

ACI 546R-14

Guide to Concrete Repair

Reported by ACI Committee 546



American Concrete Institute
Always advancing



American Concrete Institute
Always advancing

First Printing
September 2014
ISBN: 978-0-87031-933-4

Guide to Concrete Repair

Copyright by the American Concrete Institute, Farmington Hills, MI. All rights reserved. This material may not be reproduced or copied, in whole or part, in any printed, mechanical, electronic, film, or other distribution and storage media, without the written consent of ACI.

The technical committees responsible for ACI committee reports and standards strive to avoid ambiguities, omissions, and errors in these documents. In spite of these efforts, the users of ACI documents occasionally find information or requirements that may be subject to more than one interpretation or may be incomplete or incorrect. Users who have suggestions for the improvement of ACI documents are requested to contact ACI via the errata website at <http://concrete.org/Publications/DocumentErrata.aspx>. Proper use of this document includes periodically checking for errata for the most up-to-date revisions.

ACI committee documents are intended for the use of individuals who are competent to evaluate the significance and limitations of its content and recommendations and who will accept responsibility for the application of the material it contains. Individuals who use this publication in any way assume all risk and accept total responsibility for the application and use of this information.

All information in this publication is provided “as is” without warranty of any kind, either express or implied, including but not limited to, the implied warranties of merchantability, fitness for a particular purpose or non-infringement.

ACI and its members disclaim liability for damages of any kind, including any special, indirect, incidental, or consequential damages, including without limitation, lost revenues or lost profits, which may result from the use of this publication.

It is the responsibility of the user of this document to establish health and safety practices appropriate to the specific circumstances involved with its use. ACI does not make any representations with regard to health and safety issues and the use of this document. The user must determine the applicability of all regulatory limitations before applying the document and must comply with all applicable laws and regulations, including but not limited to, United States Occupational Safety and Health Administration (OSHA) health and safety standards.

Participation by governmental representatives in the work of the American Concrete Institute and in the development of Institute standards does not constitute governmental endorsement of ACI or the standards that it develops.

Order information: ACI documents are available in print, by download, on CD-ROM, through electronic subscription, or reprint and may be obtained by contacting ACI.

Most ACI standards and committee reports are gathered together in the annually revised ACI Manual of Concrete Practice (MCP).

American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
Phone: +1.248.848.3700
Fax: +1.248.848.3701

www.concrete.org

Guide to Concrete Repair

Reported by ACI Committee 546

John S. Lund, Chair

David W. Whitmore, Secretary

James Peter Barlow
Michael M. Chehab
Marwan A. Daye
Michael J. Garlich
Paul E. Gaudette
Timothy R. W. Gillespie
Yelena S. Golod
Fred R. Goodwin
Harald G. Greve

Ron Heffron
Robert F. Joyce
Lawrence F. Kahn
Brian F. Keane
Benjamin Lavon
Kenneth M. Lozen
James E. McDonald
Myles A. Murray
Jay H. Paul

Richard C. Reed
Johan L. Silfwerbrand
Joe Solomon
Michael M. Sprinkel
Ronald R. Stankie
Joseph E. Tomes
David A. VanOcker
Alexander M. Vaysburd
Kurt Wagner

Patrick M. Watson
Mark V. Ziegler

Consulting Members
Peter Emmons
Noel P. Mailvaganam
Kevin A. Michols
Richard Montani
Don T. Pyle

This guide presents recommendations for the selection and application of materials and methods for repairing, protecting, and strengthening concrete structures. An overview of materials and methods is presented as a guide for selecting a particular application. References are provided for obtaining in-depth information on the selected materials or methods.

Keywords: anchorage; coating; concrete repair; joint sealant; placement; polymer; protective systems; repair materials; structural strengthening.

CONTENTS

CHAPTER 1—INTRODUCTION, p. 2

- 1.1—Guide use, p. 2
- 1.2—Repair methodology, p. 2
- 1.3—Sustainability, p. 6

CHAPTER 2—DEFINITIONS, p. 6

CHAPTER 3—CONCRETE REMOVAL AND SURFACE PREPARATION, p. 6

- 3.1—Introduction, p. 6

- 3.2—Concrete removal, p. 6
- 3.3—Surface preparation, p. 14
- 3.4—Quality control and assurance, p. 17

CHAPTER 4—REPAIR MATERIALS, p. 17

- 4.1—Introduction, p. 17
- 4.2—Concrete replacements and overlays, p. 17
- 4.3—Crack repair materials, p. 27
- 4.4—Bonding materials, p. 31
- 4.5—Coatings on reinforcement, p. 32
- 4.6—Reinforcement, p. 32
- 4.7—Material selection, p. 33

CHAPTER 5—CONCRETE AND REINFORCEMENT REPAIR TECHNIQUES, p. 34

- 5.1—Introduction, p. 34
- 5.2—Crack repair, p. 34
- 5.3—Concrete replacement, p. 37
- 5.5—Anchorage, p. 44
- 5.6—Quality control and assurance, p. 45

CHAPTER 6—PROTECTIVE SYSTEMS, p. 46

- 6.1—Introduction and selection factors, p. 46
- 6.2—Typical problems that can be mitigated with protection systems, p. 46
- 6.3—Total system concept, p. 47
- 6.4—Surface treatments, p. 48

ACI Committee Reports, Guides, and Commentaries are intended for guidance in planning, designing, executing, and inspecting construction. This document is intended for the use of individuals who are competent to evaluate the significance and limitations of its content and recommendations and who will accept responsibility for the application of the material it contains. The American Concrete Institute disclaims any and all responsibility for the stated principles. The Institute shall not be liable for any loss or damage arising therefrom.

Reference to this document shall not be made in contract documents. If items found in this document are desired by the Architect/Engineer to be a part of the contract documents, they shall be restated in mandatory language for incorporation by the Architect/Engineer.

ACI 546R-14 supersedes 546R-04 and was adopted and published September 2014. Copyright © 2014, American Concrete Institute.

All rights reserved including rights of reproduction and use in any form or by any means, including the making of copies by any photo process, or by electronic or mechanical device, printed, written, or oral, or recording for sound or visual reproduction or for use in any knowledge or retrieval system or device, unless permission in writing is obtained from the copyright proprietors.

6.5—Concrete surface preparation and installation requirements, p. 53

6.6—Joints, p. 54

6.7—Cathodic protection, p. 54

6.8—Chloride extraction, p. 55

6.9—Realkalization, p. 56

CHAPTER 7—STRUCTURAL REPAIR AND STRENGTHENING, p. 56

7.1—General, p. 56

7.2—Internal structural repair (restoration to original member strength), p. 56

7.3—External reinforcement (encased and exposed), p. 57

7.4—External post-tensioning, p. 59

7.5—Jackets and collars, p. 60

7.6—Supplemental members, p. 61

Special considerations for repair of structural elements, p. 61

7.7—Repair of concrete columns, p. 61

7.8—Repair of concrete beams, girders, and joists, p. 63

7.9—Repair of concrete structural slabs, p. 64

CHAPTER 8—REFERENCES, p. 65

Authored documents, p. 69

CHAPTER 1—INTRODUCTION

1.1—Guide use

This document provides guidance on removal and preparation, selecting material and application methods for repair, protection, and strengthening of concrete structures. The information is applicable to repairing deteriorated or damaged concrete structures; correcting design or construction deficiencies; and strengthening the structure for new uses or to comply with current, more restrictive building codes.

Current practices in concrete repair are summarized and information provided for the initial planning of repair work and selecting repair materials and methods for various conditions.

1.2—Repair methodology

The methodology for repairing a concrete structure typically includes a condition assessment of the structure, designing repairs, developing construction documents, bidding and negotiation processes, and performing the repair work. Preparing a maintenance plan for the repaired structure is also recommended. A basic understanding of the causes of concrete distress, deterioration, or deficiencies is essential to performing meaningful evaluations and completing successful repairs (ACI 364.1R). Once the cause of deterioration or deficiency is determined, the appropriate repair program can be selected to address these conditions. Depending on the cause and extent of the damage, repair is not always warranted.

Assessment of the structure should determine the cause of the deterioration or deficiency and not focus only on the symptoms. For example, cracking can be a symptom of distress that may have a variety of causes, such as restraint

of drying shrinkage, restraint of movement due to thermal cycling, overloading, corrosion of embedded metal, or inadequate design or construction. The cause of distress should be assessed for proper selection and implementation of an appropriate repair program (Fig. 1.2).

1.2.1 Condition assessment—The process of repairing a concrete structure starts with the evaluation of existing conditions. The evaluation can be divided into several steps:

- Reviewing available design and construction documents, previous reports, repair/maintenance records, and test data, if available;
- Visually examining the existing structure;
- Performing structural analysis of members in question or the structure in its deteriorated condition;
- Evaluating corrosion activity;
- Performing invasive or nondestructive testing, or both;
- Reviewing physical, chemical, and petrographic analysis results of laboratory-tested concrete samples.

Additional information on conducting condition surveys can be found in ACI 201.1R, 207.3R, 222R, 224.1R, 228.2R, 364.1R, 437R, and 562.

1.2.1.1 Unsafe conditions—During the condition assessment, conditions discovered that pose an immediate safety issue should be identified and reported to the owner for mitigation. Local building codes may require that the licensed design professional (LDP) report unsafe conditions to the authorities and typically require that the owner take measures to protect the public safety where hazardous conditions exist. For example, if loose concrete on overhead or vertical surfaces is discovered, access should be limited in the areas adjacent to and below until the hazards are removed or stabilized. If structural members exhibit compromised integrity, these members should be stabilized or the affected areas removed from service.

1.2.1.2 Global issues—The performance of a structure depends on maintaining the integrity of the structure and envelope of the building. If the LDP becomes aware of an item of concern outside the assigned scope of work that could compromise the integrity of the structure or jeopardize public safety, the appropriate parties should be notified for implementation of remedial action.

1.2.1.3 Determination of cause and extent—During the condition assessment of a structure, the cause of distress, deterioration, or 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 happened to a particular concrete structure. After completing the assessment, a suitable remedial action plan can be developed, repair applications and materials selected, and contract documents prepared. If a delay occurs between the condition survey and performing the repair work, additional deterioration and distress could occur and consideration should be given to updating the condition survey to minimize variations between estimated and actual quantities of repair work.

1.2.2 Design considerations—When designing a concrete repair, strengthening system, or protective system, the LDP should consider the safety and serviceability of the structure

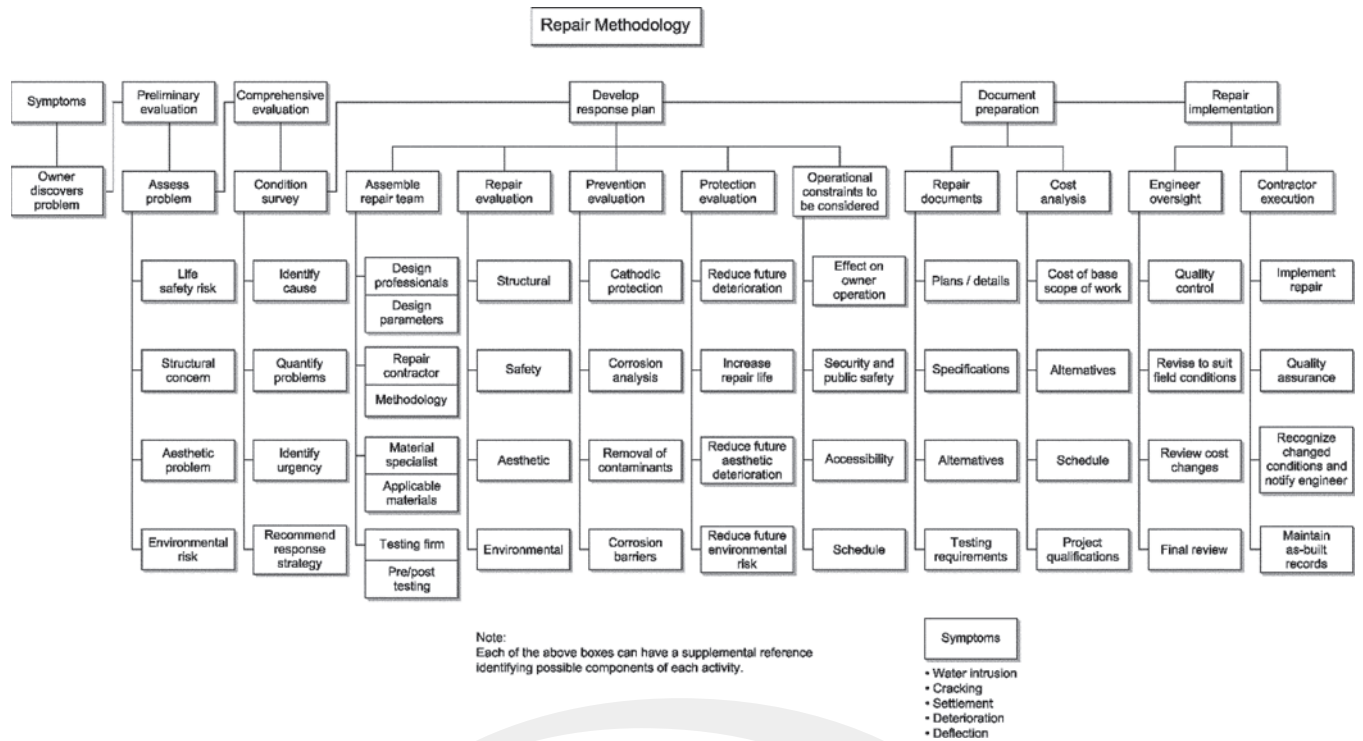


Fig. 1.2—Repair methodology.

during construction and its performance at completion. At a minimum, the repaired structure should satisfy the building code requirements for which it was designed. If required by the governing agency, the repaired structure may have to satisfy current building code requirements and be repaired and strengthened to meet these criteria. In any case, it is the LDP's responsibility to satisfy applicable code requirements for all structural components within the LDP's scope of work. Structural code provisions such as those contained in **ACI 318** may not be directly applicable to the current situation. In such cases, **ACI 562** requirements should be followed. The LDP should apply basic principles of structural mechanics and have an understanding of material behavior to evaluate and design a structural repair, a strengthening procedure, or a protective system. Several design considerations are discussed 1.2.2.1 through 1.2.2.6.

1.2.2.1 Current load distribution—In a deteriorated condition, a structural member or system may distribute dead and live loads differently than when the structure was undamaged. Cracking, deteriorated concrete, and corroded reinforcement can alter the behavior of members, leading to changes in shear, moment, and axial load distribution. As concrete and reinforcement are removed and replaced during the repair operation, these redistributed forces may be further modified. To understand the final behavior of the structural system, the engineer should evaluate the redistribution of the forces. To reestablish the original load distribution, a member can be relieved of the load by jacking or other means before repair implementation. If the structure is not jacked and the dead load is not relieved, the repaired and adjacent members may support loads differently than was assumed in the original design of the structure.

1.2.2.2 Compatibility of materials—If a repair and the existing substrate materials have the same stiffness or modulus of elasticity, the behavior of the repaired member may be assumed to be the same as the original member before deterioration or damage. Conversely, if the stiffnesses differ, then the composite nature of the repaired system should be considered. A mismatch of other material characteristics further exacerbates the effects of thermal change, vibration, long-term creep, and shrinkage. If the coefficient of thermal expansion of the repair material differs from that of the original material, stresses will be generated in both the repair and original material by temperature changes.

1.2.2.3 Creep and shrinkage—Reduction in length, area, or volume of both the repair and original materials due to creep, shrinkage, or both, affect the structure's 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. Controlled-shrinkage cementitious repair materials and systems can contribute to the reduction of the volume change effects.

1.2.2.4 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.

1.2.2.5 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 a repair design consideration.

1.2.2.6 Material behavior characteristics—After a repair is executed, the structural behavior of the repaired section

can differ from the behavior of the original section and is dependent on the characteristics of the repair materials. For example, if a beam's steel reinforcement has corroded extensively and has lost part of its tensile load-carrying capacity, the lost portion of the reinforcement may be replaced by external strengthening techniques (Chapter 7), such as fiber-reinforced polymer (FRP) applied externally to the tension face of the beam, following repair or replacement of the deteriorated section caused by steel corrosion. The LDP should consider the behavior and performance of the new repair under the expected service and ultimate loads, and design the repair to provide a level of safety that is at least equivalent to the original design. Such a design is outside the scope of ACI 318 and ACI 562 should be used.

1.2.3 Selecting repair methods and materials—Successful concrete repair depends on the selection and proper application of the materials and methods to be used. ACI 546.3R provides specific guidelines for the properties of repair materials and their selection and ACI 546.2R covers underwater repair materials. Repair materials and methods should be selected with the following considerations.

1.2.3.1 Concrete environment—Repair work can incorporate adjustments or modifications to remedy the cause of the concrete deterioration, such as reducing exposure to moisture, changing the water drainage pattern, eliminating sources of erosion and water infiltration, providing for differential movements, or eliminating exposure to deleterious substances. However, it is not always possible to correct causes of deterioration. For example, it may not be possible to change the environmental exposure of a concrete structure. In such cases, reasonable efforts should be made to mitigate the problem by providing appropriate protection measures.

1.2.3.2 Limiting constraints—Repair work may be affected by constraints such as limited access to the structure or areas of repair. For example, underwater concrete deterioration provides access challenges. Other constraints may include the operating schedule of the structure, weather, and limitations imposed by the owner of the structure, including repair budget.

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

1.2.3.4 Environmental and safety requirements—These items may have a major impact on the selection of materials, methods, or both. Environmental and occupational safety constraints are governed by specific requirements and policies of owners: the Environmental Protection Agency (EPA); the Occupational, Safety, and Health Administration (OSHA); and local regulations. Refer to International Concrete Repair Institute (ICRI) 120.1 for information on further safety aspects relevant to the concrete repair industry.

1.2.3.5 Service life—The intended service life of the repaired member and structure should be considered in the selection of repair materials and methods.

1.2.3.6 Constructibility—Availability of repair materials, equipment and methods, and the technical feasibility of using them should be evaluated during the material and repair procedure selection process. Manufacturers or suppliers can provide assistance in the selection, availability, and cost of repair materials and for selecting application techniques. When selecting the appropriate repair material, consider that the technical data presented in manufacturer's literature may be incomplete and insufficient because the tests performed may not be representative of the use of the material in a particular application.

1.2.4 Temporary shoring—The repair process, including concrete removal and reinforcement 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 need, extent, and locations of shoring and bracing required as well as the sequence of repair to satisfy structural safety and force distribution considerations.

