diameter (No. 6) deformed bar was inserted into the hole. The bar was approximately 75 mm (3 in.) shorter than the full depth of the hole so that the top of the bar did not protrude out of the top of the hole.

In this reinforcing procedure, the bond between the steel reinforcement and the concrete was provided by the epoxy bond of the bar to the surrounding concrete after first repairing the concrete using epoxy injection.

Similar repair procedures were used on another project where about 800 concrete roof joists did not have adequate shear capacity. The structure was below grade, covered with backfill, and had an elastomeric waterproofing membrane and concrete protection slab on top of the roof slab. Drilling from the top was not feasible, and the following repair procedure was used:

1. Cracks were injected with an epoxy adhesive;

2. Holes were drilled from the bottom of the joists upward at a 45-degree angle;

3. After cleaning, a two-component fast-setting adhesive resin in a premeasured cartridge was inserted into the hole;

4. A reinforcing bar with a specially designed threaded coupler was inserted in the hole while being rotated by a drill. The rotating bar broke apart the cartridge and mixed the two-component resin. The specially designed coupler prevented the resin from running out of the hole and held the bar in place until the adhesive hardened in a few minutes; and

5. The threaded coupler was removed and the remaining shallow hole was patched.

5.4—Exterior reinforcement (encased and exposed)

5.4.1 *Description*—Exterior reinforcement may consist of steel brackets; steel plates; reinforced concrete shrouding; and composite materials such as CFRP, GFRP, or equivalent materials placed on the exterior of an existing concrete member. The new reinforcement may be encased with concrete, shotcrete, mortar, plaster, waterproofing, fireproofing, or other product, or it may be left exposed and

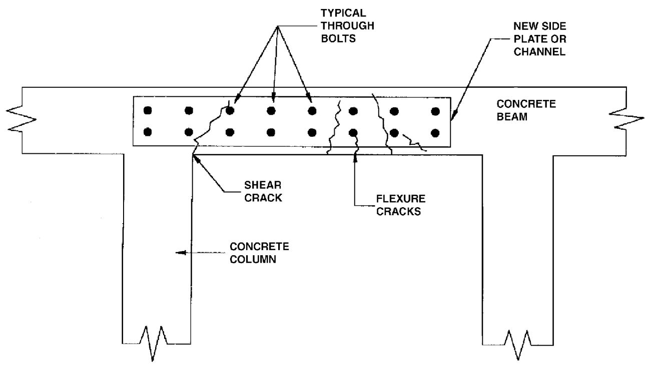

Fig. 5.5—Application of steel strapping to repair of 66 ft (20.1 m) long girders (Stratton, Alexander, and Nolting 1982).

protected from corrosion with a coating. The reinforcement may be deformed bars, welded-wire fabric, steel plate, steel rolled sections, steel strapping, composite materials, or specially fabricated brackets. For members damaged by overload, erosion, abrasion, or chemical attack, the deteriorated or cracked concrete can be removed and new reinforcement installed around and adjacent to the remaining concrete. The new reinforcement is encased in conventionally placed concrete or in shotcrete. Where the existing concrete is in good condition, the new reinforcement may be bonded directly to the existing concrete surface after preparing the surface as described in Chapter 2. Epoxy and other chemical adhesives and portland-cement concrete can be used to bond the new reinforcement to the substrate or it may be mechanically fastened to the existing concrete.

5.4.2 *Advantages and typical uses*—Placement of exterior reinforcement may be the most convenient method for repair and strengthening where obstructions limit access of equipment needed for placement of interior reinforcement. If surface repair and placement of surface epoxy mortar, plaster, or shotcrete is required for concrete rehabilitation, placement of external reinforcement for strengthening may be accomplished in the same construction process.

External flexural, shear, and torsion reinforcement for beams and girders may be provided by bonding deformed reinforcing bars or plates to the surface of concrete girders with shotcrete, cast-in-place concrete, or epoxy and polymer concrete (Kajfasz 1967; Toemsmeyer 1981; Holman and Cook 1984; Kahn, Townsend, and Kaldjian 1975). Anchors may be required in the repair method to ensure composite action.

Steel plates can be attached to existing girders using bolts as illustrated in Fig. 5.5. Using a structural adhesive requires adequate surface preparation of both the steel and the concrete and selection of the appropriate adhesive to bond steel to concrete. Sandblasting of both the steel plate and concrete surface is the best method of preparation, but surface cleaning with mechanical means or high-pressure water blasting may be adequate for many cases.

Banding girders and columns with steel straps, as shown in Fig. 5.5, is effective in increasing shear resistance (Lunoe and Willis 1957; Kahn 1980). The 13 to 50 mm (1/2 to 2 in.) wide steel packaging bands are not bonded to the member, but their banding prestress effectively anchors each strap. Steel clamps bolted around an existing reinforced concrete member are also effective (Kahn 1980).

Beams, girders, columns, and walls can be strengthened by placement of longitudinal reinforcing bars and stirrups or ties around the members and then encasing the members with shotcrete or cast-in-place concrete (Fratt 1973; Davis 1978; Strand 1973). The shotcrete bonds the new reinforcement to the existing member. The added shotcrete also increases the size of the member and adds strength and stiffness. Consideration should be given to the additional dead weight of the member resulting from such additions.

Both masonry and reinforced concrete walls have been strengthened by adding surface layers of reinforcing bars or welded-wire fabric and by applying shotcrete (Strand 1973; Fratt 1973; Kahn 1984). The shotcrete bonds the new reinforcement to the wall. Often, new dowels are embedded in holes drilled into perimeter columns and beams for a connection between the frame members and the strengthened, infilled shotcrete wall.

Beams, columns, and wall sections have recently been strengthened by carbon fiber, CFRP, glass fiber, GFRP, or equivalent composite materials installed with bonding resin. Laboratory tests performed by the manufacturers provide design criteria for the use of their proprietary systems. Evaluation of long-term strength in field conditions are not available in the United States; however, this method has been used for several years in Europe. Banding and strengthening with carbon or glass sheets or strips and equivalent composite materials is a cost-effective strengthening method used to upgrade structural members requiring seismic code upgrade modifications. This method is suitable for use where service loads have been increased as long as ductility is considered along with proper provisions for fireproofing and waterproofing.

5.4.3 *Limitations*—Because the stiffness of most repaired members is increased when plates or concrete encasement is used, load distribution in the structure is altered and should be analyzed. New possible overstresses may result at transitions between strengthened and unstrengthened sections. Both repaired and unrepaired members, including foundations, should be checked for service load conditions. Strengthening with composite materials may provide the necessary strengthening but, because of the elastic behavior of FRPs, the ultimate behavior of FRP reinforcement should be carefully evaluated to ensure satisfactory performance.

External reinforcement always occupies space that was available for other uses before the repair.

Surface preparation of both steel and concrete is critical if bonding is required for composite action. Chapter 2 presents surface preparation techniques.

Careful consideration should be exercised when using structural adhesives, especially epoxy, due to their softening and loss of strength at elevated temperatures near to and above their glass-transition temperature, which can be as low as 50 °C (120 °F). Where required, appropriate fire protection should be provided. Proper detailing and protection is especially critical when using composite materials for strengthening because they are generally adhered to substrates with epoxies. Most adhesives are water-sensitive and, when applied to damp or moist surfaces, do not properly bond.

5.4.4 *Examples*—The shear resistance of large reinforced concrete girders was increased with steel straps surrounding the girders (Lunoe and Willis 1957). The 50 mm wide by 1.3 mm thick (2 by 0.05 in.) straps are identical to those used in material handling and packaging applications. The corners of the girders were protected with bent 1.5 mm thick (16 gauge) steel plates. The straps were tensioned to a stress of approximately 290 MPa (42,000 psi); two seals were crimped in place to secure the straps. Tests at the Portland Cement Association (PCA) (Elstner and Hognestad 1957) indicated that the shear resistance provided by the strap reinforcement could be predicted in the same manner as the strength provided by embedded stirrups. The PCA tests indicated that a prestress of about 170 MPa (25,000 psi) was necessary to ensure full efficiency of the banding straps.

The shear strength of reinforced concrete girders in a parking garage was increased by adding external stirrups encased in shotcrete. The repair procedure was as follows:

1. Side and bottom surfaces of beams were sandblasted to provide clean and rough surfaces;
2. New external stirrups consisting of pairs of U-shaped bars lapped together were placed around the existing beam. The horizontal section of the upper U-bar was placed in a slot cut into the top of the girder and vertical legs extended through holes cut in the slab;
3. The new stirrups were encased in shotcrete; and
4. Shotcrete encasements were sounded to detect unbonded areas. Unbonded areas were removed and replaced.

Carbon fiber reinforcement should be installed over a properly prepared concrete surface. Sandblasting or mechanical cleaning concrete is recommended to prepare the concrete surface. A bond adhesive is applied over the concrete surface and then the premanufactured carbon fiber mesh is installed over the member surface. A final protective cover should be applied to protect the repair from the elements and fire. A multistory parking structure (Michols and Vincent 1990) and many bridges in Europe, Japan, the United States, and Canada have been strengthened using this technique (Cruickshank 2002).

5.5—Exterior post-tensioning

5.5.1 *Description*—Girders and slabs may be strengthened in both flexure and shear by the addition of external tendons, rods, or bolts that are prestressed. Figure 5.6 illustrates this external post-tensioning. The strands may be straight or deflected.

5.5.2 Advantages and typical uses—The use of high-strength prestressing reinforcement effectively reduces the amount of reinforcement required in the repair and strengthening procedure. The prestressing also provides beam camber that helps to reduce deflection. The addition of a single post-tensioning arrangement increases both shear and flexural strength. The shear resistance is increased due to concrete compression and to the profile of the strands, and the flexural strength is increased due to the addition of longitudinal reinforcement. The exterior post-tensioning method is performed with the same equipment and the same design criteria that all prestressing and post-tensioning projects require. The end plates should be properly designed, securely seated, and sufficient space should be provided for pulling the strands. Strand tensions should be determined in accordance with PCI, PTI, and ACI guidelines. Additional information is available in Nawy (1997).

5.5.3 *Limitations*—Placement and stressing of prestressing strands and rods are often difficult in congested buildings and interior locations. If positioning of hydraulic jacks is impossible or laborious, turnbuckles or nuts on rods can be used to tension the steel.

Closing of cracks may not be possible because debris often collects in the open crack. As stressing progresses, debris and aggregates already in contact keep the crack open. Cleaning and filling the crack with epoxy or other appropriate material may be necessary before prestressing.

Providing adequate anchorage of the tendons should be mandatory even though it may be difficult. Use of steel grillages on the far side of columns and at the end of beams (Fig. 5.6) is best because such anchorage uses bearing forces and permit stressing of the entire length of a member, including end connections. Bolted or bonded side anchors can be alternate anchorages. The anchors should be designed for the eccentric forces, and the portion of the concrete members beyond the anchors should be checked for strength.

In indeterminate frames, post-tensioning induces secondary shear and moments that should be considered when planning the repair. Also, the new, exposed post-tensioned strands are visible and may be distracting. Strands can be concealed in concrete or by cladding. Building codes should be checked for the installation of fireproofing on exposed reinforcement.

5.5.4 *Example*—Exterior post-tensioning was used to restore strength and serviceability of beams in a parking garage located in San Francisco (Aalami and Swanson 1988). Two multi-strand tendons per beam were added, one on each side. New tendons were deflected down to the bottom of the beam at midspan using specially designed hardware. The corrosion protection and fireproofing system around the new tendons consisted of a 50 mm (2 in.) diameter corrugated PVC pipe encased in a 165 mm (6.5 in.) square precast concrete tube. The innovative repair design allowed the garage to remain fully operational during installation of the new tendons.

For conditions where the original structure is underdesigned, the actual imposed service loads should be reduced or shoring provided until the strengthening is completed. Unloading of the structural elements should be required if the new reinforcement is to support the existing loads.

5.6—Jackets and collars

5.6.1 *Description*—Jacketing is the process whereby a section of an existing structural member is restored to original dimensions or increased in size by encasement by a variety of materials. A steel reinforcement cage or composite material wrap can be constructed around the damaged section onto which shotcrete or cast-in-place concrete is placed.