The need for temporary supports, shoring, bracing, and strengthening to perform the repairs should be determined before performing repair work. Concrete removal often exposes reinforcing steel that may have suffered loss of cross-sectional area; concrete removal around reinforcing steel reduces the bond between the reinforcement and the structural element, thereby reducing the shear, bending, tensile, and compression capacity or stability of the structure. Shoring and bracing may be required to address these temporary conditions.

If structural analysis is performed, the analysis should assess the effects of existing conditions and proposed repairs on the structure, including the evaluation of applicable loads and dimensional changes resulting from thermal differentials. Areas of special concern that may result in conditions that require temporary shoring and bracing include redistribution of negative moments in slabs and beams, joint and connection details, precast spandrel beams and columns, as well as live loads imposed by repair equipment and material storage.

In most cases, unless the dead and live loads are relieved via shoring before the repair work proceeds, the repaired portion of the element may not be effective in supporting loads applied after the repair installation is completed. In cases where restoration of the load-carrying capacity is a primary concern, existing loads may need to be removed and members jacked and shored to support dead loads.

1.2.5 Contract documents—The preparation of contract documents includes drawings, details, and specifications. Contract documents should include provisions specific to the project and quality assurance criteria. Because the full extent of concrete deterioration and distress may not be completely known until repair work begins, drawings and specifications for repair projects should be prepared with some flexibility regarding work items such as concrete removal, surface preparation, reinforcement replacement, and quantities of repair materials. Provisions should be included in the documents for addressing hidden conditions and potential changes to the contract estimate of repair quantities.

Where 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. Structural components and their supports in the immediate area of the repairs should be evaluated to determine if shoring, bracing, or both are required. Redistribution of member internal forces during repair is an important consideration for continuous slabs and beam-and-girder systems, and is critical for unbonded post-tensioned structures. Repair specifications should be clear and concise, delineating the scope of work, repair work items, information on existing concrete conditions if available, material and equipment requirements, surface preparation, installation procedures, quality assurance and testing criteria, and performance testing standards. Supporting documents referencing specific requirements such as tensile strength, surface profiling, compressive strength, bond strength, length change, or shrinkage behavior should be included as appropriate. Details of the concrete repair should be provided, including parameters for concrete removal and, if possible, boundaries of concrete removal and replacement along with any special features of repair system installation necessary, such as protection systems and strengthening techniques. Special attention is required for the details of reinforcement repair or replacement areas and the preparation of existing concrete before surface protection system application.

1.2.5.1 Contract types—To perform repairs, the contractor or design/build team typically enters into a contractual agreement with the owner. This contract defines the scope of work, price, and schedule. Payment for contractual requirements can be stated in lump sum or fixed-price terms, unit prices, time and material, or a combination thereof. Refer to ICRI 130.1R for further information on contract types.

1.2.5.1.1 Lump sum—This contract type is suitable for a specific, well-defined scope of work in which all repair items, quantities of work, and work conditions are known. The contractor is paid a fixed fee for completed work, regardless of the actual repair items and quantity of work performed.

1.2.5.1.2 Unit price—This contract type is suitable for a specific scope of work where all repair items are known but exact quantities are unknown. The final contract sum for the repair items covered by unit prices is based on actual quantities of the repair items properly installed by the contractor. Some individual repair items may be paid on a lump sum or time and material basis.

1.2.5.1.3 Time and material—This contract type is suitable for an undefined scope of work where both the repair items and quantities of work are undefined. The contractor is paid at a defined hourly rate for labor and reimbursed for materials and equipment with applicable overhead and profit.

1.2.5.2 Performance objectives—Repair documents should define the structural, environmental, aesthetic, and protection objectives to be achieved. Mockups are recommended where appropriate; for example, where the color or

texture is specified to match the surrounding concrete (ACI 311.1R, 311.4R, and 311.5).

1.2.6 Bid or negotiation process—Selecting a contractor is one of the most important aspects of a repair project. Contractor selection can be determined through a bidding process or negotiated directly with potential contractors. Contractor selection should be based on demonstrated evidence of expertise and successful previous experience with repairs included in the project. Manufacturer-approved installers should be considered for specialized repair products.

1.2.6.1 Prebid conference—Before bids are submitted, the engineer, owner or owner’s representative, and potential contractors should attend a prebid conference to answer questions regarding the contract documents and to define expectations of the owner. This meeting should include a visit to the repair site.

1.2.6.2 Addenda—Clarifications to the contract documents based on questions raised at the prebid conference, during the bid period, or both, can be issued in addenda to the contractors before submission of bids.

1.2.7 Execution of work—Repair work should be executed in accordance with the contract documents. The concrete repair process generally consists of removing deteriorated or damaged concrete; surface preparation; repairing, supplementing, or replacing reinforcing steel; implementing specified repair techniques; and installing repair materials. Protecting the executed repairs and strengthening may also be incorporated into the repair process.

1.2.7.1 Preconstruction conference—Before starting a project, it is important to have the owner or owner’s representative and LDP attend a meeting with the contractor’s project manager, superintendent, and foreman to discuss the parameters, means, methods, and materials necessary to achieve the repair objectives. At this meeting, the contractor should present a construction schedule for the owner to review its acceptability and resolve any conflicts with daily operations. The frequency and types of required correspondence should be defined among all parties executing the work to communicate progress, discovered items, and construction problems.

1.2.7.2 Material submittals and shop drawings—Prior to use on the project, the contractor should submit technical, installation, and safety data for proposed repair materials in accordance with the contract documents to the LDP for review and acceptance. In addition, shop drawings for items such as shoring and bracing, reinforcing steel, specific repair methods, and repair sequencing should also be submitted by the contractor to the LDP for review and acceptance prior to initiation of the work.

1.2.7.3 Quality assurance and control—Quality assurance and control measures should be implemented to provide acceptance criteria, verify conformance to contract documents, and to verify the properties and quality of the installed repair materials. ACI 311.4R provides guidance on setup of quality control, inspection, and testing procedures. ACI 311.1R and ACI 311.5 provide guidance on inspection and testing.

1.2.7.3.1 Quality assurance—The LDP should outline quality assurance requirements in the contract documents by

establishing standards or criteria for acceptance. The quality assurance program dictates the desired level of quality for repairs.

1.2.7.3.2 Quality control, inspection, and testing—Successful completion of the repair program can be verified by implementing quality control measures during the course of the work. Such measures include:

- a) Periodic site observations by the LDP to verify general conformance with contract documents;
- b) Periodic testing by an independent testing agency to verify that requirements of the contract documents are met;
- c) Contractor supervision of the crew's work and weather conditions, including temperature, humidity, and precipitation, which may affect the repair procedure;
- d) Recordkeeping by the project team, including documentation of repair locations, repair modifications, quantities installed, documentation reviews by the LDP, weather conditions, changes to the contract documents, and periodic progress reports.

1.2.7.3.3 Testing and inspection agency qualifications—The testing agency should meet the requirements of **ASTM E329**. Inspection or testing personnel should be special inspectors, engineers, the LDP, or ACI-certified, or approved equal, field/laboratory technicians, as applicable.

1.2.7.4 Project closeout—If requested by the owner and upon completion of the project, the owner should receive and retain a project manual from the contractor or LDP containing copies of relevant contract correspondence issued during construction. The project manual should include items such as repair drawings and specifications, additional details or clarifications provided by the engineer, field and test reports, photographs, submittals, permits, change orders, requests for information (RFIs) and their responses, insurance certificates, bonds, pay applications, warranties, waivers of lien, and as-built drawings.

1.2.8 Maintenance after completion of repairs—Maintenance of the concrete structure and any protection or strengthening system is recommended after completion of a concrete repair project. Lack of adequate maintenance may result in premature failure of the repair or surrounding areas. If requested by the owner, the LDP should develop a maintenance manual for repair work.

1.2.8.1 Warranty coverage—Warranty requirements should be followed, including periodic inspections or recoating of membranes and sealers to ensure that warranty coverage is maintained. Warranty items should be reviewed during the warranty period and the contractor contacted to repair failures that occur under warranty.

1.2.8.2 Periodic inspections—Continued evaluation of the structure through periodic engineering reviews should be considered to monitor structural integrity and performance of repairs.

1.2.8.3 Post-repair monitoring systems—Systems to detect concrete/system failures should be considered, such as corrosion monitoring for detecting corrosion and acoustic monitoring for detecting tendon failures, in post-tensioned structures.

1.3—Sustainability

A green structure is an environmentally sustainable structure designed, constructed, and maintained to minimize its impact on the environment (**Schokker 2010**). The primary objectives when designing a green structure include reducing energy consumption, conserving water, and recycling waste. Repairing and maintaining concrete structures instead of replacement is a green concept and a sustainable philosophy. Repairing concrete structures meets sustainability objectives by conserving existing materials and thereby reducing energy consumption and material waste.

CHAPTER 2—DEFINITIONS

ACI provides a comprehensive list of definitions through an online resource, "ACI Concrete Terminology," <http://www.concrete.org/Tools/ConcreteTerminology.aspx>. Additional concrete repair definitions are found in the International Concrete Repair Institute (ICRI) terminology database, <http://www.icri.org/GENERAL/repairterminology.aspx>.

CHAPTER 3—CONCRETE REMOVAL AND SURFACE PREPARATION

3.1—Introduction

This chapter covers the removal of deteriorated and sound concrete, and preparation of the concrete surface to receive new repair materials. Exercising care during the removal and preparation phases of a repair project can be the most important factor in ensuring the longevity of the repair, regardless of the material or technique used. Concrete should be removed carefully so as not to damage the existing adjacent concrete or affect bond of the new repair material.

Care should be taken when removing concrete around prestressing steel, both bonded and unbonded strands. When high-energy impact tools such as chipping hammers are used, operators should avoid contacting the strands with the tools, as such contact may cause wires to rupture, possibly leading to strand failure and a reduction in the structure's load capacity. Removing surface concrete may also cause excessive stresses in the concrete to remain and redistribute secondary forces and moments in indeterminate structures. The extent of concrete removal should not be permitted to exceed stress limits in the remaining concrete.

Personal protective equipment should be used in all processes and the containment, handling, and disposal of concrete dust, slurry, and debris considered with the processes employed. Refer to **ICRI 120.1** for recommendations on safety in the concrete repair industry to minimize vibrations and noise.

3.2—Concrete removal

A repair project typically involves removing deteriorated, damaged, or defective concrete. In most cases, however, determining the extent of the deteriorated or damaged concrete requiring removal can be difficult. Common techniques to identify distressed concrete are sounding by hammer tapping or by chain drag (**ASTM D4580**). Deep delaminations and microcracks may not be detected by

hammer sounding or chain dragging. Other techniques include impact-echo testing (**ASTM C1383**), ground-penetrating radar (GPR) (**ASTM D6087**), and infrared thermography (**ASTM D4788**), typically accompanied by sounding by hammer tapping, chain drag, or impulse response. Refer to **ACI 228.2R** and **ICRI 210.4** for additional information and other test methods used to identify deteriorated and damaged concrete.

Removing concrete using aggressive methods can damage concrete that is intended to remain in place. For example, blasting with explosives or the use of heavy impact equipment can result in additional concrete delamination or cracking. Impact equipment with a blunt-tipped removal apparatus can cause more delamination and cracking than sharp-tipped tools. Removing concrete using impact tools may also result in microcracking or bruising (**Hindo 1990**) to the surface concrete left in place. Unless this damaged layer is removed, the substrate concrete will be weakened, possibly resulting in reduced bond strength between the substrate concrete and repair material. Thus, removing all damaged or delaminated concrete, including bruising, at the interface of the repair and the substrate concrete is an important part of achieving a strong bond and composite action between the substrate concrete and the repair material. If all deteriorated and damaged concrete is not removed, then the possibility of repair failure is increased, even if a high-quality repair material is used.

In most cases in which concrete has been removed from a structure by primary means, such as impact methods, the concrete left in place should be prepared by a secondary method, such as abrasive blasting or high-pressure water jetting, to remove any remaining damaged and microcracked surface material. Careful visual inspection and sounding of prepared surfaces should be conducted and loose, delaminated, or unsound concrete should be removed before placing repair materials. The suitability of the substrate to receive the repair material can be verified by tensile pull-off testing of trial repairs (**ICRI 210.3**; **ASTM C1583**).

Localized removal of deteriorated concrete in a member includes squaring the edges of the repair area by saw cutting or chipping square to avoid feather edging the repair material, removal of delaminated and unsound concrete, and removal of sound concrete as necessary to provide a minimum thickness of repair material throughout the repair area. Feather edging of repair materials should be avoided. For shotcrete repairs, the edges of the removal areas should taper toward the center of the removal area to avoid acute angles that can make shotcrete placement more difficult (**ACI 506R**). Reentrant corners in the perimeter of removal areas should be avoided. Large variations in the depth of removal within short distances should also be avoided, as they may result in cracking due to varying behavior of the repair materials with different thicknesses.

Unless otherwise directed by the project specifications, existing reinforcement should not be cut. Reviewing drawings and using a cover meter or similar device to obtain data as to the location and depth of existing reinforcement are good practice. In addition, removing deteriorated and

damaged concrete is commonly performed before saw cutting to determine the location and depth of reinforcement.

3.2.1 General considerations—Concrete repair requires removal of deteriorated and damaged material. Removing sound concrete, however, may also be necessary to ensure that all unsound or contaminated material is removed, facilitate structural modifications, provide adequate repair geometry and configuration, and provide development and encapsulation of reinforcing steel and embedded items. Some methods may be more effective in sound concrete, whereas others may work better for deteriorated and damaged concrete.

Concrete removal techniques should be effective, safe for the structure and the construction workers, economical, suited to the requirements of the owner, 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 for the repair. Some techniques may permit a portion of the repair 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 existing concrete. Field testing mockups of selected removal techniques can help determine the best procedures to use for the repair.

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 effective removal method subject to the engineer's acceptance. In some instances, however, the engineer may need to specify the removal techniques necessary for the repair and those prohibited due to constraints and logistics of the structure.

The size, quantity, and location of reinforcing steel; size and type of aggregates; concrete compressive strength modulus of elasticity as well as the presence of existing coatings, toppings, overlays; and other existing or as-built conditions are important information to consider in determining the effective method(s) and cost of concrete removal. Whenever possible, this information should be made available to contractors during bidding. This information is also beneficial to the engineer when they specify concrete removal and surface preparation criteria and for select repair materials.

3.2.2 Temporary shoring—It is essential to evaluate the removal operations and their impact on the structure's integrity throughout the repair process and to limit the extent of damage to the structure and the existing concrete that remains. Before removing concrete, the need for temporary supports, shoring, bracing, and strengthening should be assessed (Fig. 3.2.2a). Structural elements to be repaired may require temporary shoring, removal of applied loads, or both, to prevent damaging structural deformations, buckling or movement of reinforcement, or possible collapse. A structural analysis may need to be performed to assess the effects of existing conditions and proposed repairs on the structural integrity of the building and the need for temporary shoring.



Fig. 3.2.2a—Temporary shoring parking garage floor (left) and balcony slabs (right).

Care should be used during removal of concrete to avoid cutting and damaging existing reinforcing steel (Fig. 3.2.2b). Because embedded reinforcement may not be located in the specified position and is not visible before concrete removal, unanticipated damage can occur when saw cutting and removing concrete. This damage, as well as excess concrete removal, could affect structural integrity during repairs and the need for temporary shoring.

3.2.3 Quantity of concrete to be removed—In most repair projects, all damaged or deteriorated concrete should be removed because deterioration continues with time. The anticipated quantity of concrete removal is directly related to the extent of the distress, repair or removal technique, and elapsed time between the initial survey and repair construction; potential quantity overruns may be realized between preparation of the quantity estimates and actual removal. Estimating inaccuracies can be minimized by a thorough condition survey performed as close as practical to the time the repair work is executed. When, by necessity, the condition survey is performed far in advance of the repair work, estimated removal 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 upon completion of the work. This is typically done jointly by the engineer and the contractor. It is not uncommon for final measured quantities to exceed the initial estimated quantities. Refer to **ICRI 130.1R** for guidelines for methods of measurement for concrete repair work.

3.2.3.1 Anodic ring (halo) effect—Accelerated corrosion occurs where reinforcement located in existing contaminated concrete extends into repair area containing uncontaminated repair material due to the difference in electrical potential between the contaminated and uncontaminated concrete volumes (6.2.5). In general, concrete removal should continue until no further delamination, cracking, or significant corrosion is observed along the reinforcing steel. Additional measures should be considered to minimize future concrete damage due to corrosion if chloride contamination or carbonated concrete will remain beyond the repair area. Concrete excavation that damages the bond of the reinforcing steel to the existing concrete requires additional concrete removal and undercutting of the reinforcing steel.



Fig. 3.2.2b—Saw cutting perimeter of repair area.

3.2.3.2 Removal of concrete surrounding reinforcing steel—Concrete removal should include the removal of deteriorated and damaged concrete surrounding existing reinforcing steel. Care should be used to prevent further damage to the reinforcing or prestressing steel caused by the process of removing the concrete. For this reason, a cover meter or reinforcing bar locator, along with structural drawings if available, should be used to help determine the depth, size, quantity, and approximate location of the existing reinforcement within the repair area.

Once the selected concrete has been removed within the repair area, a less invasive removal method, such as light (15 lb [67 N]) chipping hammers should be used to remove the concrete around exposed reinforcement. Care should be taken to not damage the reinforcement or otherwise cause loss of reinforcing bond to the existing remaining concrete. If the existing reinforcing steel is only partially exposed after concrete removal, it may not be necessary to remove additional concrete to expose the full circumference of the reinforcement, provided the reinforcement is uncorroded, has not lost bond to the existing concrete; the existing concrete is sound and not chloride-contaminated (**AASHTO T 260**, **ASTM C1152**, and **ASTM C1218**); and the pH (**ASTM F710**) has not been affected by carbonation (6.2.4) or acid attack. When the exposed reinforcing steel has loose rust,

corrosion products, is not well bonded to the surrounding concrete, or if the concrete surrounding the reinforcing steel is chloride-contaminated, concrete removal should continue around the reinforcing steel to create a minimum of 0.75 in. (19 mm) clearance to the surrounding concrete or 0.25 in. (6 mm) larger than the coarsest aggregate in the repair material. Undercutting the exposed reinforcing steel provides clearance around the reinforcing steel for cleaning, complete bonding of the repair material to the existing concrete, and structurally secures the repair (ICRI 310.1R). Pay special attention when removing concrete around both bonded and unbonded prestressed steel strands. Excessive concrete removal can lead to changes in strand profile, reduce forces in the strand, and compromise the integrity of the structure.