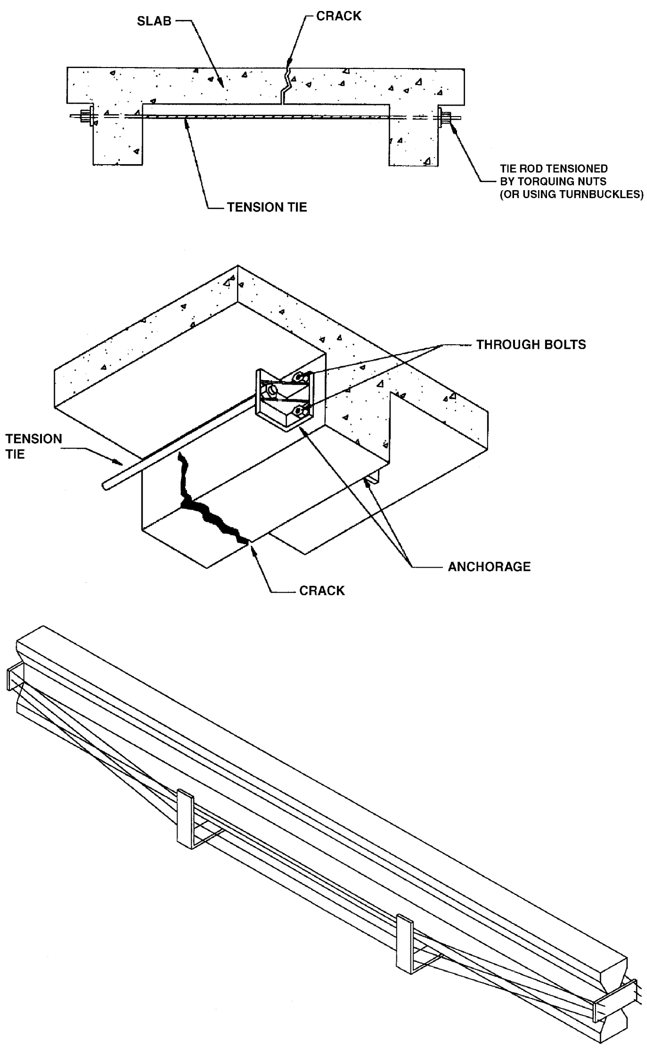

Fig. 5.6—External prestressing to close cracks and strengthen section.

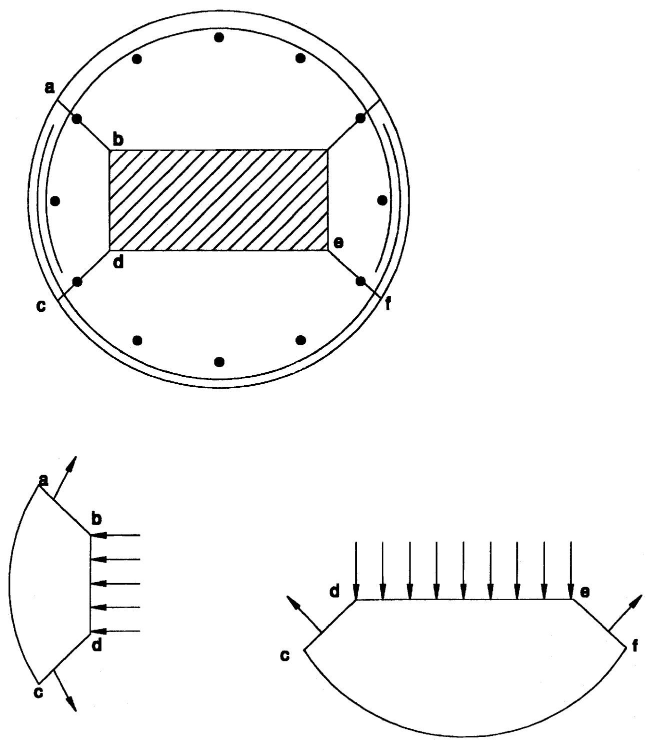

Fig. 5.7—Collar strengthening of a column (Klein and Gouwens 1984).

Collars are jackets that surround only a part of a column or pier and typically are used to provide increased support to the slab or beam at the top of the column (Fig. 5.7).

The form for the jacket may be temporary or permanent and may consist of timber, corrugated metal, precast concrete, rubber, fiberglass, or special fabric, depending on the purpose and exposure. The jacket form is placed around the section to be repaired, creating an annular void between the jacket and the surface of the existing member. The form should be provided with spacers to ensure equal clearance between it and the existing member.

A variety of materials, including conventional concrete and mortar, epoxy mortar, grout, and latex-modified mortar and concrete, have been used as encasement materials. Techniques for filling the jacket include pumping, tremie, or preplaced aggregate concrete.

5.6.2 *Advantage and typical uses*—Jacketing is particularly applicable in the repair of deteriorated columns, piers, and piling where all or a portion of the section to be repaired is under water. The method is applicable for protecting concrete, steel, and timber sections against further deterioration and for strengthening. Permanent forms are advantageous in marine environments where added protection from weathering, abrasion, and chemical pollution is desired. Collars are effective in providing new capitals on existing columns for supporting slab floors (Klein and Gouwens 1984). The collar provides increased shear capacity for the slab, and it decreases the effective length of the column. Collars may help satisfy architectural constraints better than jacketing the column for its full height.

5.6.3 *Limitations*—Both jackets and collars require that all deteriorated concrete be removed, cracks repaired, existing reinforcement cleaned, and surfaces prepared. This preparation is required so that the newly placed materials bond with the existing structure. Because jackets are often under water, such preparation is expensive and difficult. Nevertheless, the applicability of jackets and collars is widespread and is cost-effective, especially if the alternative is replacement of the structure supported by the deteriorated member. For underwater conditions, a plastic shell may be applied at the splash zone to help minimize abrasion.

Jackets and collars occupy space that was available for other uses before repair.

5.6.4 *Installation details*—The forming techniques and the decision to use permanent or temporary forms are important details in jacketing. Timber or cardboard forms may be used as temporary or permanent forms. Corrugated steel forms are easy to assemble and are adequate temporary or permanent forms. Permanent fiberglass, rubber, and fabric forms have gained wide acceptance and some may provide resistance to chemical attack after the repair is complete.

5.7—Supplemental members

5.7.1 *Description*—Supplemental members are new columns, beams, braces, or infilled walls that are installed to support damaged structural members, as illustrated in Fig. 5.8. The supplemental members are typically placed below the failure or deflected areas to stabilize the structural framing.