3.2.3.3 Preparation of repair perimeter—Following the removal of unsound concrete, the perimeter of the repair area should be saw cut to create a simple symmetrical configuration, preferably square or rectangular with square corners. Saw cut to a depth of 0.50 to 0.75 in. (13 to 19 mm) at the repair perimeter to provide a vertical edge for consolidating the repair material. Saw cutting to a lesser depth may be necessary at locations where the reinforcing steel has reduced concrete cover. A perimeter saw cut is often unnecessary with concrete removal methods such as high-pressure water jets. Refer to the manufacturer's recommendations for the selected repair material. Following saw cutting, some removal of sound concrete may be required to achieve the repair perimeter. Care should be taken to avoid abrupt changes in depth within the repair area, as the repair material is more susceptible to cracking at these locations.

3.2.4 Concrete removal methods—Removal and excavation methods can be categorized by the way in which the process acts on the concrete. Common processes are cutting, impacting, and high-pressure water jets (hydrodemolition). Other less-common methods of concrete removal are also available, including blasting and presplitting methods, but are typically only used for large-scale concrete removal. Table 3.2.4 provides a general description of these processes, lists the specific removal techniques within each category, and provides a summary of information on each technique. Refer to ICRI 310.1R, 310.2R and Society for Protective Coating (SSPC)-SP13/NACE 6 for additional information on concrete removal.

3.2.4.1 Cutting methods—Cutting methods generally employ mechanical sawing or high-pressure water jets to cut around the perimeter of concrete sections to permit their removal. The size of the sections 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, and stitch drilling.

3.2.4.1.1 High-pressure water jet without abrasives—A narrow high-pressure water jet driven at high velocities, commonly producing pressures of 10,000 to 40,000 psi (69 to 276 MPa) to cut and remove concrete and prepare concrete surfaces. Section 3.2.4.3 describes using a water jet as a concrete removal method. High-pressure water jets without abrasives will remove concrete but will not cut or damage the reinforcing steel. The addition of abrasives to

the water jet can result in cutting of the concrete and the reinforcing steel.

3.2.4.1.2 Saw cutting—Diamond or carbide saws are available in sizes ranging from small and capable of being hand-held, to large and capable of cutting depths of approximately 5 ft (1.5 m) to substantially larger depths.

3.2.4.1.3 Diamond wire cutting—Diamond wire cutting is accomplished with a wire containing nodules (small lumps) impregnated with diamonds. The wire is wrapped around the concrete mass to be cut and connected with a 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 determine the size of the concrete structure that can be cut. This system provides an efficient method for cutting and dismantling large or small concrete structures.

3.2.4.1.4 Mechanical shearing—The mechanical shearing method uses hydraulically powered jaws to cut or crush concrete and shear the reinforcing steel. This method is applicable for making cutouts through slabs, decks, and other thin concrete members and especially applicable where total demolition of the member is desired. The major limitation of this method is that cuts need to start from free edges or from holes made by hand-held breakers or other means.

3.2.4.1.5 Stitch drilling—The stitch drilling method uses overlapping boreholes along the removal perimeter to cut out sections. This method is applicable for making cutouts in concrete members where access to only one face is possible, and the depth of cut is greater than can be economically cut by the saw cutting and diamond wire cutting methods. 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.

3.2.4.2 Impact methods—Impact methods are the most commonly used concrete removal techniques. Impact methods involve striking a concrete surface repeatedly with a high-energy tool with hardened points or large mass to fracture and spall the concrete. Using impact methods for concrete removal can result in microcracking of the surface concrete left in place and creating a weakened plane below the bond line of the repair material. Impact methods also transmit vibrations through the structure and reinforcing steel, which may cause further damage and loss of bond between the reinforcing steel and existing concrete. Factors such as the weight, size, and power of the impact removal equipment should be considered, balancing the desire for productive removal while minimizing damage and microcracking. Secondary methods, such as abrasive blasting and water blasting, should be considered to remove microcracking following the use of impact methods. Tensile pull-off testing in accordance with ICRI 210.3 or ASTM C1583 is recommended to determine the suitability of the final prepared surface to receive repair materials and coatings.

3.2.4.2.1 Hand-held breakers—The hand-held breaker or chipping hammer is probably the most common and best known of all concrete removal devices. Hand-held breakers

Table 3.2.4—Summary of features, considerations, and limitations for concrete removal methods

Category	Features	Considerations/limitations
Water-jet cutting (uses perimeter cuts to remove large pieces of concrete)	a) High-pressure water jet without abrasives b) Applicable for making cutouts through slabs, decks, and other thin concrete members c) Cuts irregular and curved shapes d) Makes cutouts without overcutting corners e) Cuts flush with intersecting surfaces f) No heat, vibration, or dust is produced g) Handling of debris is efficient because bulk of concrete is removed in large pieces	a) Cutouts for removal limited to thin sections b) Cutting is typically slower and more costly than diamond blade sawing c) Moderate levels of noise may be produced d) Controlling flow of waste water may be required e) Additional safety precautions are required due to the high water pressure produced by the system f) Does not cut reinforcing steel
Saw cutting (applicable for making cutouts through slabs, decks, and other thin concrete members)	a) Makes precision cuts b) No vibration is produced c) Handling of debris is efficient because bulk of concrete is removed in large pieces	a) Cutouts for removal limited to thin sections b) Performance is affected by type of diamonds and the diamond-to-metal bond in blade segments (segment selection is based on aggregate hardness) c) The higher the percentage of steel reinforcement in cuts, the slower and more costly the cutting d) The harder the aggregate, the slower and more costly the cutting e) Controlling flow of waste water may be required
Diamond wire cutting (applicable for cutting large, thick pieces of concrete)	a) The diamond wire chain can be any length b) No dust or vibration is produced c) Large blocks of concrete can be lifted out by a crane or other mechanical methods d) The cutting operation can be efficient in any direction	a) The cutting chain must be continuous b) Access to drill holes through the concrete must be available or access around the full section of the structure if the full section is to be cut c) Water must be available to the chain d) Controlling the flow of waste water may be required e) The harder the aggregate or concrete, and the higher the percentage of steel, the slower and more costly the cutting f) Performance is affected by the quality, type, and number of diamonds as well as the diamond-to-metal bond in the chain.
Mechanical shearing (applicable for making cutouts through slabs, decks, and other thin concrete members)	a) Steel reinforcement can be cut b) Limited noise and vibration are produced c) Handling of debris is efficient because bulk of concrete is removed in large pieces	a) Limited to thin sections where an edge is available or a hole can be made to start the cut b) Exposed reinforcing steel is damaged beyond reuse c) Remaining concrete is damaged d) Extremely rugged profile is produced at the cut edge e) Ragged feather edges remain after removal
Stitch drilling (applicable for making cutouts through concrete members where access to only one face is feasible)	a) Handling of debris is more efficient because bulk of concrete is removed in large pieces	a) Rotary-percussion drilling is more expedient and economical than diamond core drilling; however, it may result in more damage to the concrete that remains, especially at the point of exit from the concrete b) 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 c) Uncut portions between adjacent boreholes will hamper or prevent the removal d) 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. e) Personnel must wear hearing protection due to high noise levels.
Impacting (uses repeated striking of the surface with a mass to fracture and spall the concrete)	a) Hand-held breakers b) Applicable for limited volumes of concrete removal c) Applicable where blow energy must be limited d) Widely available commercially e) Can be used in areas of limited work space f) Produces relatively small and easily handled debris	a) Performance is a function of concrete soundness and aggregate toughness b) Significant loss of productivity occurs when breaking action is other than downward c) Removal boundaries will likely require saw cutting to avoid feathered edges d) Concrete that remains may be damaged (microcracking) e) Produces high levels of noise, dust, and vibration
Boom-mounted breakers (applicable for full-depth removal of slabs, decks, and other thin concrete members and for surface removal from more massive concrete structures)	a) Can be used for vertical and overhead surfaces b) Widely available commercially c) Produces easily handled debris	a) Blow energy delivered to the concrete may have to be limited to protect the structure being repaired and the surrounding structures from damage b) Performance is a function of concrete soundness and aggregate toughness c) Damages remaining concrete d) Damages reinforcing steel e) Produces feathered edges f) Produces high level of noise and dust

Table 3.2.4 (cont.)—Summary of features, considerations, and limitations for concrete removal methods

Category	Features	Considerations/limitations
Scabblers	a) Low initial cost b) Can be operated by unskilled labor c) Can be used in areas of limited work space d) Removes deteriorated concrete from wall or floor surfaces efficiently e) Readily available commercially	a) High cyclic energy applied to a structure will produce fractures in the surface of the remaining concrete b) Produces high level of noise and dust c) Limited depth removal
Needle scaler (applicable for limited volumes of concrete removal)	a) Can be used in areas of limited work space b) Can be used for vertical and overhead surfaces c) Produces relatively small and easily handled debris	a) Limited concrete removal depth b) Concrete that remains can be damaged (microcracking) c) Produces feathered edges
Scarifier/milling (applicable for removing deteriorated concrete surfaces from slabs, decks, and mass concrete)	a) Boom-mounted cutters are applicable for removal from wall and ceiling surfaces b) Removal profile can be controlled c) Method produces relatively small and easily handled debris	a) Removal is limited to concrete without steel reinforcement b) Sound concrete reduces the rate of removal c) Can damage concrete that remains (microcracking) d) Noise, vibration, and dust are produced
Erosion/expansive pressure (hydrodemolition) (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)	a) Does not damage the concrete that remains b) Steel reinforcement is left clean and undamaged for reuse c) Method produces easily handled aggregate-sized debris	a) Productivity is reduced when sound concrete is being removed b) Removal profile will vary with changes in depth of deterioration c) Method requires large source of potable water to meet water demand d) Waste water may have to be controlled e) A permit may be required if waste water is to enter a waterway f) Personnel must wear hearing protection due to the high level of noise produced g) Flying debris is produced h) Additional safety requirements are required due to the high pressures produced by these systems
Blasting—explosives (uses rapidly expanding gas confined within a series of boreholes to produce controlled fracture and removal of concrete)	a) Most expedient method for removing large volumes where concrete section is 10 in. (250 mm) thick or more b) Produces good fragmentation of concrete debris for easy removal	a) Requires highly skilled personnel for design and execution b) Stringent safety regulations must be complied with regarding the transportation, storage, and use of explosives due to their inherent dangers c) Blast energy must be controlled to avoid damage to surrounding structures resulting from air-blast pressure, ground vibration, and flying debris
Presplitting (uses hydraulic jacks, water pulses, or expansive agents in a pattern of boreholes to presplit and fracture the concrete to facilitate removal)	a) Large sections can be presplit for removal, thereby making handling of debris more efficient b) Development of presplitting plane in direction of boreholes depth is limited c) Development of presplitting plane is decreased by presence of reinforcing steel normal to presplitting plane d) Presplit opening must be wide enough to allow cutting of steel reinforcement e) Secondary means of breakage may be required to complete removal a) Loss of control of presplitting plane can result if boreholes are too far apart or if holes are located in severely deteriorated concrete	a) Development of presplitting plane is decreased by presence of reinforcing steel normal to presplitting plane b) Presplit opening must be wide enough to allow cutting of steel reinforcement c) Secondary means of breakage may be required to complete removal d) 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)	a) Typically less costly than cutting members b) Direction of presplitting can be controlled by orientation of wedges and drill hole layout c) Can be used in areas of limited access d) Limited skills required by operator e) No vibration, noise, or fly rock is produced except by the drilling of boreholes and secondary breakage method	
Water pulse splitter	a) Economical, portable, rugged, and easy to use and maintain b) Devices have self-contained power sources c) Negligible vibration	a) Requires boreholes at close intervals to control crack propagation b) Control of crack plane depth is limited c) Not applicable to vertical surfaces d) Produces some noise e) Drill holes must hold water
Expansive agents (applicable where 9 in. [230 mm] or more of a concrete face is to be removed)	a) Can be used to produce vertical splitting planes of significant depth b) No vibration, noise, or flying debris is produced other than that produced by the drilling of boreholes and secondary breakage method	a) Best used in gravity-filled vertical or near-vertical holes b) Agents of putty consistency are available for use in horizontal or overhead holes c) Development of presplitting plane is decreased by presence of reinforcing steel normal to presplitting plane



Fig. 3.2.4.2.1—Hand-held breakers.

are available in various sizes with different levels of energy and efficiency (Fig. 3.2.4.2.1). These tools are generally defined by weight and vary in size from 8 to 90 lb (3.5 to 41 kg). Note that the larger the hammer, the greater the potential for microcracking. Smaller hand-held breakers are commonly used in partial depth removal of sound and unsound concrete and concrete removal around reinforcing steel because they minimize damage to the existing concrete and reinforcing steel. Larger breakers are used for removal of large volumes of concrete. Exercise care when selecting the size of breakers to minimize damage to the structure and to avoid shattering or breaking through the remaining section. Sharp-pointed tools also tend to reduce the potential for microcracking of the surface concrete left in place, as do lightweight breakers used at a shallower angle to the removal substrate.

3.2.4.2.2 Boom-mounted breakers—The boom-mounted breaker is similar to the hand-held breaker except that it is mechanically operated and considerably larger than a hand-held breaker (Fig. 3.2.4.2.2). Boom-mounted breakers differ from hand-held breakers as they function on the principle of high energy and low frequency rather than low energy and high frequency, and are driven by compressed air or hydraulic pressure. The reach of the hydraulic arm enables the breaker to be used on vertical or overhead surfaces at a considerable distance above and below the machine level. The boom-mounted breaker is a highly productive means of removing concrete; however, the high-cycle impact energy delivered to a structure by the breaker generates forces that may damage the existing concrete and reinforcing steel and adversely affect the integrity of the structure.

3.2.4.2.3 Scabblers—Scabblers are best known for their ability to remove shallow depths of concrete from a surface. Scabblers impact the concrete with cutting heads to create a chipping and pulverizing action. Scabbler heads are available in various sizes and geometrical shapes, and varying numbers of heads may be mounted on the cylinders. Equipment size depends on the number of cylinders. The equipment is normally driven by compressed air. A scabbler is designed to remove deteriorated or sound concrete from



Fig. 3.2.4.2.2—Boom-mounted breaker.

horizontal, vertical, and overhead surfaces to relatively shallow depths (1/8 to 3/4 in. [3 to 19 mm]). Scabblers produce a very irregular surface with a fractured coarse aggregate profile and typically cause microcracking of the concrete surface. Scabblers can also remove brittle coatings and provide deep profiling in preparation for overlays and other repair materials.

3.2.4.2.4 Needle scaler—Needle scaling uses the pointed tips of a bundle of steel rods contained by a steel tube to impact and remove concrete to shallow depths (1/16 to 1/8 in. [1.5 to 3 mm]) at horizontal, vertical, and overhead surfaces. This method is typically used at edges and areas inaccessible by larger equipment. Needle scaling creates the potential for microcracking of the concrete surface. Needle scalars are also used for removing existing coatings and preparation of surfaces for coatings and thin overlays.

3.2.4.2.5 Scarifier—A scarifier is a concrete removal device that uses rotary action and several cutter bits mounted on a drum to fracture and pulverize the concrete from a surface. Concrete removal depths typically range from 1/8 to 3/4 in. (3 to 19 mm) and may require multiple passes with the equipment. Scarifying can remove deteriorated and sound concrete in which some of the concrete contains form ties and wire mesh. Scarifiers are available in a range of sizes for removing deteriorated concrete on horizontal, vertical, and overhead surfaces. Advantages include well-defined limits of concrete removal, relatively small and easily handled concrete debris, and simplicity of operation. Scarifiers may cause microcracking in the concrete. Scarifiers are also used for the removal of brittle coatings and adhesives, and profiling of concrete surfaces in preparation for high-build coatings, overlays, and other repair materials.

3.2.4.2.6 Milling methods—Milling methods can remove a specified amount of concrete from large areas of horizontal or vertical surfaces. The removal depth typically ranges from 1/4 to 4 in. (6 to 100 mm). Removal depth is determined by the number and size of teeth. Milling operations typically leave a sound surface with fewer microfractures than impact methods (Virginia Transportation Research

Council 2001). Tensile pull-off testing is recommended on surfaces prepared by milling to determine the suitability of the surface to receive repair materials.

3.2.4.3 Hydrodemolition—High-pressure water jetting (hydrodemolition) can be used to remove sound and unsound concrete on horizontal, vertical, and overhead surfaces without damaging the existing reinforcing steel and without causing microcracking of the remaining in-place concrete. Hydrodemolition disintegrates concrete and leaves a saturated, rough, irregular surface profile capable of providing an excellent composite bond with the repair material (Fig. 3.2.4.3). The method is very efficient at removing unsound concrete over large areas of the concrete surface cleaning the reinforcing steel in the process by abrasive action of the water and the pulverized concrete. The existing concrete surface is ready for placement of the repair material, provided the surface is thoroughly cleaned and rinsed following hydrodemolition and any ponded water is removed. Hydrodemolition uses high-pressure pumps capable of generating pressures from 10,000 to 40,000 psi (69 to 276 MPa) with water flow rates between 6 to 100 gpm (25 and 380 lpm) (ICRI 310.3). The high-pressure water jet may be applied using a robotic removal unit. The robotic unit moves the water jet uniformly over the concrete to be removed. Hydrodemolition can also remove deteriorated concrete that exists below the specified removal depth. This is called selective removal and it is used to help ensure that all unsound concrete has been removed. The water jet does not transmit vibration into the concrete and will not cause further damage to the existing structure. Hydrodemolition is a wet process that minimizes dust from the work area and uses large quantities of water that must be collected and disposed of properly. Wastewater contains suspended particles and typically has a pH of 11 to 12.5. The wastewater must be filtered and treated to reduce the particulates and the alkalinity reduced to a pH typically between 5 and 10 before discharge. Hydrodemolition should not be used in post-tensioned structures with unbonded tendons, except under the direct supervision of a structural engineer.

Water jet hand lances operating at pressures of 10,000 to 40,000 psi (69 to 276 MPa) with water flows of 2 to 12 gpm (8 to 45 lpm) are available to remove concrete in areas difficult to access with larger equipment (ICRI 310.3; SHRP-S-336).

Hydrodemolition is also used to remove coatings and waterproofing membrane systems. This removal is generally accomplished with a multiple-jet nozzle instead of the single-jet nozzle (ICRI 310.3; ACI RAP-14). Hydrodemolition is also used in surface preparation for overlays.

Typically, this method is not used for small-scale repairs primarily for reasons of practicality, containment, and equipment mobilization. Additionally, large volumes of water are required for this method as well as the subsequent water treatment and disposal.

3.2.4.4 Blasting methods—Blasting methods generally use rapidly expanding gas confined within a series of bore holes to produce controlled fractures and removal of the concrete.