5.7.2 *Advantages and typical uses*—This repair method can be used if none of the other strengthening techniques is adequate for repair or if the structural configuration precludes use of other techniques. Supplemental members are quickly installed and, therefore, are suitable temporary emergency repair solutions. Typically, new members are installed to support seriously cracked and deflected flexural members. Often, the use of supplemental members may be the most economical alternative.

5.7.3 *Limitations*—Installing new columns or beams may restrict space within the repaired column bay. A new column obstructs passage and new beams reduce head room. Aesthetically, the new beam or new column is noticeable and distracting. The use of cross bracing, infilled walls, or other means of providing resistance to lateral forces may be necessary if the original structure does not provide the necessary resistance. Such bracing further restricts interior space utilization. Loads and stresses in the existing structure may not be relieved unless special procedures are used.

The supplemental members may cause a redistribution of loads and forces that overstress an existing nearby member, foundations, or both.

5.7.4 *Installation details*—The new supplemental members may be timber, steel, concrete, or masonry. The new members should be tightly shimmed, wedged, or anchored in position so that loads are transferred to the new members. Care should be taken not to lift existing structural members to avoid load-redistribution overstress of beams or adjacent support members.

In Fig. 5.8(a), the new post supports a beam weak in flexural capacity. The new post requires adequate foundation support. The single span has become continuous so that negative and positive moment regions may be reversed. If cracking occurs in the new negative moment region, its acceptability should be considered.

Figure 5.8(b) illustrates a post added to improve the shear resistance and reduce the effective span of the existing beam. Often, such posts added adjacent to the existing column are more economical than collars. The post may be positioned eccentrically on an existing footing that should then be analyzed to determine if its size or strength needs to be increased. In placing the post, a permanent jack, shims, or both, may be required. The engineer should consider if jacking effectively redistributes dead loads to the new posts.

Figure 5.8(c) illustrates placement of a supplemental beam beneath an existing deflected slab. The space between the new beam and existing structure should be shimmed or dry packed. To provide lateral stability to the supplemental member, it may be necessary to mechanically anchor it to the existing slab, columns, or both. If the existing slab is supported on beams, the new beam could be supported on the existing beams instead of new posts.

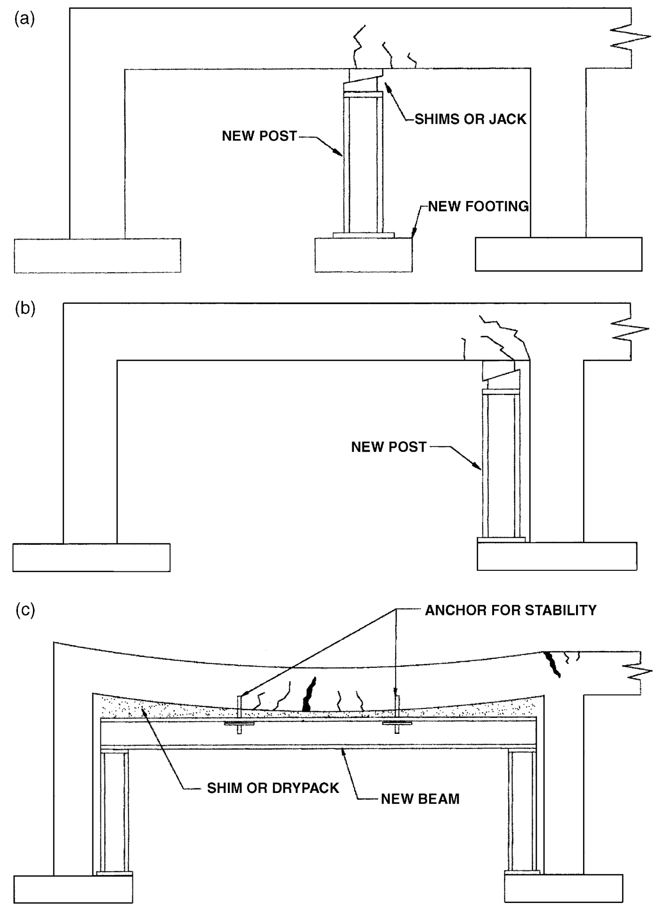

Fig. 5.8—Supplemental members.

5.7.5 *Example*—A reinforced concrete multistory condominium suffered severe shear and flexural cracking in the second-story girders. The girders supported a lobby area over a parking level. Because parking access would not be adversely affected, steel columns were shimmed into position beneath many girders. Small reinforced concrete spread footings were located beneath the new columns.

5.8—Repair of concrete columns

Repair procedures for concrete columns should consider the compressive loads on the columns to be repaired. Structural columns, by their nature, are designed to carry vertical and sometimes lateral loads and moment-induced loads. Specifically, the dead load weight of the structure plus the live loads should be taken into consideration.

5.8.1 *Concrete column repair categories*—Repairs to concrete columns typically fall into two categories: surface or cosmetic repairs that are designed to address local deterioration, or structural repairs that enhance or re-establish the load-carrying capacity of the affected columns.

When concrete columns require repair due to surface deterioration, compressive loads are redistributed to the undisturbed cross section of the column. If the deterioration does not significantly reduce the cross section, the conventional concrete repair procedures can successfully address the damage.

When column deterioration is significant, then unloading the column may be required so that once the repair is complete, the entire cross section of the column is capable of carrying the reintroduced design load. If it is not possible to remove the load from the column, then a supplemental column system can provide an alternative method of support in conjunction with the repair of the existing column.

5.8.2 *Column repair strategies*—Column repair strategies vary with the condition parameters but include:

- Encasement or enlargement of the column cross section;
- Cathodic protection to stop reinforcing steel corrosion;
- Realkalization of the reinforcing steel to stop corrosion;
- Chloride extraction to retard the reinforcing steel corrosion;
- Confinement using steel plate, carbon, or glass fiber materials;
- Addition of shear collars to increase the shear capacity of intermediate floors;
- Addition of a steel plate assembly to increase moment capacity;
- Supplemental columns; and
- The application of a protection system to prevent future corrosion.

5.9—Column repair parameters

5.9.1 *Unloading columns*— Unless the column is unloaded by transferring the vertical load above the repair to adjacent shoring before performing the column repair, it is unlikely that the new repair will carry any load, particularly if the repair exhibits any drying shrinkage. Unfortunately, it can be difficult and expensive to unload columns, especially in high-rise buildings.

5.9.2 *Redistribution of the load*—Redistribution of the vertical load has already occurred in the vicinity of the corroded reinforcing steel and resulting delamination before implementing any repairs. The designer should be aware of this and carefully evaluate the remaining cross section to determine if the redistribution load has overstressed the column locally. If so, it might be necessary to at least partially relieve the column of load before the repair operation.

5.9.3 *Supplemental vertical reinforcing steel*—Supplemental vertical column bars should ideally be placed within the column ties, such as the cage; however, this is difficult to accomplish without cutting column ties. A contractor should not cut column ties for fear of buckling loaded vertical reinforcing steel bars. Thus, it is usually prudent to place additional reinforcing steel bars outside the cage (Section 5.9.6).

5.9.4 *Concrete removal*—The removal of concrete within a column cage, especially along the length of the vertical reinforcing bars, is a major concern. When too much concrete is removed from inside the cage, the vertical bars could start to buckle, even if the column ties are left in tact. In addition, the removal of any concrete in a loaded column during the repair process results in the remaining concrete and reinforcing bars trying to carry the vertical load. If it is not done carefully, vertical bars could buckle, resulting in a compression failure of the column.

5.9.5 *Corroded reinforcing steel*—It is not necessary to remove the corroded reinforcing bar with reduced cross-sectional area if the loss is supplemented with additional reinforcing bars. The lap length of such a splice should then be provided to full cross-sectional areas to either side of the corroded portion of the reinforcing bar that is supplemented. The supplemental bar(s) would then extend off the deteriorated portion of the corroded reinforcement to replace the lost cross sectional area and be lapped beyond. Any corroded reinforcing that is left in place should be thoroughly cleaned by sandblasting, to bare metal if possible.

5.9.6 *Supplemental reinforcing steel*—When the supplemental bars are placed outside the tie bars, the column dimensions should be increased to provide adequate cover. Hairpin ties are often added, usually stainless steel, to laterally support the supplemental bars.

5.9.7 *Corroded ties*—If column repairs due to corroded ties are required, it is important that alternative methods be provided to laterally support the vertical bars. This can be accomplished by adding stainless steel hairpin ties that are anchored into the concrete. It is often necessary to build out columns to provide adequate cover over the supplemental ties. Potential buckling problems in the vertical bars may be avoided with this technique.

5.9.8 *Low-strength concrete*—Where the concrete strength is low, resulting in insufficient load-carrying capacity, several alternatives are available:

- Shore the column and remove and replace the in-place concrete;
- Shore the column and increase the size of the column to reduce the bending stresses, thereby stiffening it so that the axial load-carrying capacity is increased;
- Wrap the column with carbon- or glass-reinforced plastic; and
- Install a supplemental column.

CHAPTER 6—REFERENCES

6.1—Referenced standards and reports

The standards and reports listed below were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest version.

AASHTO

T 259 Standard Method of Test for Resistance of Concrete to Chloride Ion Penetration

American Concrete Institute

201.1R Guide for Making a Condition Survey of Concrete in Service

201.2R Guide to Durable Concrete

210R Erosion of Concrete in Hydraulic Structures

210.1R Compendium of Case Histories on Repair of Erosion-Damaged Concrete in Hydraulic Structures

211.1 Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete

222R Protection of Metals in Concrete Against Corrosion

222.2R Corrosion of Prestressing Steels

223 Standard Practice for the Use of Shrinkage-Compensating Concrete
224.1R Causes, Evaluation, and Repair of Cracks in Concrete Structures
224.3R Joints in Concrete Construction
228.2R Nondestructive Test Methods for Evaluation of Concrete in Structures
234R Guide for the Use of Silica Fume in Concrete
301 Specifications for Structural Concrete
304R Guide for Measuring, Mixing, Transporting, and Placing Concrete
304.1R Guide for the Use of Preplaced Aggregate Concrete for Structural and Mass Concrete Applications
304.2R Placing Concrete by Pumping Methods
304.5R Batching, Mixing, and Job Control of Lightweight Concrete
304.6R Guide for Use of Volumetric-Measuring and Continuous-Mixing Concrete Equipment
305R Hot Weather Concreting
306R Cold Weather Concreting
306.1 Standard Specification for Cold Weather Concreting
308R Guide to Curing Concrete
308.1 Standard Specification for Curing Concrete
311.1R (SP-2) ACI Manual of Concrete Inspection
311.4R Guide for Concrete Inspection
311.5R Guide for Concrete Plant Inspection and Testing of Ready-Mixed Concrete
318 Building Code Requirements for Structural Concrete and Commentary
345R Guide for Concrete Highway Bridge Deck Construction
350.1/350.1R Tightness Testing of Environmental Engineering Concrete Structures and Commentary
355.1R State-of-the-art Report on Anchorage to Concrete
362.1R Guide for the Design of Durable Parking Structures
362.2R Guide for Structural Maintenance of Parking Structures
364.1R Guide for Evaluation of Concrete Structures Prior to Rehabilitation
437R Strength Evaluation of Existing Concrete Buildings
439.3R Mechanical Connections of Reinforcing Bars
440.3R Guide Test Methods for Fiber Reinforced Polymers (FRP) for Reinforcing or Strengthening Concrete Structures
503R Use of Epoxy Compounds with Concrete
503.4 Standard Specifications for Repairing Concrete with Epoxy Mortars
503.6R Guide for the Application of Epoxy and Latex Adhesives for Bonding Freshly Mixed and hardened Concretes
504R Guide to Sealing Joints in Concrete Structures
506R Guide to Shotcrete
506.1R Committee Report on Fiber Reinforced Shotcrete
506.2 Specification for Shotcrete
506.3R Guide to Certification of Shotcrete Nozzlemen
515.1R Guide to the Use of Waterproofing, Dampproofing, Protective, and Decorative Barrier Systems for Concrete
544.1R State-of-the-Art Report on Fiber-Reinforced Concrete
544.3R Guide for Specifying, Proportioning, Mixing, Placing, and Finishing Steel Fiber Reinforced Concrete
544.4R Design Considerations for Steel Fiber Reinforced Concrete
546.1R Guide for Repair of Concrete Bridge Superstructures
546.2R Guide to Underwater Repair of Concrete
548.1R Guide for the Use of Polymers in Concrete
548.2R Guide for Mixing and Placing Sulfur Concrete in Construction.
548.3R Polymer Modified Concrete
548.4 Standard Specification for Latex-Modified Concrete (LMC) Overlays
548.5R Guide for Polymer Concrete Overlays
549R State-of-the-art Report on Ferrocement
549.1R Guide for the Design, Construction and Repair of Ferrocement