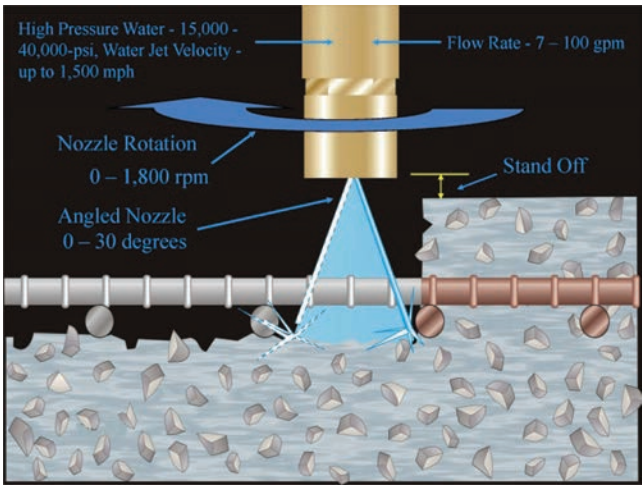


Fig. 3.2.4.3—Hydrodemolition.

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. An example is 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 3 in. (75 mm) diameter or smaller bore holes parallel to the removal face, loading each hole with light charges of explosive (typically a 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 use proper timing sequences to provide greater control in reducing ground vibration. Controlled blasting has been used successfully on numerous repair projects. Selecting the 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 licensed personnel with proven experience and ability.

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

3.2.4.5.1 Hydraulic splitter—A hydraulic splitter is a wedging device used in predrilled bore holes 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.

3.2.4.5.2 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.

3.2.4.5.3 Expansive product agents—Commercially available cementitious expansive products, 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 depth greater than 12 in. (300 mm). 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.

3.3—Surface preparation

Following concrete removal, the most important step in the repair of a concrete structure is the surface preparation and cleanliness of the repair area. Regardless of the nature, sophistication, or cost 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 and concrete surface to develop a good bond with the repair material. This section discusses the surface preparation of concrete and reinforcing steel for a repair. Project documents should include requirements for inspection of substrates to confirm adequate completion of preparation measures prior to placement of the repair materials.

3.3.1 General considerations—Surface preparation consists of the final steps necessary to prepare the concrete surface to receive repair materials. The appropriate preparation of the concrete surface depends on the procedures used to remove the concrete and on the type of repair to be undertaken.

Most of the concrete removal methods described in 3.2 can also be used for surface preparation. An effective method for concrete removal, however, may not be effective or appropriate for a specific surface preparation requirement. For example, some concrete removal methods leave the concrete surface too rough or irregular for the intended repair material

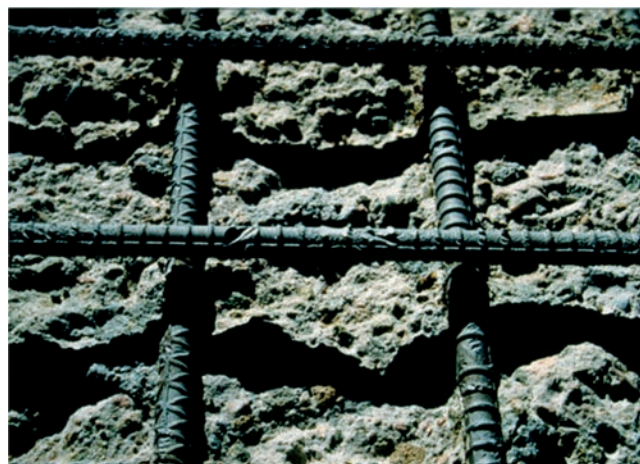


Fig. 3.3.1—Concrete surface profile.

such as membrane coatings. In these cases, removal methods specifically intended for the final surface preparation may be required. **ICRI 310.2R** provides a concrete surface profile (CSP) system for determining the surface profile necessary for various repair applications (Fig. 3.3.1). Sandpaper grit size is also commonly used as a means to characterize surface profile (**ASTM D7682**).

Some concrete removal methods damage or weaken the remaining concrete surface. This may be critical if structural bonding of the repair material is important. For example, microcracking of the concrete surface occurs when using certain removal methods. Left in place, microcracking often produces a weakened plane in the underlying concrete substrate, below the bond line. In this case, a less invasive and aggressive method of final surface preparation, such as abrasive or water blasting, may be necessary to remove the damaged surface layer.

In many repair situations, the proposed repair requires only surface profiling, 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 concrete removal methods described in 3.2 can be used for this type of surface preparation. The referenced methods offer a wide range of surface characteristics and results. For example, the required surface preparation may vary from a light abrasive cleaning suitable for the application of a coating, to a deeper roughening and surface profiling necessary for good bond and reliable performance of the repair material. Selecting a suitable method of surface preparation is extremely important, as it influences the cost and performance of the repair.

3.3.2 Surface preparation methods—Common surface preparation techniques include abrasive blasting, shot-blasting, waterblasting, and cleaning methods. Table 3.3.2 provides a general description of surface preparation techniques. Refer to ICRI 310.2R for additional information on these methods.

3.3.2.1 Abrasive blasting—Abrasive blasting removes and cleans concrete by propelling an abrasive medium in compressed air or high-pressure water at high velocity

Table 3.3.2—Summary of features and considerations/limitations for surface preparation methods

Category	Features	Considerations/limitations
Abrasive blasting— Sandblasting (dry/wet) (uses equipment that propels an abra- sive medium at high velocity at the concrete to abrade the surface)	a) Efficient method for roughening the surface and exposing aggregate b) Cleans reinforcing steel c) Removes surface contamination	a) Dry sandblasting procedure produces large volumes of dust b) Wet sandblasting is slow
Low-pressure water blasting (with abrasives)	a) Selectively removes defective concrete b) Precise control of removal process c) Cleans reinforcing steel while removing concrete d) Produces minimal damage to remaining concrete e) Produces no heat or dust f) Abrasives enable jet to cut steel reinforcement and hard aggregates	a) High initial investment b) Additional protection and safety procedures may be required due to water pressure c) Controlling flow of contaminated waste water may be required
Abrasive blasting—Shotblasting	a) Efficient method for roughening the surface and exposing aggregate b) Low dust emissions c) Removes surface contaminants d) Controlled depth of concrete removal e) Readily available commercially	a) Large units may produce high noise levels b) High voltage power requirements
Chemical cleaning	a) Application of chemicals to remove dirt and contaminants from surface b) Additional surface preparation may be neces- sary to remove cleaning chemical residue c) Follow cleaning chemical manufacturer’s recommendations d) Follow coating manufacturer’s instructions	a) Additional surface preparation may be necessary b) May not be effective in removing all contaminants c) May have strong chemical odor d) Protection of adjacent area necessary
Acid etching	a) Effective in removing cement-based materials b) Increases porosity of concrete surface	a) Protective clothing is necessary b) Large amounts of water necessary c) May permanently stain concrete d) Protection of adjacent area necessary e) May contaminate and damage remaining concrete surface
Low-pressure water cleaning (without abrasives)	a) Effectively removes surface contaminants without chemical residue b) Minimal protection of adjacent areas necessary	a) Control of large amounts of water b) Removal of small amounts of concrete may be noticeable

against the concrete surface to abrade and pulverize it. Abrasive blasting is commonly used to clean reinforcing steel and clean and profile concrete surfaces in preparation for sealers, coatings, and overlays. The method can also remove surface contaminants, brittle coatings, and adhesive films. The method can be used on horizontal, vertical, and overhead surfaces. Depth of concrete removal depends on the duration of the blasting as well as the size and cutting efficiency of the abrasive medium. Commonly used methods include sandblasting, shotblasting, and high-pressure water blasting.

3.3.2.1.1 Sandblasting—Sandblasting is the most commonly used abrasive blasting method in the construction industry (Fig. 3.3.2.1.1). The process uses common sands, silica sands, metallic sands, granular slags, or other media for the abrasive medium. The process may be executed by three techniques (3.3.2.1.1.1 through 3.3.2.1.1.3).

3.3.2.1.1.1 Dry sandblasting—In dry sandblasting, abra-
sive particles are propelled at concrete and steel surfaces in
a stream of compressed air at a minimum pressure of 100 psi
(0.7 MPa). The compressor size will vary depending on the
size of the sandblasting pot. The abrasive particles are typi-
cally angular and may range in size from passing a No. 70
to a No. 4 (212 to 4.75 mm) sieve. The rougher the required
surface condition, the larger the abrasive particle size
necessary. Finer sands are used for removing contaminants

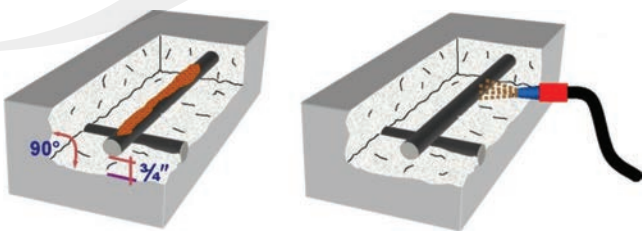


Fig. 3.3.2.1.1—Sandblasting.

and laitance from the concrete and loose scale from rein-
forcing 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 rein-
forcing steel. Although abrasive blasting has the ability to
remove concrete, it is not economically practical to remove
more than 0.25 in. (6 mm) from the concrete surface. Dry
abrasive blasting will produce large quantities of airborne
particulates, such as dust, and precautions should be taken to
protect personnel and equipment in the work area.

3.3.2.1.1.2 Wet sandblasting—In 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 reduces the amount of airborne partic-



Fig. 3.3.2.1.2—Shotblasting.

ulates, some of the water intercepts the sand being projected at the concrete surface and reduces the efficiency of the abrasive blasting operation to effectively remove concrete and provide surface profile. This process is generally limited to cleaning a concrete surface or reinforcing steel.

3.3.2.1.1.3 Low-pressure water blasting with abrasives—Low-pressure water blasting with abrasives is a cleaning system using a stream of water at low pressure (1500 to 5000 psi [10 to 35 MPa]) 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 and leaving the concrete surface clean and free of dust.

This method eliminates airborne particles that normally occur when using dry/wet abrasive blasting procedures. The water used is collected and the abrasive removed before the water is discharged into a wastewater system.

3.3.2.1.2 Shotblasting—Shotblasting equipment cleans and removes concrete by projecting steel shot at the concrete surface at high velocity within an enclosed blast chamber that recovers and separates most of the dust and reusable shot (Fig. 3.3.2.1.2). The shot erodes and pulverizes the concrete at the surface. The depth of removal is controlled by shot size, machine setup, and rate of travel. 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 profile required determines the size of the shot and the speed at which the machine will travel. A surface cleaning operation is achieved by using smaller (0.017 to 0.028 in. [0.43 to 0.71 mm] diameter) shot and setting the machine for maximum travel speed. Concrete removal up to 1/4 in. (6 mm) is possible in a single pass by increasing the size of the shot (0.033 to 0.055 in. [0.84 to 1.4 mm] diameter) and reducing the travel speed. Shotblasting can provide CSP ranging from CSP2 to CSP8 (ICRI 310.2R), making

this method suitable for roughening horizontal surfaces in preparation for receiving sealers, coatings, and overlays. It is also useful for removing existing coatings, adhesives, and surface contaminants. Hand-held equipment is also available for use on vertical surfaces.

3.3.2.2 Cleaning methods—Common methods of cleaning concrete surfaces are discussed in the following.

3.3.2.2.1 Chemical cleaning—In many 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 sealers and coatings under certain conditions, however, it may be possible to use detergents, trisodium phosphate, and various proprietary concrete cleaners to remove dirt, oil, grease, buildup, and oxidation deposits from concrete surfaces (ASTM D4258). 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 may dissolve the contaminant and carry it deeper into the concrete.

3.3.2.2.2 Acid etching (ASTM D4259 and D4260)—Acid etching of concrete surfaces has long been used to remove laitance and surface concrete. The acid removes cement paste and provides a slight surface profile by exposing fine aggregate. This lightly profiled surface promotes penetration and adhesion of sealers and coatings, especially thin-film coatings. Acid should only be used when no alternative means of surface preparation can be used. 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 can weaken the remaining cement paste on the concrete surface. Care should be taken to neutralize residual acid on prepared surfaces (ASTM D4262) and in wastewater. Thorough removal of etching debris is required to eliminate bond-inhibiting contaminants on the concrete surface. As such, large quantities of rinse water, mechanical scrubbing, and vacuum removal are necessary.

3.3.2.2.3 Low-pressure water cleaning without abrasives—Water is sprayed to remove dirt and loose, friable material. Using pressures less than 5000 psi (35 MPa), this method does not remove any significant amount of concrete. It can be used outdoors or indoors where mist and water can be tolerated. This method can be used in horizontal, vertical, or overhead situations. However, it generally does not produce any significant increase in texture, profile, or pattern in the concrete and therefore should be a supplement to other methods in most surface preparation applications (ICRI 310.2R).

3.3.3 Cleaning reinforcing steel—All exposed surfaces of the reinforcement should be thoroughly cleaned of all loose concrete, rust, oil, and other contaminants. Sandblasting is the preferred method. The degree of cleaning depends on the repair procedure and material selected. For limited areas, wire brushing or other hand methods of cleaning may be acceptable. When cleaning the steel and removing loose particles from 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 should be used. High-pressure water blasting is also effective in cleaning the surface and reinforcing steel. When using moisture-sensitive repair materials, it will be necessary to remove any excess water before application of repair material. Saw-cut edges of the repair area should also be abrasive-blasted to roughen the saw cut vertical surface. Refer to 5.4 for reinforcement repair and 6.7.4 and 6.7.5 for installation of embedded galvanic anodes and cathodic protection systems to reinforcing steel.

3.3.4 Final cleaning of repair area—Upon completion of surface preparation and cleaning of the reinforcing steel, the surface should be clean, dry, and free of microcracking and bruising. All loose concrete, aggregate, rust, oil, and other contaminants should be removed from the concrete surface and reinforcing steel. This may require further sand, water, or air blasting; vacuuming; or other techniques.

3.4—Quality control and assurance

Quality control and assurance procedures are required throughout the concrete removal and surface preparation operations. These can be accomplished through visual inspection, soundings, and use of a covermeter or other means to locate reinforcement. Sounding is an excellent means to identify delaminated and debonded concrete; however, identification of subsurface cracks, microcracking, and the extent of deterioration or other internal defects may not be determined by these methods alone. Other means of evaluation should be used to properly identify the extent of deteriorated and damaged concrete and to verify its complete removal in the repair process.

Various evaluation methods (ACI 228.2R and ICRI 210.4) can provide valuable information about the condition of the concrete repair before, during, and after concrete removal. Some test methods require concrete core samples from the existing concrete or repair material. More common test methods and evaluations performed during the repair process are outlined:

- 1) Methods for concrete strength
 - a) Cores and cylinders—compressive, tensile, modulus of elasticity, and splitting tensile strength
 - b) Pullout (ASTM C900)
 - c) Pulse velocity (ASTM C597)
- 2) Reinforcing steel location
 - a) Covermeter
 - b) Ground-penetrating radar (GPR)
 - c) X-ray
- 3) Delamination, debonding identification, and microcracking (can be identified by petrographic analysis)
 - a) Visual examination
 - b) Acoustic impact (ASTM D4580—includes chain drag/hammer sounding, electromechanical sounding, and rotary percussion methods)
 - c) Impact-echo (ASTM C1383)
 - d) Infrared thermography (ASTM D4788)
 - e) Ground-penetrating radar (GPR) (ASTM D6087)
- 4) Surface cleanliness and strength (ACI 515.1R)
 - a) Test for pH of cleaned surfaces (ASTM D4262)

- b) Surface cleaning concrete for coatings (ASTM D4258)
 - c) Testing by tape lift (ASTM E1216)
- 5) Measuring surface profile (ICRI 310.2R)
- a) Replica putty (ASTM D7682)
 - b) Replica tape (ASTM D4417)
 - c) Sand method (ASTM E965)
 - d) Laser profilimetry
- 6) Tensile pull-off testing (ASTM C1583, ICRI 210.3)

CHAPTER 4—REPAIR MATERIALS

4.1—Introduction

This chapter describes the various categories of materials 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. General guidance on selecting repair materials is also provided. For more information on selecting repair materials, refer to ACI 546.3R.

4.2—Concrete replacements and overlays

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

4.2.1 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, reduce drying shrinkage, reduce the potential for corrosion of steel reinforcement, reduce expansion of alkali-aggregate reaction, or alter other properties of the concrete (ACI 212.3R). Pozzolanic materials such as fly ash, silica fume, granulated blast-furnace slag or other supplementary cementitious materials 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 be designed to provide the workability, density, strength, and durability necessary for the particular application (ACI 211.1). To minimize shrinkage cracking, the repair concrete should have a water-cementitious materials ratio (*w/cm*) as low as possible and a coarse aggregate content as high as possible. When proper restraint is provided, shrinkage-compensating concrete also contributes to the reduction of drying shrinkage cracking (ACI 223R). Frost-resistant normalweight concrete should have a *w/cm* not exceeding 0.45 for thin sections, such as 1.5 to 3 in. (40 to 75 mm) and 0.50 for greater thicknesses (ACI 201.2R). Mixing, transporting, and placing of conventional concrete should follow the guidance given in ACI 301, ACI 304R, ACI 304.1R, ACI 304.2R, ACI 304.5R, and ACI 304.6R.

4.2.1.1 Advantages—Conventional concrete is readily available, well understood, economical, has similar properties to the existing concrete, and is relatively easy to produce, place, finish, and cure. Generally, concrete mixtures can be proportioned to match the hardened properties of the underlying concrete, making conventional concrete applicable in a wide range of repairs.

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

4.2.1.2 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 an appropriate design mixture for the exposure condition is used or a reduced service life is acceptable. For example, if the original deterioration was caused by acid attack, aggressive water attack, or abrasion-erosion, repair with conventional concrete may not be successful unless the cause of deterioration is mitigated.

The shrinkage properties of the repair materials are critical because the new repair materials are being placed on an existing substrate that has likely undergone essentially all the shrinkage that it will experience. Curing requirements, as well as the required properties of the repair material, including strength, modulus of elasticity, and limitations on maximum shrinkage strain, if needed, should be addressed by the licensed design professional (LDP) in the repair specifications.

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

4.2.1.3 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 2 in. (50 mm), especially where the overlay extends beyond the reinforcement, or where the repair area is large. Conventional concrete is 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 et al. 1958).

4.2.1.4 Standards—ASTM C94 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 addressed in this specification and should be specified separately.

4.2.2 Conventional mortar—Conventional mortar is a mixture of portland cement, fine aggregate, and water.

Water-reducing admixtures, supplementary cementitious materials, expansive components, and other modifiers are often used with conventional mortar to minimize shrinkage and adjust other properties.

4.2.2.1 Advantages—The advantages of conventional mortar are similar to those of conventional concrete, except that mortar can be placed in thinner sections. A wide variety of prepackaged mortars are available. They are particularly appropriate for small repairs.