ASTM International

A 185 Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement
A 497 Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement
A 615/615M Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
A 616/616M Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement
A 617/617M Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement
A 706/706M Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement
A 767/767M Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement
A 775/775M Specification for Epoxy-Coated Reinforcing Steel Bars
A 780 Practice for Repair of Damaged and Uncoated Areas of Hot-Dip Galvanized Coatings
B 117 Test Method of Salt Spray (Fog) Testing
C 33 Specification for Concrete Aggregates
C 67 Test Methods of Sampling and Testing Brick and Structural Clay Tile
C 94 Specification for Ready-Mixed Concrete
C 267 Test Method for Chemical Resistance of Mortars
C 387 Specification for Packaged, Dry, Combined Materials for Mortar and Concrete
C 642 Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete

C 672	Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals
C 685	Specification for Concrete Made by Volumetric Batching and Continuous Mixing
C 806	Test Method for Restrained Expansion of Expansive Cement Mortar
C 836	Specification for High Solids Content, Cold Liquid-Applied Elastomeric Waterproofing Membrane for Use with Separate Wearing Course
C 845	Specification for Expansive Hydraulic Cement
C 878	Test Method for Restrained Expansion of Shrinkage-Compensating Concrete
C 881	Specification for Epoxy-Resin-Base Bonding Systems for Concrete
C 928	Specification for Packaged, Dry, Rapid-Hardening Cementitious Materials for Concrete Repairs
C 937	Specification for Grout Fluidifier for Preplaced-Aggregate Concrete
C 938	Practice for Proportioning Grout Mixtures for Preplaced-Aggregate Concrete
C 1059	Specification for Latex Agents for Bonding Fresh to Hardened Concrete
C 1107	Specification for Packaged Dry Hydraulic-Cement Grout (Nonshrink)
C 1116	Specification for Fiber-Reinforced Concrete and Shotcrete
C 1202	Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration
C 1240	Specification for Silica Fume for Use in Hydraulic-Cement Concrete and Mortar
C 1438	Specification for Latex and Powder Polymer Modifiers for Hydraulic Cement Concrete and Mortar
C 1439	Test Methods for Polymer-Modified Mortar and Concrete
D 56	Test Method for Flash Point by Tag Closed Tester
D 93	Test Methods for Flash Point by Pensky-Martens Closed Tester
D 412	Test Methods for Vulcanized Rubber and Thermoplastic Rubbers and Thermoplastic Elastomers-Tension
D 429	Test Method for Rubber Property-Adhesion to Rigid Substrates
D 522	Test Methods for Mandrel Bend Test of Attached Organic Coatings
D 523	Test Method for Specular Gloss
D 1353	Test Method for Nonvolatile Matter in Volatile Solvents for Use in Paint, Varnish, Lacquer, and Related Products
D 1474	Test Methods for Indentation Hardness of Organic Coatings
D 1640	Test Methods for Drying, Curing, or Film Formation of Organic Coatings at Room Temperature
D 1642	Test Methods for Elasticity or Toughness of Varnishes
D 1647	Test Method for Resistance of Dried Films of Varnishes to Water and Alkali
D 1653	Test Method for Water Vapor Transmission of Organic Coating Films
D 2047	Test Method for Static Coefficient of Friction of Polish-Coated Floor Surfaces as Measured by the James Machine
D 2134	Test Method for Determining the Hardness of Organic Coatings with a Sward-Type Hardness Rocker
D 2196	Test Method for Rheological Properties of Non-Newtonian Materials by Rotational (Brookfield) Viscometer
D 2197	Test Methods for Adhesion of Organic Coatings by Scrape Adhesion Tester
D 2240	Test Method for Rubber Property-Durometer Hardness
D 2247	Practice for Testing Water Resistance of Coatings in 100% Relative Humidity
D 2620	Test Method for Light Stability of Clear Coatings
D 2794	Test Method for Resistance of Organic Coatings to the Effects of Rapid Deformation (Impact)
D 3274	Test Method of Evaluating Degree of Surface Disfigurement of Paint Films by Microbial (Fungal or Algal) Growth or Soil and Dirt Accumulation
D 3278	Test Method for Flash Point of Liquids by Setaflash-Closed-Cup Apparatus
D 3359	Test Methods for Measuring Adhesion by Tape Test
D 3363	Test Method for Film Hardness by Pencil Test
D 3456	Practice for Determining by Exterior Exposure Tests Susceptibility of Paint Films to Microbiological Attack
D 4138	Test Method for Measurement of Dry Film Thickness of Protective Coating Systems by Destructive Means
D 4141	Practice for Conducting Accelerated Outdoor Exposure Tests for Coatings
D 4214	Test Methods for Evaluating Degree of Chalking of Exterior Paint Films
D 4541	Method for Pull-Off Strength of Coatings Using Portable Adhesion-Testers
D 4285	Test Method for Indicating Oil or Water in Compressed Air
E 84	Test Method for Surface Burning Characteristics of Building Materials
E 96	Test Methods for Water Vapor Transmission of Materials

E 329	Standard Specification for Agencies Engaged in the Testing and/or Inspection of Materials Used in Construction
E 514	Test Method of Water Permeance of Masonry
G 26	Practice for Operating Light Exposure Apparatus (Xenon-Arc Type) with and without Water for Exposure of Nonmetallic Materials

AWS

D1.4	Structural Welding Code—Reinforcing Steel

ICBO

AC 125	Acceptance Criteria for Concrete and Reinforced and Unreinforced Masonry Strengthening Using Fiber Reinforced Composite Systems

ICRI

03730	Guide for Surface Preparation for the Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion
03731	Guide for Selecting Application Methods for the Repair of Concrete Surfaces
03732	Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, and Polymer Overlays
03734	Guide for Verifying the Field Performance of Epoxy Injection of Concrete Cracks
03735	Guidelines for Methods of Measurement and Contract Types of Concrete Repair
03736	Guide for the Evaluation of Unbonded Post-Tensioned Concrete Structures
03737	Guide for Preparation of Concrete Surfaces for Repair Using Hydrodemolition Methods