4.2.2.2 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. Proper air entrainment, which is approximately 4 to 6 percent, is often required to provide adequate resistance to freezing and thawing as well as scale resistance; however, air content does reduce the strength of the mortar. Prepackaged mortars typically include air-entraining admixtures (Types IA, IIA, or IIIA).

4.2.2.3 Applications—Conventional mortar should not be used in many situations when thin repair sections are required. These applications should be reviewed when installed in the path of vehicular traffic that is subject to repetitive loading. Because field conditions affect the development of the performance of mortar, testing for relevant properties should be performed to closely simulate the application conditions.

4.2.2.4 Standards—ASTM C387 covers the production, properties, packaging, and testing of packaged, dry, combined materials for mortar and concrete. Special consideration should be given to properties not covered in this specification, such as shrinkage and durability.

4.2.3 Proprietary concrete and mortars—Proprietary repair mortars are prepackaged mortars that are a blend of portland cement or specialty cement; fine aggregate; and admixtures such as plasticizers, expansive components, densifiers, accelerators, polymers, thixotropic agents, and other additives. Depending on the specific use, mortars may contain variable aggregate gradations for different placement depths and consistencies. Proprietary repair mortars are also available that are trowelable, sprayable, and specifically intended for overhead or vertical applications. Some proprietary repair mortars can be extended with coarse aggregate for deeper application sections.

4.2.3.1 Advantages—The advantages of proprietary repair materials are convenience of use and the broad range of products with different physical and mechanical properties to meet different conditions, such as partial-depth repairs on vertical and overhead surfaces without the need of forms as well as form-and-pour applications. They often have faster set times and shorter cure times when compared with conventional concrete and mortar. They are suitable for performance-based requirements, as the composition typically is formulated to achieve published values. The preblended materials used in proprietary mortars are convenient for small quantity placements where the proportioning of conventional mortars is inconvenient; difficult to disperse;

or too complex for field conditions, such as the quantity of material used, rapid hardening, or accessibility.

4.2.3.2 Limitations—Proprietary repair mortars may have different mechanical properties than the concrete being repaired. In addition, the substrate concrete has likely already experienced most of its shrinkage and property development. Because proprietary mortars may be cement-rich and use a complex blend of modifiers, they can have a propensity for high shrinkage compared with conventional concrete. Shrinkage characteristics can be provided by the material manufacturer. Although there are no industry standards specifying bond strength of these mortars, International Concrete Repair Institute (ICRI) 210.3 or ASTM C1583 tensile bond testing is appropriate for evaluating in-place bond.

4.2.3.3 Applications—Some proprietary mortars are recommended for repairs as thin as 1/8 in. (3 mm). Thin mortar applications should use special products developed for this purpose, especially when installed in the path of vehicular traffic or subjected to repetitive loadings. Testing should be performed under actual field conditions to ensure the satisfactory performance of the material and the installation.

4.2.3.4 Standards—ASTM C928, the standard specification for packaged, dry, rapid-hardening cementitious materials for concrete repairs, cautions that chloride levels greater than 1 lb/yd³ (600 g/m³) of the hardened repair material is indicative that the packaged material contains sufficient chlorides to cause corrosion-to-steel reinforcement when the concrete is exposed to weather, is on the ground, or is in a moist environment (ACI 222R). Much lower chloride ion content should be used in prestressed concrete. The allowable drying shrinkage is up to 0.15 percent at 28 days according to ASTM C157, although this much shrinkage or additional drying shrinkage will likely compromise the integrity of the repair. This standard is not all-inclusive, and compliance to ASTM C928 is no assurance of the required suitability or durability of the repair material. The U.S. Army Corps of Engineers (USACE) UFGS-03 01 32 guide specifications (USACE 2009) limit unrestrained drying shrinkage to 0.04 and 0.10 percent at 28 days and ultimate, respectively, for repair of civil works-type structures. Limits on restrained shrinkage are also specified. ICRI 320.3R and ACI 364.3R provide guidance for appropriate tests to determine properties and describe characteristics of proprietary repair mortars. Information on criteria for selecting proprietary repair mortars can be found in ACI 546.3R.

4.2.4 Cement grouts—Cement grouts are commonly mixtures of hydraulic cement, fine aggregates, and water that produce a plastic, flowable or fluid consistency grout. Some grout formulations do not include all of these components or provide alternate components to achieve the same type of final consistency, workability, and strength. Admixtures are frequently included in the grout to reduce segregation of the contents, reduce mixing water, accelerate or retard time of setting, minimize shrinkage, reduce or eliminate bleed, improve pumpability or workability, or improve the durability of the grout. Fly ash may be used to enhance perfor-

mance or reduce cost. Silica fume may be used to increase chemical resistance, density, durability, and strength while decreasing permeability. Prepackaged grout compliant with ASTM C1107 may not be appropriate for some types of repairs that do not provide restraint or provide proper curing and should never be modified without the manufacturer's approval.

4.2.4.1 Advantages—Cement grouts are economical, readily available, easy to install, and closely compatible with concrete. Admixtures can modify cement grouts to meet specific job requirements. Shrinkage-reducing admixtures and expansive components are available to minimize shrinkage effects. Non-shrink grouts are also available.

4.2.4.2 Limitations—Cement grouts can be used for repairs by injection, but only where the width of the opening is sufficient to accept the solid particles suspended in the grout. The minimum crack width at the point of introduction should be approximately 0.25 in. (6 mm).

4.2.4.3 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. Non-shrink cement grouts can be used to repair spalled or honeycombed concrete or to install anchor bolts in hardened concrete, but should be used in accordance with the manufacturer's instructions.

4.2.4.4 Standards—The U.S. Army Corps of Engineers CRD C621-93 (USACE 1993) covers three grades of packaged, dry, hydraulic-cement grouts (non-shrink) intended for use under applied load, such as to support a structure or a machine, where shrinkage is to be minimized.

4.2.5 Polymer-portland-cement concrete and mortar—Polymer-portland-cement concrete and mortar (PPCC) and latex-modified concrete (LMC) are identified as portland cement and aggregate combined, at the time of mixing, with organic polymers dispersed or redispersed in water. This dispersion is called a latex, where 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.

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 PPCC, 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 commonly used latexes include polymers and copolymers of vinyl acetate with ethylene (VAE) or other vinyl monomers such as vinyl versatate.

When emulsified and mixed with concrete, epoxies provide excellent resistance to freezing and thawing, reduced permeability, and improved chemical resistance. The bond of epoxies is excellent whereas flexural, compressive, and tensile strengths can be high. Epoxy emulsions, however, have had limited use in concrete due to the desire to use repair materials with physical properties similar to the substrate.

Mixture proportions depend on the specific application and the type of polymer used in the PPCC. Levels of 10 to 20 percent polymer solids by mass of cement are required for most applications. Typical *w/cm* for workable PPCC 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). While latex polymers are used almost exclusively for PPCC overlays, redispersible polymer powders have been used as well. However, these polymer powders are used more commonly in repair situations.

4.2.5.1 Advantages—Polymer-portland cement concrete 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. Latex-modified concrete 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 Transportation Research Council 2001).

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

Styrene-butadiene PPCC has excellent durability for exterior exposures or environments where moisture is present.

4.2.5.2 Limitations—Polymer-portland-cement concrete should be placed and cured at 45 to 85°F (7 to 30°C) with special precautions taken when either extreme is reached (ACI 548.1R; ACI 306.1).

Handling and finishing PPCC is limited to less than 30 minutes; therefore, mobile volumetric mixers fitted with an additional storage tank for the latex should be used for large applications of LMC. When small batches are mixed in drum or mortar mixers, the mixing time should be limited to 3 minutes. Longer mixing times result in an increase in total air content with subsequent reductions in compressive strength. Latex-modified concrete has a tendency for plastic shrinkage cracking during field placement. Special precautions are necessary when the evaporation rate exceeds 0.1 lb/ft²/h (0.5 kg/m²/h) (ACI 308R; 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. Surface discoloration occurs

when the concrete is exposed to UV light. Where such discoloration is not acceptable, acrylic polymers should be used.

4.2.5.3 Applications—Polymer-portland-cement concrete applications include overlays of bridge decks, parking structures, and decks, as well as repair 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 are particularly suitable in exterior white cement applications where color retention is important.

Polymer-portland-cement concrete is most commonly used for overlays and is normally applied in sections ranging from 1.5 to 4 in. (40 to 100 mm) thick for concrete and 0.75 to 1.5 in. (20 to 40 mm) for mortars. These systems restore lost sections and provide a new high-strength wearing surface that is very durable against weathering and can be placed into service when it has developed sufficient strength, which is dependent upon the hydration of the cement.

Although mainly used as overlay materials, PPCC is also an effective repair material. Typically, PPCC repairs are relatively shallow; therefore, mixture proportions similar to those shown in ACI 548.3R should be considered.

4.2.5.4 Standards—ASTM C685 covers concrete made from materials continuously batched by volume and mixed in a volumetric 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 C1438 covers latex and redispersible polymer powder modifiers, and ASTM C1439 covers test methods for PPCC. ACI 548.1R includes a bibliography of major references covering polymers in concrete and a glossary of terms.

4.2.6 Polymer concrete and mortar—Polymer concrete and mortar (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.

Polymer concrete and mortar has been made with a variety of resins and monomers, including polyester, epoxy, furan, vinylester, high-molecular-weight methacrylates (HMWM), and styrene (ACI 548.1R). Polyester resins are desirable because of moderate cost, availability of a great variety of formulations, and relatively 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, and some formulations bond to damp surfaces without requiring a primer. ACI 503R provides information on the use of epoxy compounds with concrete. A special form of PC is sulfur concrete, which is placed hot and is thermoplastic. For further information, refer to ACI 548.2R.

The properties of PC depend 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

resistance to freezing and thawing; low permeability to water and aggressive solutions; and good chemical resistance.

4.2.6.1 Advantages—PC can provide a fast-curing, high-strength repair 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 formulations, 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. Polymer mortars that are trowelable, specifically intended for overhead or vertical applications, or both, are also available.

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 for thin repairs and overlays less than 0.75 in. (20 mm).

4.2.6.2 Limitations—Organic solvents are required to clean equipment when using polyesters and some epoxies. Manufacturers should be consulted before using proprietary materials to address the issue of volatile organic compound (VOC) restrictions. Epoxy materials are also sensitizers capable of causing allergic reactions and other health hazards. The disposal of epoxy materials that are unreacted may also be classified as hazardous waste in some areas and should therefore be done in accordance with local ordinances.

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 minutes to more than 1 hour. Ambient and concrete temperatures may affect polymer viscosity, cure time, or performance.

The coefficients of thermal expansion of polymer materials vary from one product to another and are 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 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, damp, or moist 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 450°F (230°C) and can 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.

4.2.6.3 Applications—PC repair 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, provided consideration is given to the modulus, creep, thermal, and other properties of a specific product.

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

Polymer mortars have been used in a variety of repairs where only thin sections, such as 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 1 ft² (0.1 m²) and greater than 15 ft (4.5 m) in length (**Project Profile 1993**).

4.2.6.4 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 C881**. It includes requirements for adhesive labeling, storage, handling, mixing and application, surface evaluation and preparation, and inspection and quality control.

ASTM C881 covers two-component, epoxy-resin bonding systems for application to PCC, which are able to cure under humid conditions and bond to damp surfaces.

4.2.7 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 cement 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.

4.2.7.1 Advantages—Magnesium phosphate concrete and mortar is typically classified as ultra-rapid hardening (Type URH) cement. This classification requires a minimum compressive strength of 3000 psi (21 MPa) at 1.5 hours and a maximum 28-day drying shrinkage of 0.07 percent (**ASTM C1600**). Retarded versions with extended setting times of 45 to 60 minutes at room temperature are also available. Scaling 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 C672**. 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°F (0°C), or lower if the mixing water and material are heated. Magnesium phosphate mortars typically achieve 6000 psi (40 MPa) compressive strength within 48 hours and do not increase much beyond

that level. The material has good bond strength to portland cement and low permeability. Thin repairs with magnesium phosphate mortar often have improved performance when compared with portland cement repairs because they generally do not require a moist cure.

4.2.7.2 Limitations—Magnesium phosphate concrete and mortar 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. Magnesium phosphate concrete and mortar reacts chemically with the dust of the fracture, sawing residue, portland cement contamination, or carbonated zone in the concrete 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 to be applied over it. The MPC and membrane manufacturers should be contacted for recommendations.

Because of the short time between initial and final setting times, MPC is generally 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 percent. 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. Typically, the mortar should not be placed at temperatures above 80°F (27°C) or in the sunlight within the temperature ranges of 60 to 80°F (15 to 27°C). However, hot and cold weather formulas are available and may have different temperature ranges. The manufacturer should be consulted for temperature ranges and when placing the material in thickness greater than 4 in. (100 mm) or in insulated cavities where the heat generated cannot escape. Extending the mixture with aggregates or using cool water can slow down the exothermic reaction.

As it hardens, MPC quickly produces high strength and a high modulus of elasticity. Therefore, MPC lacks the flexibility and 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 180°F (80°C) in service, strength reductions can develop, but its compressive strength can be expected to exceed that of normal concrete.

4.2.7.3 Applications—Repair applications are the most common use of MPC, and are cost-effective for rapid repairs where a short downtime 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 reac-

tion, heating the materials and the substrates is not typically necessary unless the temperature is below 32°F (0°C). Magnesium phosphate concrete and mortar is useful for cold-weather embedments and anchoring because of its high bond strength and low shrinkage rate.

4.2.8 Supplementary cementitious materials—In addition to the information provided in this section, [ACI 232.1R](#), [ACI 232.2R](#), and [ACI 234R](#) may also be consulted.

4.2.8.1 Silica-fume concrete—Silica fume ([ASTM C1240](#)), categorized as a supplementary cementitious material, is a very fine noncrystalline silica and by-product in the manufacture of silicon and ferrosilicon alloys. It is an efficient pozzolanic material that enhances mechanical and durability properties of concrete. Silica fume is normally added to concrete in quantities of 5 to 15 percent by weight of cement, reducing the amount of cement used in a given mixture. Silica fume is available in several forms, as a slurry, densified (compacted), and pelletized.

4.2.8.1.1 Advantages—In general, the addition of silica-fume to concrete increases compressive and bond strength, decreases permeability, and produces a concrete that is more resistant to abrasion and attack from sodium sulfate and some chemicals ([ACI 234R](#)). When used in sufficient quantity and properly dispersed, it can be an effective means of mitigating alkali-silica reaction (ASR) in concrete. Silica-fume concrete requires no significant changes from the normal transporting, placing, and consolidating practices associated with conventional concrete. Silica-fume concrete is more cohesive and thus, less prone to segregation. It also exhibits little or no bleeding for improved durability. Without bleeding, finishing operations can be continuous from placement to texturing and curing, taking less time for finishing. Silica-fume concrete can also produce higher early age strengths, lessening the time required to strip forms and tension prestressing steel in structures. It is also possible that coatings can be applied sooner on silica-fume concrete than conventional concrete.

4.2.8.1.2 Limitations—Because silica-fume concrete is more susceptible to plastic shrinkage cracking, exhibiting significantly reduced bleeding, special attention to curing and protection is necessary to prevent cracking. Additionally, plastic silica-fume concrete tends to be sticky, making it more difficult to finish; however, finishers have overcome this condition without any significant difficulties ([Holland 1987](#)). Because silica fume concrete has little or no bleed water, it makes it difficult to provide a steel trowel finish. Hardened concrete containing silica-fume is generally darker than conventional concrete, and may not blend well with the appearance of the adjacent existing concrete. The addition of silica-fume adds cost to the repair material.

4.2.8.1.3 Applications—The first major applications of silica fume concrete in the United States were for the 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 is currently widely used to produce high-performance concrete. Additionally, it is used for corrosion protection in parking structures and bridges, both cast-in-place and precast. Another application is as an admixture for shotcrete to reduce rebound loss, increase placement thickness, and improve durability.

4.2.8.1.4 Standards—**ASTM C1240** is the specification covering silica fume.

4.2.8.2 Shrinkage-compensating cement—Shrinkage-compensating cement is used in concrete to minimize cracking caused by drying shrinkage. The basic materials and methods are similar to those used 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 or expansive component to counteract potential shrinkage is available and has been used successfully for years. For definitions of expansive cement and expansive component types G, K, M, and S, refer to **ACI 223R**.

4.2.8.2.1 Advantages—Ideally, unrestrained shrinkage-compensating concrete will expand by an amount equal to or slightly greater than the drying shrinkage. Generally, however, reinforcement or external supports result in restraint, which initially causes compressive stresses in the concrete. These stresses are subsequently relieved by drying shrinkage, but are of sufficient magnitude to prevent 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.

4.2.8.2.2 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 223R**. Low curing temperatures can reduce expansion, so water-curing at moderate temperatures for at least 7 days is essential for the development of the designed expansion. Consult with the manufacturer to determine the need for any special cleaning of transit mixer drums to prevent contamination.

Provisions should be made to allow for initial expansion of the material to provide positive strain on the internal steel restraint. Other types of restraint, such as adjacent structural elements, integral abutments, and subgrade friction also provide restraint and produce compression in the concrete. Provisions need to be taken to assure an expansion coefficient higher than that of shrinkage in order to avoid cracking from drying shrinkage. The forces created during the expansion process could push out walls or fail the forms along the perimeter of the pours.

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

4.2.8.2.4 Standards—**ASTM C845** provides standards and limits for expansive hydraulic cement and expansive components, including strength, setting time, and expansion. Mortar and concrete expansion is typically determined in accordance with **ASTM C806** and **C878**, respectively. Guidelines for the use of shrinkage-compensating concrete are outlined in **ACI 223R**.

4.2.8.3 Rapid-setting cements—Rapid-setting cementitious materials are characterized by faster setting times than conventional cements, as defined by **ASTM C1600**. Some may exhibit rapid strength development with compressive strengths in excess of 3000 psi (21 MPa) in 1-1/2 hours.

4.2.8.3.1 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 reduced protection times, lower traffic-control costs, and improved safety. Some rapid-setting cements provide significant reduction in shrinkage and increase durability in sulfate exposures and with reactive aggregates.

4.2.8.3.2 Limitations—Although most rapid-setting materials are as durable as normal portland-cement concrete, some, due to their constituents, may not perform well in a specific service environment. Rapid-setting cement systems are varied and should be evaluated for suitability in the intended application.

Some rapid-setting materials obtain their strength development, expansive properties, or both, 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.

Some rapid-setting materials are gypsum-based and may not have long-term durability in wet environments.

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.

4.2.8.3.3 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.

4.2.8.3.4 Standards—**ASTM C1600** covers performance requirements for rapid-hardening hydraulic cements. They are classified as URH, very-rapid hardening (VRH), medium-rapid hardening (MRH), or general-rapid hardening (GRH) based on setting time, compressive strength, and other specific criteria. There are no restrictions on the compositions of the cement or its constituents. Cements

are classified based on specific requirements for very early compressive strength development such as time of setting, compressive strength, and other test methods. As mentioned in 4.2.8.3.2, some rapid-setting cements may not perform well in specific service environments. The standard and optional physical requirements of ASTM C1600 are valuable in determining the suitability of a rapid-setting cement for specific applications. ASTM C928 covers packaged, dry, cementitious mortar or concrete materials for rapid repairs to hardened hydraulic-cement concrete pavements and structures. Use caution to check on exposure conditions, such as 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. Advances in the compounding of rapid-setting cements have taken place in recent years.

4.2.9 Common admixtures—ASTM C125 defines an admixture as “a material other than water, aggregates, hydraulic cementitious material, and fiber reinforcement that is used as an ingredient of a cementitious mixture to modify its freshly mixed, setting, or hardened properties and that is added to the batch before or during its mixing.”

4.2.9.1 Accelerating admixtures—Accelerators increase the rate of hydration of hydraulic cement in concrete or mortar, shortening the set time, increasing the rate of strength development, or both. Accelerating admixtures used in concrete should meet the requirements of ASTM C494, Type C or E.

4.2.9.1.1 Advantages—Accelerators are useful for modifying the properties of concrete or mortar, particularly in cold weather, to expedite finishing operations, reduce the time required for proper curing and protection, and increase the rate of early-strength development to permit earlier removal of forms and earlier opening of construction (ACI 212.3R). Using accelerators in cold-weather concrete typically is not sufficient to counteract effects of low temperature, and should be combined with cold-weather concreting practices (refer to ACI 306R).

4.2.9.1.2 Limitations—Accelerating admixtures should not be considered anti-freeze, because at normal dosages, accelerating admixtures lower the freezing point of water in concrete by just 4°F (2°C). Significant lowering of the freezing point of water requires very high dosages of accelerators, which can cause undesirable side effects. One of the major disadvantages of calcium chloride accelerators is that it can cause corrosion of steel reinforcement embedded in the concrete or repair material when in the presence of sufficient moisture and oxygen. Non-chloride accelerators should be used when the potential for corrosion is present.

4.2.9.1.3 Applications—Both chloride and non-chloride accelerators are used in concrete or mortar applications in cold weather, or whenever a fast set and strength gain is required.

4.2.9.1.4 Standards—ASTM C494 and D98 cover material to be used as chemical-accelerating admixtures and added to cementitious mixtures in the field.

4.2.9.2 Retarding admixtures—Retarders are admixtures that delay the setting of the cement paste and reduce the rate of hydration in concrete. Retarding action is achieved by the addition of sugar, carbohydrate derivatives, soluble salts and borates, or methanol to the concrete mixture.

4.2.9.2.1 Advantages—Retarding admixtures are used to decrease slump loss and extend workability, especially in hot weather, to ease placement and finishing of concrete.

4.2.9.2.2 Limitations—Retarders generally reduce the strength of concrete at an early age. If excessive quantities of retarders are used, they can totally prohibit the setting of the concrete or otherwise impart undesirable properties. Results are also dependent on when the retarder is added to the mixture.

4.2.9.2.3 Applications—Retarders are often used when placing concrete in hot weather when the normal setting time is shortened, during difficult placement, mass-concrete pours, or when placing concrete that requires a unique finishing technique such as exposed aggregate. Often in exposed aggregate finishing, retarders are applied to the interior face of formwork before placement so that the hardening of the cement on the formwork is delayed. This allows the cement adjacent to the formwork to be removed more easily to obtain the desired exposed-aggregate finish.

4.2.9.2.4 Standards—ASTM C494 covers material to be used as chemical-retarding admixtures and added to cementitious mixtures in the field.

4.2.9.3 Water-reducing admixtures—Water-reducing admixtures (WRAs) are chemical admixtures that, when added to cementitious materials such as concrete or mortars, release the water entrapped between the cement particles and make it available to increase the workability of the mixture. They are available in both liquid and powder form. Water-reducing admixtures used in prebagged repair materials are typically in powder form and those used for concrete production are typically in liquid form. Water-reducing admixtures used in cement mortars or concrete should meet the requirements of ASTM C494, Type A, D, E, F, or G. Depending on their water-reducing capacity, WRAs can be classified as water-reducing admixtures (Types A, D, or E) or HRWRA (Types F or G). For more information on WRAs, refer to ACI 212.3R.

4.2.9.3.1 Advantages—WRAs are used in cementitious concrete or mortars for three primary reasons: 1) to increase the workability at the same w/cm ; 2) to increase strength by reducing the w/cm at the same workability; or 3) to optimize the cementitious content by keeping the workability and w/cm the same. Typically, a combination of all three benefits is taken during concrete or mortar mixture design proportioning.

Increased workability allows the concrete or mortar to be placed easily, especially in confined spaces. Reducing the w/cm allows increasing the strength and durability of the concrete or mortar without compromising workability. Cement optimization allows controlling the heat generated during the heat of hydration of cement, and also allows lowering the cost of the cementitious mixture.

4.2.9.3.2 Limitations—Adding the correct amount of WRA in cementitious mixtures is important; a dosage rate that is too high can cause problems such as bleeding, segregation, or extension of set time. A dosage rate that is too low can result in reduced workability which can prevent proper placement of the mixture. Some WRAs can also cause an increase in the air content of the mixture.

4.2.9.3.3 Applications—Water-reducing admixtures are used for concrete production in the ready-mix, precast, and dry-cast industry, and in on-site or mobile concrete plants. Water-reducing admixtures are also used in prebagged cementitious mortars or concrete. High-range water-reducing admixtures are an important constituent of self-consolidating concrete (SCC). However, aggregate proportioning and other mixture constituents are also important design considerations for SCC (ACI 237R).

4.2.9.3.4 Standards—ASTM C494 covers material to be used as chemical water-reducing admixtures and added to cementitious mixtures in the field.

4.2.9.4 Corrosion-inhibiting admixtures—Several classifications of corrosion-inhibiting admixtures are available. They can be generally grouped into three types: cathodic, anodic, and ambiotic (or mixed) inhibitors, depending on their area of activity on the corroding steel. Cathodic inhibitors act to prevent the reaction at the cathode. Cathodic inhibitors act indirectly because they do not prevent metal dissolution. Anodic inhibitors reduce the rate of reaction at the anode. They typically react with the corrosion products to form a protective coating on the metal surface. Nitrite-based corrosion inhibitors are one common type of anodic inhibitor. Mixed inhibitors influence both the anodic and the cathodic sites that are claimed to be advantageous in reinforced concrete due to the prominence of microcell corrosion.

Inhibitors are classified by the Strategic Highway Research Program SHRP-S-666 (Al-Qadi et al. 1992) as adsorbed layer formers, oxidizing inhibitors, passivators, conversion layer formers, and scavengers. Adsorbed layer formers are organic inhibitors that strongly adsorb to the metal surface and interfere with the anodic or cathodic reactions in the area of adsorption. Nitrogen is typically the active atom in an adsorbed layer inhibitor acting in a non-acid electrolyte on steel. Typical compounds of nitrogen used as inhibitors are organic nitrate and amines. Vapor phase inhibitors (VPIs) or volatile corrosion inhibitors (VCIs) are similar to adsorbed layer inhibitors. Vapor phase inhibitors are secondary electrolyte layer inhibitors that possess appreciable saturated vapor pressures under atmospheric conditions, thus allowing significant vapor phase transport of the inhibitive substance. Aliphatic and cyclic amines and nitrites with a high vapor pressure typically make up these inhibitors. Vapor phase inhibitors\volatile corrosion inhibitors can be added to cementitious materials as admixtures and be used as SACI. Oxidizing inhibitors or passivators are another form of barrier inhibitor that act by shifting the electrochemical potential of the corroding metal such that an insoluble oxide or hydroxide forms on the metal surface. Sodium nitrite and chromates are examples of this inhibitor. Another form of

passivators is metal soaps that are a form of the basic alkali and oxidation products of oil and that form passivating films on the metal surface. Conversion layer inhibitors form insoluble compounds on metal surfaces without oxidation. In neutral or basic solution, the presence of calcium and magnesium ions inhibits corrosion by the formation of an insoluble calcareous scale on the metal surface. Finally, scavengers act as neutralizing inhibitors by removing concentrations of corrosive materials such as chloride ion.

The National Association of Corrosion Engineers (NACE) classifies all corrosion inhibitors, not just those specific to concrete applications, as passivating inhibitors, cathodic inhibitors, organic inhibitors, precipitation inhibitors, and VCIs. Passivating inhibitors cause a large anodic shift of the corrosion potential, forcing the metallic surface into the passivation range and are further divided into oxidizing anions and non-oxidizing ion types. Cathodic inhibitors either slow the cathodic reaction itself or selectively precipitate on cathodic areas to increase the surface impedance and limit the diffusion of reducible species to these areas, and are further classified as cathodic poisons and oxygen scavengers. Organic inhibitors can show both anodic and cathodic effects but, as a general rule, affect the entire surface of a corroding metal when present in sufficient concentration. Organic inhibitors are typically designated as film-forming, to protect the metal by forming a hydrophobic film on the metal surface. Precipitation-inducing inhibitors are film-forming compounds that also have a general action over the metal surface, blocking both anodic and cathodic sites indirectly by forming a precipitated deposit on the surface of the metal to provide a protective film. Volatile corrosion inhibitors and VPIs are compounds transported in a closed environment to the site of corrosion by volatilization from a source. On contact with the metal surface, the vapor of VCI salts condenses and is hydrolyzed by moisture to liberate protective ions.

4.2.9.4.1 Advantages—Corrosion inhibitors used as admixtures can be precisely controlled for dosage, pretested for compatibility in the material, and, once used, are in intimate contact with the steel to be protected. Surface-applied corrosion inhibitors (SACIs) for hardened concrete are intended to penetrate into the concrete to reduce corrosion activity.

4.2.9.4.2 Limitations—The effectiveness of corrosion inhibitors depends on their concentration. The required amount of corrosion inhibitor is a function of the level of corrosion activity. Anodic inhibitors can increase the rate of attack in unprotected areas (Al-Qadi et al. 1992), similar to the haloing effect that is sometimes produced around repairs due to the increase in the cathodic area after repair. Because of this concentration dependency and effects on adjacent areas—that is, areas that have received insufficient concentrations—inhibitors with these properties are sometimes called dangerous, as they may accelerate corrosion.

Most corrosion inhibitors are water-soluble with some inhibitors also being volatile; therefore, the concentration of inhibitor may change with time. The corrosion activity will also change as concrete deterioration occurs, chlo-

ride ions ingress, and carbonation occurs that may require a higher concentration of inhibitor than what is within the concrete. Corrosion is also a complex series of electrochemical processes in a changing environment that can produce varying results with inhibitors in field conditions.

No standards exist to specifically address SACIs and VCIs for concrete, with much of the literature presenting differing opinions about their effectiveness.

4.2.9.4.3 Applications—Another classification of corrosion inhibitors is in terms of application. Inhibitors are either admixed with the concrete or repair material as an ingredient, or can be applied to hardened concrete surfaces, relying on penetration to reach the reinforcing steel surface.

4.2.9.4.4 Standards—ACI 212.3R discusses miscellaneous admixtures for concrete, including corrosion-inhibiting admixtures. ASTM C1582 is a specification for admixtures used as corrosion inhibitors. ASTM G109 is a widely used test method for evaluating the effectiveness of corrosion-inhibiting admixtures, and various modifications have been used for SACI and VCI types.

4.2.9.5 Lithium-based admixtures—Various lithium compounds (Li_2CO_3 , LiOH , Li_2SO_4) are used in the formulation of set accelerators for calcium-aluminate-cement concrete, and both lithium hydroxide monohydrate ($\text{LiOH} \cdot \text{H}_2\text{O}$) and lithium nitrate (LiNO_3) have been used to control alkali-silica reactivity (ASR) in portland-cement concrete. Lithium admixtures consist of water-soluble salts and are used in high-alkali concrete containing reactive aggregates to inhibit and control ASR. For an in-depth discussion of the chemistry related to ASR, refer to FHWA-HRT-06-133 (Thomas et al. 2007).

The amount of lithium required to suppress expansion depends on the form of lithium, the nature of the reactive aggregate, and the amount of alkali in the concrete. Many studies have shown that the expansion of concrete for a given aggregate depends on the amount of lithium relative to the amount of sodium plus potassium in the mortar or concrete mixture.

Numerous structures have been treated by spraying the surface of the structure with a solution of lithium (both LiNO_3 and LiOH have been used). Very little lithium penetrates below 1 to 2 in. (25 to 50 mm) unless the concrete is heavily cracked. Even in heavily cracked concrete, the lithium concentration decreases with depth, and its ability to suppress ASR is questionable.

Electrochemical impregnation techniques have been used to increase lithium penetration on a number of structures. Significant lithium can penetrate to a depth of at least 0.75 to 1.25 in. (19 to 32 mm), and these dosages are theoretically high enough to have a beneficial effect on reducing ASR-induced expansion.

Vacuum impregnation is an alternative to pressure injection and has been used to increase grout penetration into cracked concrete. A number of structures have been treated with lithium using this technique; however, documentation of the benefits is not sufficient to draw reliable conclusions.

Before treating a structure with lithium-based compounds, an investigation should be conducted to ensure that the

primary cause of damage is ASR. Lithium treatment is unlikely to cure any other deterioration processes such as freezing-and-thawing damage, corrosion of embedded steel, or even alkali-carbonate reaction (ACR). Proper diagnosis involves extracting samples for petrographic analysis and other testing in the laboratory. Following treatment, the potential for further expansion and damage due to ASR can still remain.

Lithium treatment will not repair any damage that has already occurred. A protocol for selecting structures that may be suitable for lithium treatment is available from the Federal Highway Administration (FHWA) HRT-04-113 (Thomas et al. 2004) and FHWA recommendations for lithium to mitigate ASR.

Treating structures with lithium is a technology that is still under development, and recommended protocols for selecting the type of treatment (for example, topical, electrochemical, or vacuum impregnation) or methodologies for performing the treatment do not exist. Electrochemical and vacuum impregnation requires specialized knowledge and equipment, and should be conducted only by an experienced contractor.

4.2.9.6 Anti-washout admixtures—Anti-washout admixtures (AWAs) are chemical admixtures that bind the free water in the concrete mixture and reduce the loss of fine materials from the concrete mixture into the water during placement of concrete under water.

4.2.9.6.1 Advantages—Anti-washout admixtures are often used in concrete mixtures proportioned for underwater placement. The AWA improves the flow and cohesion characteristics of the mixture. The loss of cement and other fine material from the mixture due to contact with water during placement is reduced, thus improving the in-place concrete properties. The mixtures can be proportioned to be self-consolidating and to flow around reinforcement.

4.2.9.6.2 Limitations—Concretes made with AWAs are often tacky, may exhibit long setting times, and are sensitive to slight changes in the w/cm and admixtures. Concretes made with AWAs typically are rich in cementitious materials and may therefore require evaluation of thermal stresses when used in large placements. High-range water-reducing admixtures are used to maximize the benefit of the AWA. Compatibility of admixtures, particularly when air-entraining admixtures are used, should be evaluated using trial batches.

4.2.9.6.3 Applications—Concretes using AWAs are used for underwater concrete placement. The mixtures can be placed by tremie, pump, or slipform. Applications include concrete repair as well as new construction. Anti-washout admixtures may allow free fall of concrete mixtures through 2 to 3 ft (0.6 to 0.9 m) of water during placement. Concrete made with AWA is less susceptible to loss of material due to contact with slowly flowing water.

4.2.9.6.4 Standards—Anti-washout admixtures should conform to USACE specification CRD-C 661 (USACE 2006). Test method CRD-C 61 (USACE 1989) can be used to measure the loss of cement paste/fines from freshly mixed concrete resulting from contact with water.

4.2.10 Other concrete and mortar additives—Materials added to concrete and mortar that are not supplementary cementitious materials and do not fall under the definition of an admixture are contained in this section.

4.2.10.1 Fiber reinforcement—Fiber reinforcement consists of conventional concrete reinforced with discrete reinforcing fibers. **ACI 544.1R** identifies four classifications of fiber-reinforced concrete (FRC): steel fiber-reinforced concrete (SFRC); glass fiber-reinforced concrete (GFRC); synthetic fiber-reinforced concrete (SNFRC); and natural fiber-reinforced concrete (NFRC).

Fiber reinforcement of building materials such as brick and adobe has been practiced for thousands of years. Reinforcing concrete with fibers has become an accepted design option where crack control due to plastic, drying, or service-related shrinkage is desired. Fiber-reinforced concrete has been shown to provide significant increases in durability and impact resistance. Using polyolefin fibers to improve the spalling resistance of concrete in fire events by allowing steam ventilation is well documented. Research has developed promising results in various structural applications such as steel-deck-formed slabs, structural slabs-on-ground, and formed elevated concrete slabs. Using fibers in combination with conventional reinforcing steel is practiced where both structural strength and improved crack resistance are needed.

Fiber-reinforced concrete has been successfully used for concrete repairs using conventional and shotcrete placement. Though all four types of fibers have been used to produce FRC, SFRC and SNFRC are more applicable to the concrete repair industry, while GFRC and NFRC are more applicable to the fabricated building products industry. Information on fiber-reinforced concrete or shotcrete can be obtained from **ACI 544.1R**, **ACI 544.3R**, **ACI 544.4R**, and **ACI 506.1R**.

4.2.10.1.1 Advantages—Fibers are typically added during concrete production whether plant- or site-mixed. These fibers can be used to provide reinforcement in thin overlays where installation of conventional reinforcing bars is impractical. The addition of fibers can also 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, can benefit from the addition of fiber reinforcement.

4.2.10.1.2 Limitations—The addition of fibers reduces the slump and can cause workability problems for inexperienced workers. Rust stains may occur at the surface of SFRC due to corrosion of fibers at the surface.

4.2.10.1.3 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. Research has found potential structural applications for FRC in new construction. Advances in the use of FRC may provide future applications in the repair industry.

4.2.10.1.4 Standards—**ASTM C1116** covers the materials, proportions, batching, delivery, and testing of fiber-reinforced concrete and shotcrete.

4.2.11 Combination materials—The preceding repair materials are often used in combination to enhance important properties for specific applications. As an example, rapid-setting cement is used instead of conventional Type I/II to produce very-high-early-strength polymer-cement concrete. Whereas conventional LMC overlays should be cured 4 to 7 days before being opened to traffic, high-early-strength LMC (LMC-HE or LMC-VE) can be subjected to traffic in less than 1 day. Using LMC-VE reduces down time and inconvenience to motorists; allows for installation at night; provides negligible to very low permeability; and produces high strength, particularly high early strength (**Sprinkel 1998**).