NSF

NSF 61	Drinking Water System Components—Health Effects

These publications may be obtained from these organizations:

American Association of State Highway and Transportation Officials (AASHTO)
444 North Capitol Street NW, Suite 225
Washington, DC 20001

American Concrete Institute (ACI)
P. O. Box 9094
Farmington Hills, MI 48333-9094

ASTM International
100 Barr Harbor Drive
West Conshohocken, PA 19428-2959

American Welding Society (AWS)
P. O. Box 351040
Miami, FL 33135

International Concrete Repair Institute (ICRI)
3166 South River Road, Suite 132
Des Plaines, IL 60018

NACE International (NACE)
1440 South Creek Drive
Houston, TX 77084-4906

NSF International (NSF)
P. O. Box 130140
Ann Arbor, MI 48113-0140

6.2—Cited references

Aalami, B. O., and Swanson, D. T., 1988, "Innovative Rehabilitation of a Parking Structure," *Concrete International*, V. 10, No. 2, Feb., pp. 30-35.

Babaei, K., and Hawkins, N., 1988, "Evaluation of Bridge Deck Protective Strategies," *Concrete International*, V. 10, No. 12, Dec., pp. 56-66.

Berke, N. S.; Dallaire, M. P.; Weyers, R. E.; Henry, M.; Peterson, J. E.; and Powell, B., 1992, "Impregnation of Concrete Corrosion Inhibitors, Corrosion Forms and Control for Infrastructure," *ASTM STP 1137*, V. Chaker, ed., ASTM International, West Conshahocken, Pa., 432 pp.

Brown, M.; Bryant, J.; and Weyers, R. E., 1999, "Evaluation of a Post Treatment System," *American Concrete Institute Spring Convention Technical Session*, Chicago, Ill.

Cady, P.; Weyers, R. E.; and Wilson, D. T., 1984, "Durability and Compatibility of Overlays and Bridge Deck Substrate Treatments," *Concrete International*, V. 6, No. 6, June, pp. 36-44.

Carter, P. D, 1993, "Evaluation of Dampproofing Performance and Effective Penetration Depth of Silane Sealers in Concrete," *Concrete Bridges in Aggressive Environments, Philip D. Cady Symposium,* SP-151, R. E. Weyers, ed., American Concrete Institute, Farmington Hills, Mich., pp. 95-117.

Cruickshank, S., 2002, "Innovative Approach for Restoration of Historic Bridge," *Concrete Repair Bulletin*, July-Aug., pp. 6-7.

Davis, H. A., 1978, "The Center for Educational Development—A Medium Scale Seismic Upgrading," *Proceedings, North American Masonry Conference*, Paper No. 42, Boulder, Colo.

Elsner, B., 2000, "Corrosion Inhibitors for Steel in Concrete," *International Congress on Advanced Materials,* Keynote Lecture, Materials Week, Munich, Germany.

Elsner, B.; Buchler, M.; Stadler, F.; and Bohni, H., 1999, "Migrating Corrosion Inhibitor Blend for Reinforced Concrete," *Corrosion,* NACE, V. 55, No. 12.

Elsner, B.; Buchler, M.; and Bohni, H., 1997, "Corrosion Inhibitors for Steel in Concrete," *Proceedings of Eurocorr*, Trondheim, Norway.

Elstner, R. C., and Hognestad, E., 1957, "Laboratory Investigation of Rigid Frame Failure," ACI JOURNAL, *Proceedings* V. 53, 1957, pp. 637-668.

Emmons, P. H., 1994, *Concrete Repair and Maintenance Illustrated: Problem Analysis, Repair Strategy, Techniques*, R. S. Means Co., Kingston, Mass., 300 pp.

Federal Specification TT-C-555B, "Coating, Textured (for Interior and Exterior Masonry Surfaces)."

Federal Specification TT-P-1411A, "Paint, Copolymer-Resins Cementitious (for Waterproofing Concrete and Masonry Walls)."

Fratt, J. M., 1973, "Structural Repair of Earthquake Damaged Buildings," *ASCE Structural Engineering Meeting,* No. 1996, San Francisco, Calif.

Gaul, R. W., 1984, "Preparing Concrete Surfaces for Coatings," *Concrete International*, V. 6, No. 7, July, pp. l7-22.

U.S. Army Corps of Engineers, 1995, "Evaluation and Repair of Concrete Structures," *Engineer Manual* 1110-2-2002, Washington D.C., 184 pp.

Holland, T. C., 1987, "Working with Silica-Fume Concrete," *Concrete Construction*, V. 32, No. 3, pp. 261-266.

Holland, T. C., and Gutschow, R. A., 1987, "Erosion Resistance with Silica-Fume Concrete," *Concrete International*, V. 9, No. 3, Mar., pp. 32-40.

Holman, J. W., and Cook, J. P., 1984, "Steel Plates for Torsion Repair of Concrete Beams," *Journal of Structural Engineering*, ASCE, V. 110, No. 1, pp. 10-18.

Issac, S., and White, D., 2002, "Seismic Upgrade of I-80 Bridge in Salt Lake City Using Carbon Fiber Reinforced Polymers," *Concrete Repair Bulletin*, Mar.-Apr., pp. 6-9.

Jones, G.; Lambert, P.; and Wood, I., 2000, "Preventing an Achilles Heel," *Concrete Engineering International*, 2000.

Kahn, L. F., 1980, "Strengthening Existing RC Columns for Earthquake Resistance," *Proceedings,* V. 4, Seventh World Conference on Earthquake Engineering, Istanbul, Turkey, pp. 327-334.

Kahn, L. F., 1984, "Shotcrete Strengthening of Masonry Walls," *Concrete International*, V. 6, No. 7, July, pp. 34-40.

Kahn, L. F.; Townsend, W. H.; and Kaldjian, M, 1975, "Ferrocement Steel-Plate Composite Beams," ACI JOURNAL*, Proceedings* V. 72, No. 3, Mar., pp. 94-97.

Kajfasz, S., 1967, "Concrete Beams with External Reinforcement Bonded by Gluing-Preliminary Investigation," *RILEM International Symposium*, Paris.

Klein, G. J., and Gouwens, A. J., 1984, "Repair of Columns Using Collars with Circular Reinforcement," *Concrete International*, V. 6, No. 7, July, pp. 23-31.