4.3—Crack repair materials

4.3.1 Epoxy resin—Epoxy resin is a thermosetting polymer that cures when mixed with a catalyzing agent or hardener. Epoxies have been the products of choice for crack repair in the industry for decades. Once cured, epoxies have exceptional physical properties. Epoxies are recognized for their superior bond strength to concrete, which typically exceeds the tensile strength of normal concrete. Commercially available epoxies have been formulated to meet variable working time requirements and offer a wide range of viscosities.

4.3.1.1 Advantages—There is no special surface preparation required to the inside of the crack that is being repaired. If the crack is wide enough, debris can be vacuumed out of the crack. Cracks repaired with epoxies are considered structural repairs. Low-viscosity epoxies have been proven to repair cracks as thin as 0.002 in. (0.05 mm) wide. Some commercially available epoxies can develop bond in damp concrete cracks.

4.3.1.2 Limitations—Epoxies are used to repair cracks that typically range from 0.002 to 0.25 in. (0.05 to 6 mm) wide. Because the coefficient of thermal expansion of the epoxy resin and the concrete are vastly different, a thicker cross section of the epoxy in a crack exposed to large temperature variations can cause internal stresses in the repaired concrete. Epoxy resin should only be used in nonmoving or dormant cracks. Cracks that continue to move after they have been repaired with an epoxy are likely to develop cracks parallel to the original cracks. Also, some epoxies used for crack injection may be sensitive to the moisture saturation in the crack. Mixing epoxy in proper proportions is critical to the performance of the product. Epoxy resins that are incorrectly mixed result in improperly cured material.

4.3.1.3 Application—Epoxies are injected into cracks using multiple methods and techniques such as gravity feed and pressure injection. Properties of the chosen material should be considered when selecting the appropriate application method. In-depth application method descriptions can be found in **5.2.1**. Refer to **ACI 546.3R**, **ACI RAP-1**, and **Trout (2006)** for additional information on crack repair materials and typical applications.

4.3.1.4 Standards—**ASTM C881** classifies epoxies by their intended use, viscosity, and temperature limitations. The standard classifies seven different types of epoxies. Injection resins are generally classified as Type 1, non-

load-bearing applications for bonding hardened concrete to hardened concrete or Type IV, load-bearing applications for bonding hardened concrete to hardened concrete.

4.3.2 Urethane resins—Urethane resins, such as epoxy resins, are thermosetting polymers. Many moisture cure, while some cure when mixed with a catalyzing agent or hardener. They have been formulated as rigid and flexible products with a range of working times, allowing for application in a wide array of repair conditions. They are also available in different viscosities, some ranging to below 9 cP ($1.31 \times 10^{-6} \text{ lb}\cdot\text{s}/\text{in.}^2$ [$0.009 \text{ N}\cdot\text{s}/\text{m}^2$]) with surface tensions below 0.0021 lb/ft (0.03 N/m) to allow penetration of the substrate for structural repairs. Some products may also be applied with the addition of aggregate to accommodate larger repair areas. As with cement-based repair products, urethane resins with similar strength to existing concrete should be selected for structural repairs.

4.3.2.1 Advantages—Urethane-based repair materials are recognized for their rapid curing. Some fully cure in 10 minutes at 70°F (21°C). Additionally, they are recognized for their ability to penetrate and structurally repair concrete and their suitability in a wide range of repair applications. They are available for the repair of moving and static cracks of varied widths and depths. In addition, urethane products are available that can be applied at temperatures below -20°F (-23°C). Unlike epoxy resins, which often have higher compressive strengths, urethane resins are available with compressive strengths in the 4000 to 5000 psi (27.5 to 34.5 MPa) range, similar to typical concrete substrates.

4.3.2.2 Limitations—Urethane resins may be sensitive to moisture before curing, requiring surrounding concrete to be dry and cured for at least 60 days at time of application. Urethane resins may also be sensitive to UV rays, causing them to yellow in appearance over time when exposed to UV sunlight. When low-viscosity, low-surface-tension products are used, such resins may leak out of full-depth repairs. Typically, the full-depth repair area is first filled with dry sand and then the urethane is poured in. A second application may be needed after the first has cured.

4.3.2.3 Applications—Urethane resins are typically gravity-injected to achieve full- or partial-depth repairs, depending on application. When applying low-viscosity, low-surface-tension products, specified aggregates may be added to the repair to reduce penetration of material for partial-depth repairs. For full structural repairs, material should be allowed to gravity feed the full depth of the repair area. Once this has been achieved, sand may be introduced to prevent under-slab ponding and waste. The addition of sand also adds strength to the repair and brings the thermal coefficient of expansion of the repair material closer to that of the surrounding concrete.

4.3.3 Polyurethane chemical grouts—Although there are other forms of chemical grouts, polyurethanes are the most common choice of material for repair of cracks by chemical grouting, and are classified into hydrophobic and hydrophilic types. Polyurethane chemical grouts consist of a polyurethane resin that reacts with water to form an expansive, closed-cell foam or a gel. Hydrophobic types are gener-

ally recommended for applications subject to intermittent wetting and drying; hydrophilic types should be continuously wet ([USACE 1995b](#)). Refer to [ICRI 340.1](#) for additional information.

4.3.3.1 Advantages—Polyurethane chemical grouts are typically used to repair both wet and active cracks, or cracks leaking a significant amount of water. Some grouts are semi-flexible; thus, they may tolerate some change in crack width. Therefore, understanding crack movement is a key consideration in selecting a polyurethane chemical grout. The reaction time to form the foam may be controlled from a few seconds up to several minutes using different catalyst additives. For example, where a crack is leaking heavily, the polyurethane chemical grout reaction may be highly accelerated to stop the leak. These grouts penetrate effectively, and the technique of chemical grouting is a well-proven method of repairing cracks.

4.3.3.2 Limitations—Polyurethane chemical grouts generally are not suitable for structural repairs. Additionally, a highly skilled work crew is required along with special injection equipment. Finally, these materials typically are not stable when exposed to UV light. This is typically not a major concern because the material is injected into a narrow crack, where exposure to UV light is minimal.

4.3.3.3 Applications—Polyurethane chemical grouts may be used to treat cracks 0.005 in. (0.12 mm) and greater in width and are injected at high pressures. Polyurethane grouts are suitable for injection of vertical, overhead, and horizontal cracks that are active or leaking. Their ability to stop active leaks makes them particularly well-suited for liquid storage tanks, dams, tunnels, sewers, and other water-containment structures.

4.3.3.4 Standards—No standards exist for polyurethane chemical grouts. [ASTM D1623](#) may be used to determine the tensile strength and elongation properties of the grout at all urethane-to-water ratios.

4.3.4 Polyurethane sealant—Polyurethane sealants, also known as urethane sealants, are based on polymers that result from the reaction of an isocyanate and a hydroxyl-bearing compound, such as a polyol. Polyurethane polymers are typically blended with fillers, colorants, curing agents, and other additives that govern viscosity, adhesion, long-term stability, and resistance to weathering. Polyurethane sealants are formulated for treating both static and dynamic (moving) cracks, filling concrete and masonry joints. Product formulations are tailored for use in horizontal or vertical surfaces, as well as exposure to continuous water immersion.

4.3.4.1 Advantages—The advantages of polyurethane sealants include their overall adhesion to unprimed concrete and other porous surfaces. Once tested and verified, they provide virtually unlimited color selection (especially in multi-component materials); they have an ability to be used in cracks and joints (as wide as 2 in. [50 mm] in some cases); and they have extremely low incidence of surface or migratory staining when applied against concrete, masonry, and other porous materials. Some polyurethanes are formulated for environments requiring enhanced physical durability such as pedestrian or vehicular traffic surfaces. Polyurethane

sealants are also frequently used in combination with deck coatings or architectural wall coatings, with which they must be chemically compatible for both materials to function properly. The movement capability of most polyurethanes enables them to accommodate normal dimensional changes in concrete due to structural and thermal stresses.

4.3.4.2 Limitations—Single-component polyurethanes require the presence of atmospheric moisture to cross-link and develop their design properties. In arid environments or cool, dry weather conditions when atmospheric moisture is scarce, such a sealant may be damaged by repeated movement cycles or traffic exposure before it can substantially cure. Single-component materials also typically have a limited color palette from which to choose. Multiple-component sealants, on the other hand, require careful and thorough mixing to develop their intended properties after installation.

4.3.4.3 Applications—Polyurethane sealants are used in the repair of static or dynamic cracks and joints in vertical elements—for example, wall panels or horizontal surfaces such as pedestrian and vehicular traffic areas. Specialty applications include water-immersion situations such as tanks, dams, and reservoirs, and security work such as joints in window perimeters in detention facilities, which require a harder, tamper-evident material.

A good joint design is critical to ensure that the sealant material performs properly; therefore, a 2:1 joint width-to-depth ratio should be maintained. Some smaller cracks will have to be routed to comply with these limitations.

4.3.4.4 Standards—ASTM C920 governs virtually all applications where polyurethane sealants are used. Historically, Federal Specifications, such as TT-S-00230C for single-component sealants and TT-S-00227E for multiple-component sealants, have also been used. Although these specifications are still frequently cited, they are no longer maintained, and their use has been superseded by ASTM C920.

4.3.5 Silicone sealant—Silicone sealants are based on polymers where the polymer backbone consists of alternating silicon and oxygen atoms with carbon-containing side groups. These sealants have different curing mechanisms, depending on the end group of the polymer. Typically, silicone sealants contain silicone polymers, fumed silica, plasticizers, calcium carbonate fillers, and silanes for adhesion.

4.3.5.1 Advantages—Silicone sealants are highly resistant to UV light and do not chalk or change color even when exposed for long periods of time. Silicone sealants have high elongation properties (1000 percent and greater per ASTM D412), making them suitable for active joints.

4.3.5.2 Limitations—Silicone’s adhesion to concrete and masonry can be inadequate unless a primer is used. Silicone sealants should not be used for immersion; thus, they are not used for sealing joints within water tanks, reservoirs, dams, and similar applications. Additionally, silicones may only be overcoated with silicone-based coatings.

Oils within the silicone sealants will often migrate into a porous substrate, resulting in staining of the substrate, making these materials aesthetically unacceptable. Nonbleeding sili-

cones are commercially available to address this specific issue.

4.3.5.3 Applications—Silicone sealants are generally used to seal cracks that range from 0.1 to 2 in. (2.5 to 50 mm) in width. Joint widths should range from 0.25 to 1 in. (6 to 25 mm) and depths should range from 0.25 to 0.5 in. (6 to 13 mm). These products are suitable for vertical and overhead applications. Some manufacturers offer sealant formulations suitable for traffic. Silicone sealants are not suitable for structural repairs.

A good joint design is critical to ensure that the sealant material performs properly; therefore, a 2:1 joint width-depth ratio should be maintained. Some smaller cracks will have to be routed to comply with these limitations.

4.3.5.4 Standards—ASTM C920 governs virtually all applications where silicone sealants are used. Historically, Federal Specifications, such as TT-S001543A for single-component silicone sealants, have also been used. The Federal Specifications, although still frequently cited, are no longer maintained, and their use has been superseded by ASTM C920.

4.3.6 Methacrylates—Methacrylates are liquid resins used to seal concrete slabs and structurally heal small, shallow cracks wider than 0.005 in. (0.12 mm). Because of their high adhesive strength, methacrylates can bond cracks as a structural repair. The most common types are HMWM and reactive methacrylates. Both types are similar in application and performance to super-low-viscosity epoxies, with the viscosity of methacrylate resin typically below approximately 30 cP (4.35×10^{-6} lb·s/in.² [0.03 N·s/m²]).

4.3.6.1 Advantages—Although methacrylates are generally considered semi-rigid and not appropriate for resisting structural movement, cracks bonded using methacrylates can typically resist movements caused by traffic vibration. Flood-coating a concrete slab using methacrylates will fill and bond small cracks, such as shrinkage cracks, and seal the slab against the penetration of water, salts, or other deleterious chemicals. Methacrylates also perform well at cold temperatures and even achieve good bond to cured and aged methacrylate so recoating is possible, unlike epoxies.

4.3.6.2 Limitations—Because methacrylates are typically installed using flood-coat techniques, ensuring a full-depth structural repair can be difficult. Because of their low viscosity, methacrylates are rarely pressure injected. Methacrylates can leak out of full-depth cracks because they have such low viscosity and surface tension. Typically, the full-depth crack is first filled with dry sand and then the methacrylate is poured in. A second application may be needed after the first has cured. If a full-depth repair of larger cracks is needed, pressure injection using epoxies should be considered instead. Methacrylates are best applied to horizontal concrete surfaces, although they can be used on sloped concrete surfaces such as parking garage entry ramps. However, because of the very low viscosity of the materials, care should be taken on sloping surfaces to prevent the material from flowing out of cracks. Although methacrylates are commonly seeded with aggregates, the effect of the methacrylate on skid resistance should be considered for bridge

decks exposed to heavy traffic and high speeds. Some methacrylates, particularly reactive methacrylates, can have a strong odor. Protection should be provided to keep the work area away from direct contact with the public, especially in enclosed areas or in close proximity to building air intake vents. Three-component HMWM formulations should be mixed with extreme care, as they can react violently if mixed improperly. The promoter should be mixed thoroughly into the resin before adding the catalyst. Never mix the catalyst and promoter directly together.

4.3.6.3 Applications—Flood-coating with methacrylates is well suited to bridge decks, parking garages, and industrial floors or other concrete slabs that have a large number of random cracks but no evidence of steel corrosion or alkali-aggregate reaction. Before installation, care should be taken to ensure the cracks are open, clean, and free of any contaminants or other bond-inhibiting materials such as oil and grease. Because methacrylates can be sensitive to moisture, the substrate should be dry at the time of application. For more information on proper application procedures, refer to **RAP-13**.

4.3.6.4 Standards—There is no standard specification for methacrylates; however, RAP-13 may be referenced. Similar to epoxies, **ASTM C882** can be used for evaluating the bond strength of methacrylates.

4.3.7 Polymer grout—Polymer grout is a mixture where a polymer, such as an epoxy resin, serves as the binder. Sand—typically oven-dried silica with a grading from No. 20 to +40 mesh (0.8 to 0.4 mm)—is generally mixed as the filler. Depending on how much resin is added, the variability of the polymer grout can be adjusted from a stiff material suitable for hand-packing large cracks on overhead and vertical surfaces or to a pourable consistency grout suitable for gravity-feeding cracks in horizontal slabs.

4.3.7.1 Advantages—Polymer grouts bond extremely well to concrete and have low shrinkage, resulting in a liquid-tight repair in dormant cracks. Similar to epoxy resins, polymer grouts are suitable for cracks requiring structural repairs. Because of their rigidity, if polymer grouts are used in cracks that experience movement, parallel cracks may appear adjacent to the repaired cracks. Materials of varying consistencies are readily available to repair cracks in vertical, overhead, and horizontal applications. Whereas a few polymer grouts may effectively bond to concrete with some moisture present in the concrete pores, most polymer grouts marketed for construction will not bond to concrete in the presence of moisture. The chemical resistance of polymer grout is generally much better than that of normal concrete. Finally, these materials may be designed for a fast cure to minimize the downtime due to repairs.

4.3.7.2 Limitations—As with epoxy resins, it is critical to properly proportion the resin and the hardener and to thoroughly mix the sand to get the desired polymer grout. Failure to do so could result in a grout that never cures or cures incompletely. Generally, the temperature exposure of these materials should not exceed 120 to 250°F (49 to 66°C). The first detectable loss of strength could occur at these temperatures.

4.3.7.3 Application—Polymer grout is typically used to repair dormant cracks 0.25 in. (6 mm) and greater in width. Polymer grouts can be hand-applied and packed into large cracks regardless of orientation, or they can be poured into cracks located within horizontal slabs. No special equipment, except the personal protective equipment required for handling the material, is required to apply these materials. The required applicator skill levels are low to moderate. To produce a polymer grout, the resin and hardener are mixed, and then the filler is gradually added while mixing until a homogenous mixture is achieved. Mixing is typically done with an eggbeater-style mixing blade and a hand-held electric drill. Pot life or working time may be varied to suit the application. Long pot-life products enable the applicator to mix and apply larger quantities. Fast-cure products allow rapid turnovers; however, care should be taken to mix only sufficient material that may be applied within the specified pot life.

4.3.7.4 Standards—No standards exist for polymer grouts intended for use as crack repair. ASTM C882, **D638**, and **D695** should be considered when evaluating these materials. ASTM **C881** classifies epoxies by their intended use, viscosity, and temperature limitations.

4.3.8 Polymer-cement grout—Polymer-cement grout is a mixture consisting primarily of cement, fine aggregate, water, and a polymer such as acrylic, styrene-acrylic, styrene-butadiene, or a water-borne epoxy. The consistency of this material can be adjusted from a stiff material suitable for hand-packing large cracks on overhead and vertical surfaces to a pourable consistency suitable for gravity-feeding cracks in horizontal slabs.

These materials are available in a wide range of consistencies suitable for applications requiring improved physical properties including bond strength, compressive strength, and flexural strength as compared with conventional water-based cement grouts that do not use polymers.

4.3.8.1 Advantages—No special tools and equipment are necessary except the required personal protective equipment; the required applicator skill levels are low to moderate. These materials are generally more economical than polymer grouts.

4.3.8.2 Limitations—The potentially high shrinkage of polymer-cement grouts compared with polymer grouts may make it difficult to obtain a watertight repair. Additionally, these materials are not as chemically resistant as the polymer grouts.

4.3.8.3 Applications—Polymer-cement grout is generally used to repair cracks 0.25 in. (6 mm) and greater in width. The surface should be prepared to ensure a clean, open-pore substrate. The substrate should be in a saturated surface-dry (SSD) condition, with no standing water at the time of application. Mixing should be done using a drill and paddle or an appropriate mixer. The material should be scrubbed into the substrate to fill all pores and voids, and then packed into the crack and finished flush with the concrete surface.

4.3.8.4 Standards—The polymer component should conform to the requirements of **ASTM C1438**, Type II. No standards exist for mixtures of the polymer component with

grout; however, recommended tests for these materials are found in [ASTM C1439](#), [C157](#), [C293](#), [C469](#), [C496](#), and [C531](#).

4.3.9 Cement grout—Cement grout is a mixture of cementitious material and water, with or without admixtures, aggregates, or both, that is proportioned to produce consistencies ranging from pourable to stiff. Admixtures are often used in an attempt to eliminate segregation of constituents in pourable applications.

Cement grouts mixed with fine aggregate typically have mixture ratios that vary from 2:1 sand-to-cement proportions, to grouts that are neat cement without sand, measurements are parts by volume. When filling a crack with grout, the crack width should be wide enough for the grout to fill the crack and also penetrate the crack depth. For this reason, the fine aggregate size can be the limiting factor for proper crack fill of tight cracks. Cracks smaller than the sand-aggregate diameter should be filled with neat cement grout.