Kuhlmann, L. A., 1984, *The Effect of Cure Time on Chloride Permeability of Latex-Modified Concrete*, Dow Chemical Co., Midland, Mich.

Kuhlmann, L. A., 1990, "Test Method for Measuring the Bond Strength of Latex Modified Concrete and Mortar," *ACI Materials Journal*, V. 87, No. 4, July-Aug., pp. 387-394.

Lunoe, R. R., and Willis, G. A., 1957, "Application of Steel Strap Reinforcement to Girders of Rigid Frames, Special AMC Warehouses," ACI JOURNAL, *Proceedings* V. 53, pp. 669-678.

McDonald, J. E., 1987, "Rehabilitation of Navigation Lock Walls: Case Histories," *Technical Report* REMR-CS-13, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.

McDonald, J. E.; Vaysburg, A. M.; Emmons, P. H.; Poston, R. W.; and Kesner, K. E., 2002, "Selecting Durable Repair Materials: Performance Criteria—Summary," *Concrete International*, V. 24, No. 1, Jan., pp. 37-44.

Michols, K. A., and Vincent, J. F., 1990, "Reinforced Overlay and Shotcrete Restore Integrity to a Multistory Parking Structure," *Concrete International*, V. 12, No. 9, Sept., pp. 55-58.

Mietz, J.; Elsner, B.; and Polder, R., 1997, "Corrosion of Reinforcement in Concrete—Monitoring, Prevention and Rehabilitation," *Publication* No. 25, European Federation of Corrosion (Eurocorr).

Murray, M. A., 1989, "Surface Preparation for Adhesives," *Concrete International*, V. 11, No. 9, Sept., pp. 69-71.

Murray, M. A., 1993, "Damaged Bridge Girders Restored," *Concrete Repair Bulletin*, July-Aug., pp. 23-24.

NCHRP Synthesis 45, Washington, D. C., 13 pp.

Nawy, E. G., 1997, "Unbonded Post-Tensioning in Building Construction," *Concrete Construction Engineering Handbook*, CRC Press, Boca Raton, Fla., 1232 pp.

Nittenger, B., 1997, "University of Michigan Puts Hydrodemolition to the Test," *Concrete Repair Bulletin*, V. 10, No. 2, Mar.-Apr., pp. 14-17.

Ozyildirim, C., 1993, "A Field Investigation of a Concrete Overlay Containing Silica Fume on Route 50 over Opequon Creek," *Final Report*, Virginia Transportation Research Council, Charlotteville, Va., June, 22 pp.

Paul, J. H., 1998, "Extending the Life of Concrete Repairs," *Concrete International*, V. 20 , No. 3, Mar., p. 64.

Pfeifer, D. W.; Landgren, J. R; and Zoob, A., 1987, "Protective Systems for New Prestressed and Substructure Concrete," *Report* No. FHWA/RD-86/193, Federal Highway Administration, Washington, D.C., 133 pp.

Pfeifer, D. W., and Scali, M. J., 1981, "Concrete Sealers for Protection of Bridge Structures," *NCHRP Report* 244, Transportation Research Board, Washington, D.C., 138 pp.

Portland Cement Association, 1995, "Painting Concrete," *PCA IS134*, Skokie, Ill., 8 pp.

Portland Cement Association, 1996, "Resurfacing Concrete Floors," *PCA IS144*, Skokie, Ill., 8 pp.

Portland Cement Association, 1997, "Effects of Substances on Concrete and Guide to Protective Treatments," *PCA IS001*, Skokie, Ill., 36 pp.

SHRP Project 2033, 1999, "Guideline for Performing Electrochemical Chloride Extraction to Concrete Structures," AASHTO/FHWA, Mar.

Strand, R., 1973, "Earthquake Repairs, Kaiser Hospital, Panorama City, California," *ASCE National Structural Engineering Meeting,* San Francisco, Preprint No. 1996.

Stratton, F. W.; Alexander, R.; and Nolting, W., 1978, "Cracked Structural Concrete Repair Through Epoxy Injection and Rebar Insertion," *Report* FHWA-KS-RD-78-3, Kansas Department of Transportation, Topeka, Kans., 56 pp.

Stratton, F. W.; Alexander, R.; and Nolting, W., 1982, "Development and Implementation of Concrete Girder Repair by Post-Reinforcement," *Report* FHWA-KS-82-1, Kansas Department of Transportation, Topeka, Kans.

Stratton, F. W., and Crumpton, C. F., 1984, "Kansas Bridge Renovated by Post Reinforcement and Thin Bonded Concrete Overlay," *Concrete Construction*, V. 29, No. 8, pp. 705-709.

Toemsmeyer, M. J., 1981, “The Repair of Deteriorated Concrete Beams by Externally Bonded Torsion Reinforcement,” *Transportation Research Record* No.781, Transportation Research Board, Washington, D.C.

Transportation Research Board, 1977, “Rapid-Setting Materials for Patching of Concrete.”

Transportation Research Board, 2001, “Evaluation of Penetrating Corrosion Inhibitor System,” *Ninth Maintenance Management Conference, Conference Proceedings 23*, pg. 127-135.

Troxell, G. E.; Raphael, J. M.; and Davis, R. E., 1958, “Long-Time Creep and Shrinkage Tests of Plain and Reinforced Concrete,” *ASTM Proceedings*, V. 58, pp. 1101-1120.

Virginia Transportation Research Council, 2001, “Evaluation of High Performance Concrete Overlays Placed on Bridges and Pavements in Virginia,” VTRC 01-R1-R3RB.

Warner, J., 1984, “Selecting Repair Materials,” *Concrete Construction*, V. 29, No. 10, pp. 865-871.

Watson, P., 1996, “Preplaced Aggregate Concrete”, *Concrete Repair Digest*, V. 3, No. 3, pp. 148-151.

Whitmore, D. W., 2002, “Impressed Current and Galvanic Discrete Anode Cathodic Protection for Corrosion Protection of Concrete Structures,” *NACE Paper* No. 02263, Corrosion.

Whitmore, D. W., 2004, “New Developments in the Galvanic Cathodic Protection of Concrete Structures,” *NACE Paper* No. 04333, Corrosion 2004, New Orleans, La., Mar. 29.

Wyman, M. R., and Gumina, T. V., 1994, ”Concrete Removal and Surface Preparation for Overlays and Partial Depth Repairs,” *Concrete Repair Bulletin*, Jan.-Feb.