The consistency of the grout may vary from a stiff material suitable for hand-packing large cracks on overhead and vertical surfaces to a pourable consistency suitable for gravity feeding cracks in horizontal slabs. The amount of water added controls the grout consistency.

4.3.9.1 Advantages—No special tools or equipment are necessary and the required applicator skill levels are low to moderate. Cement grout is typically the least expensive repair grout because there are no additional binders other than the cement.

4.3.9.2 Limitations—Cement grouts can be used for repairs by injection, but only where the width of the opening is sufficient to accept the solid particles suspended in the grout. The minimum crack width at the point of introduction should be approximately 0.25 in. (6 mm).

4.3.9.3 Applications—Cement grout is generally used to repair cracks 0.25 in. (6 mm) and greater in width. For additional applications, refer to 4.2.4.3. The substrate should be clean, sound, and damp at the time of application. The material should be packed into the crack and finished flush with the concrete surface.

4.3.9.4 Standards—There are no standards developed strictly for grouts to repair cracks. [ASTM C1107](#) is intended for materials used under applied loads such as base plates for a structure or machine.

4.4—Bonding materials

Bonding materials can be used to bond new repair material or fresh concrete to an existing prepared concrete substrate. Based on the chemistry, bonding materials can be classified as one of four types: 1) epoxy-based; 2) latex-based; 3) modified epoxy-based; and 4) cement-based.

4.4.1 Epoxy bonding materials

4.4.1.1 Advantages—Epoxy resins have been widely and successfully used in the industry as a bonding agent for many years. Epoxy bonding agents are known for their excellent adhesive properties. Over the years, improvements to the chemistry have allowed epoxy agents to be moisture tolerant. Epoxy bonding agents perform best when the substrate is dry and when prewetting the surface is impractical. It is to be expected that when an epoxy is used as a bonding agent,

the bond strength at the bond line should exceed the tensile strength of the substrate and repair concrete.

Epoxies can have very high compressive and bond strength while maintaining good flexural properties. Their adhesion to substrates, especially imperfect surfaces, makes them useful in many applications. Epoxies commonly have a high shore hardness and can withstand abuse from weather and abrasion. If an epoxy is made up of 100 percent solids, the shrinkage values are very low. Once an epoxy is cured, its chemical resistance is very high, which makes epoxies a good choice for volatile environments. Epoxy can be formulated to cure at varying rates; thus, an epoxy can be chosen to fit the project-specific criteria.

4.4.1.2 Limitations—Care should be taken when using these materials at all times, especially in hot weather, as 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, such as slabs-on-ground in a freezing-and-thawing environment, a moisture barrier could result in failure of the repair when moisture becomes trapped beneath the moisture barrier and freezes. In such cases, epoxy resins should not be used for slab-on-ground concrete overlays. If epoxy resins are used in such situations, applying the epoxy intermittently over the area (leaving areas without an epoxy barrier) may provide satisfactory results.

4.4.1.3 Applications—Surface preparation is critical for proper epoxy performance. The surface should be clean and sound. It may be slightly damp but should be free of standing water. Dust, laitance, grease, curing compounds, impregnations, waxes, and any other contaminants should be removed before placing the epoxy resin. Concrete should have an open-textured surface prepared by blast cleaning or other equivalent mechanical methods. Steel should be blast cleaned to a white metal finish.

Two-component epoxy bonding agents are mixed mechanically to obtain a uniform mixture. Once mixed, epoxy is typically applied with a brush or roller. Larger jobs may use approved spraying equipment for a faster production rate. Proper protective gear as suggested by the manufacturer should be used in working with epoxy resins. Clean-up of the epoxy material should be performed with approved chemicals. Epoxy can bond fresh or hardened concrete to metal, wood, or concrete.

4.4.1.4 Standards—Epoxy systems are covered in [ASTM C881](#) and [AASHTO M-235](#) specifications. ASTM C881 categorizes epoxy resins by “type” (application characteristics), “grade” (viscosity), “class” (reaction rate), and color, which define their intended use.

4.4.2 Latex bonding materials

4.4.2.1 Advantages—Latex-based bonding agents are more economical in comparison to an epoxy or modified epoxy bonding agent, and are easier to handle on job sites. Clean-up of latex-based bonding agents is relatively easy before drying, but is more difficult after drying. Their very low viscosity results in relatively easy application. Latex bonding agents are classified as Type I or II per [ASTM](#)

C1059. Type I bonding agents can be applied to the bonding surface several days before placing the repair materials. Type II bonding agents are best suited for bonding when mixed with cement and water to produce a slurry. These are most commonly used with polymer-modified concrete mixtures.

4.4.2.2 Limitations—The bond strength of latex-based bonding agents is substantially less than that provided by epoxy-based bonding agents. Type I bonding agents should not be used in areas subject to water, high humidity, or structural applications. Leaving the concrete surface primed for an extended period of time can potentially contaminate the surface.

Type II bonding agents act as bond breakers once they have skimmed over or cured. Many manufacturers recommend Type II admixtures to be used as an additive in the concrete or mortar mixture to promote bond in place of a film-forming bonding bridge.

4.4.2.3 Applications—Latex bonding agents are applied with a brush, roller, or broom.

4.4.2.4 Standards—Latex systems are covered in ASTM C1059. Latex bonding agents are classified as Type I—Redispersible, and Type II—Nonredispersible.

4.4.3 Modified epoxy bonding materials

4.4.3.1 Advantages—Modified epoxy-cement bonding agents have been successfully used in the repair industry for many years. Commercial products classified as modified epoxy bonding agents are typically three-component products. In addition to the two-component resin, the third component in these systems is cement-based. These products are practical for spall-repair applications. Unlike neat epoxy resins, these products allow longer open working time for placement of material or concrete. The open times depend on temperatures. These bonding agents do not create a bond-inhibiting surface and recoating is possible if open times to place concrete or a repair material are exceeded. Manufacturers' recommendations should be followed to understand and work within the limitations of the product.

4.4.3.2 Limitations—Due to the slurry consistency of modified epoxy bonding agents, care should be taken to ensure appropriate mill thickness is achieved on the concrete.

4.4.3.3 Applications—Stiff brush is the most common method for limited size areas. Spray equipment can also be used to apply the material for larger areas. Modified epoxy bonding agents use water-based epoxy, so clean-up is easy with water, provided the material has not hardened.

4.4.3.4 Standards—There are no known industry standards that govern modified epoxy cement bonding agents. ASTM C882 has been used to test modified epoxy cements.

4.4.4 Cement-based bonding materials

4.4.4.1 Advantages—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 1-to-1 by mass. Water is added to provide a uniformly creamy consistency. Cementitious slurries are commonly used with PCC.

4.4.4.2 Limitations—Cement-based slurries dry quickly and can cause bond problems if allowed to dry prematurely. The skill and experience of the applicator is critical to

achieving proper performance. Where applicators provide a more flowable mixture for ease of application, the resultant higher w/cm may result in weaker material at the bond line.

4.4.4.3 Applications—Stiff-bristled brush is commonly used to apply the material.

4.4.4.4 Standards—There are no known standards that govern the use of cement-based slurries as a bonding agent.

4.5—Coatings on reinforcement

Some coatings applied to steel reinforcing bars have been proven to slow the rate of corrosion in laboratory environments and in the field (Popovic 1998). Commonly used coatings include epoxy, zinc, cementitious, latex cement, acrylic latex, or a combination of these materials.

4.5.1 Advantages—Coatings act as a barrier to prevent the steel from being exposed to air, water, or other corrosive chemicals. Because the steel is susceptible to corroding when exposed to air and water, the coating protects the steel from these elements. Additives are often used within coatings to further reduce or inhibit corrosion.

4.5.2 Limitations—The performance capability of coatings have been challenged due to holidays in the coating when field applied. The thickness of the coating applied can affect performance. In addition, once proprietary coatings are applied, corrosion activity can no longer be determined by standard testing procedures.

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

4.5.3 Applications—Coatings are typically applied to exposed reinforcing steel in concrete repairs. Exposed reinforcing steel bars should be dry and thoroughly cleaned, often by abrasive blasting, to remove all rust and concrete before installing a coating. Coating should be applied uniformly and fully encapsulate the reinforcing bar.

4.6—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, including various corrosion-resistant materials and conventional uncoated reinforcing steel.

4.6.1 Uncoated steel reinforcement—ACI 318 specifies steel reinforcement as deformed bars, spiral bars, and welded-wire reinforcement, with grades conforming to ASTM A615, A616, A617, or A706 for bars; ASTM A185 for plain wire reinforcement; and ASTM A497 for deformed wire reinforcement.

ACI 301 and 318 further specify minimum concrete cover requirements for various environmental exposures, maximum levels of chlorides, recommended w/cm of concrete mixtures, and other guidelines to improve concrete performance, all of which contribute to minimizing the potential for corrosion of conventional reinforcing structural elements.

4.6.2 Fusion-bonded epoxy-coated steel reinforcement—Epoxy coatings on conventional reinforcing steel (ASTM

A775) were introduced in the mid-1970s to act as a barrier to the elements that initiate corrosion, including oxygen, moisture, and chlorides. Though shown to be an effective method of reducing steel corrosion in concrete bridge decks, in-service performance of epoxy-coating varies depending on the moisture content of the concrete, 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.

4.6.3 Galvanized reinforcement—Hot-dipped galvanized reinforcing steel is also available as a method for reducing corrosion in reinforced concrete. **ASTM A767** specifies the requirements for zinc-coated (galvanized) reinforcing bars, with repair procedures identified by **ASTM A780**. The galvanizing process is based on the zinc coating providing a sacrificial corrosion layer around the reinforcing steel bars. At field cuts or damaged galvanization areas, application of zinc paint should be used to coat unprotected steel.

4.6.4 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 more resistant to corrosion in chloride environments. Stainless steel reinforcement can also be field bent or fabricated and is resistant to surface damage during handling and concrete placement. Initial cost is the primary disadvantage restricting its use, but life cycle cost analyses can indicate stainless reinforcement to be more economical over the long term.

4.6.5 Stainless-steel-clad reinforcement—Stainless-steel-clad reinforcement offers similar advantages to those of solid stainless steel bars, but at reduced costs. Field fabrication, including cutting, bending, and welding, is practical. Repairs at cut ends, however, are typically specified to be coated. Stainless-steel-clad reinforcement is not readily available.

4.6.6 Composite nonmetallic reinforcement—Fiber-reinforced polymer (FRP) composites are systems composed 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. FRPs are available in shapes similar to conventional reinforcement and are used for internal reinforcement (**ACI 440.1R**) and near-surface mounted (NSM) bars (**ACI 440.2R**). Fiber-reinforced polymer is also available in flat bar shapes or sheets to be applied on the external surface of concrete, generally with epoxy (**ACI 440.2R**). The following sections address the use of externally bonded FRP systems.

4.6.6.1 Advantages—FRP systems offer a high strength-to-weight ratio, allowing for a 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 atmospheric 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.

4.6.6.2 Limitations—Temperature and weather can influence the ability to install externally bonded FRP systems. Extreme temperatures and moisture should be avoided during the installation of such materials. Typically, in-service temperature should be below 150°F (66°C); however, some FRP materials may require lower in-service temperature limits. These systems should also be protected against UV exposure and fire.

The successful application of FRP systems depends 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 specifying 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.

4.6.6.3 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-strengthened systems have performed successfully through earthquakes and have generally performed as designed in field applications. For bonded applications such as beams, slabs, and walls, the concrete should be sound and proper surface preparation of the concrete substrate should follow the guidelines in **3.3**.

4.6.6.4 Standards—ICC-ES Acceptance Criteria **AC125** defines acceptance criteria for the use of FRP systems, including validation testing requirements, design methods, durability tests, and installation quality control. **ACI 440.3R** is a guide that covers the determination of material properties for individual material systems and references the appropriate ASTM test methods developed in the last decade to characterize FRP for concrete construction.

4.7—Material selection

As described in preceding sections, a wide variety of conventional and specialty materials are available for repairing concrete. This large selection provides a greater opportunity to match material properties with specific project requirements, but also increases the potential for selecting an inappropriate material. Bond and compressive strengths are clearly important material properties for many repairs, and this information is frequently provided by material suppliers. Some other material properties, however, can be of equal or greater importance (**Warner 1984**).

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

4.7.2 Drying shrinkage—Because most repairs are made on older portland-cement concrete 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 limit 28-day and ultimate drying shrinkage to 400 and 1000 millionths, respectively (McDonald et al. 2002). Shrinkage of cementitious repair materials can be reduced by using mixtures with low w/cm , the maximum practical size and volume of coarse aggregate, shrinkage-reducing admixtures, shrinkage-compensating concrete providing proper restraint, or by using construction procedures that minimize the shrinkage potential. Examples of application procedures that can be used to minimize the shrinkage of repair materials include dry-pack and preplaced-aggregate concrete. Shrinkage potential is much greater in thin repairs, such as 1.5 in. (40 mm) or less, when made with cementitious materials. Further, the shrinkage potential is generally believed to increase as the thickness decreases. Proper curing methods should be followed.

4.7.3 Permeability—High-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 repairs, 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 by freezing and thawing if the concrete substrate does not contain a properly entrained air-void system (USACE 1995a).

4.7.4 Modulus of elasticity—For repaired areas 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.

4.7.5 Chemical properties—Special attention has been directed in recent years to problems involving corrosion of embedded reinforcement. In the absence of chlorides or other corrosive species, a concrete pH within the range of 12 to 14 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 are used, because of constraints including cure time or strength requirements, additional protection for the existing reinforcement should be considered. This protection may 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.

4.7.6 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 called the anodic ring or halo effect. Refer to 3.2.3.1 for further discussion.

4.7.7 General considerations—The conditions under which the repair materials are applied and their anticipated service conditions are critically important considerations in material selection. Almost every repair project has unique conditions and special requirements, and only after these issues 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 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.

4.7.8 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 5—CONCRETE AND REINFORCEMENT REPAIR TECHNIQUES

5.1—Introduction

Various repair techniques are available to restore damaged and deteriorated concrete and reinforcing steel in structures. Each repair method has unique advantages, depending on specific project factors such as physical constraints of the repair area and costs. The selected repair method can have a significant influence on the long-term durability of the concrete repair. This chapter presents the various techniques for applying the materials presented in Chapter 4 to repair concrete and reinforcing steel.

5.2—Crack repair

The repair of cracks in concrete is a common concrete repair procedure, particularly so as a component protection system installation. Cracks can occur in concrete due to shrinkage, thermal changes, corrosion of embedded reinforcement, structural-related stresses, and long-term strain shortening. Only after determining the reason for the cracking can a proper repair technique be selected.

Structural cracks requiring repair should be able to reestablish load transfer across the crack and restore structural integrity. Common methods of repair include epoxy injection, gravity feed, and cement grouting. Active cracks, particularly those caused by thermal changes on exterior exposures, should be repaired to allow for future movement. Techniques such as chemical grouting, sealants, and strip-and-seal systems have proven effective for repair of active cracks.

Repairing active cracks on exterior exposures can be difficult. Most of the materials used for crack repair are temperature-sensitive and cannot be installed below 40°F (4°C).

Active cracks that move due to temperature variations tend to close in warm weather and open in cold weather. Both the contractor and the engineer should be aware of this before selecting the repair method. 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 the repair of active cracks perform better in compression than in tension (Emmons 1993; ACI 504R).

5.2.1 Epoxy injection—Epoxy injection is a crack repair technique that involves injecting epoxy resin into cracks under pressure. The injection procedure may vary, subject to the application and location of the work, as well as size and accessibility of the cracks. Horizontal, vertical, and overhead cracks (Fig. 5.2.1) require somewhat different injection approaches. Depending on specific parameters of a project, crack repair by epoxy injection can restore structural integrity and reduce moisture penetration through cracks as small as 0.005 in. (0.013 mm) in width.

For this method, injection ports are installed along the crack, and the face of the crack is sealed with an epoxy paste. Once the epoxy paste has cured, an epoxy resin is injected into the ports. Typical injection pressures range from 200 to 300 psi (1400 to 2100 kPa). Higher pressures of 300 to 500 psi (2100 to 3500 kPa) are commonly used for fine cracks. Low-pressure injection (15 psi [105 kPa] or less) is generally used in delicate applications such as architectural surfaces. Using lower pressure permits the use of easily removable sealants as the capping material. The easy removal of surface seals allows for maintaining the appearance of the injected surface.

Cracks may be injected from one or both sides of a concrete member. Where access is limited to only one side, installation procedures may include variations in epoxy viscosities, injection equipment, injection pressure, and port spacing to ensure full penetration of epoxy into the cracks. If a crack is subject to subsequent movement, an epoxy repair may not be appropriate unless the movement can be accommodated by adjacent joints.

Specification requirements for epoxy injection are contained in ACI 503.7. Refer to ACI 224.1R and ACI RAP-1 for additional information on epoxy injection of cracks and other crack repair procedures. Epoxy injection procedures and techniques in construction are also detailed in Trout (2006). Refer to International Concrete Repair Institute (ICRI) 210.1 for further information on verifying field performance of epoxy injection of concrete cracks.

5.2.1.1 Vacuum injection—Cracks may be filled with epoxy that is drawn in by creating a negative pressure inside the cracked concrete member. To successfully employ this technique, all exposed crack surfaces should be sealed sufficiently to create and maintain negative pressure inside the member.

5.2.1.2 Gravity feed—This technique commonly uses epoxies or high-molecular-weight methacrylates (HMWM) poured along the horizontal surface of a crack (Fig. 5.2.1.2). Flat squeegees are typically used to force the resin into the



Fig. 5.2.1—Overhead epoxy injection of crack.

crack. In some situations, an artificial dam with an acrylic caulk or putty is created for a temporary ponding effect. Polymer material is poured into the dam, creating a head pressure to fill the crack. The polymer material and the caulk need to be removed quickly, before they start to set up. Failure to remove the excess polymer material and the temporary caulking will create an undesirable difference of elevation at the concrete surface. This method is also used for flood coating. This method is strictly limited to horizontal surfaces. Refer to RAP-2 for additional information on crack repair by gravity feed with resin, and RAP-13 for information on methacrylate flood coat.

5.2.2 Injection grouting—Grouting is a common method for filling cracks, open joints, honeycombed concrete, and interior voids. Grouting materials include chemical, cement, polymer, polymer-cement, epoxy, polyurethane, and methacrylate. Grouting can strengthen a structure, stop water movement, or both. Before selecting a grouting repair program, the objectives of the 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 (5.6).

5.2.2.1 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 typically 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 grouting operator (Fig. 5.2.2.1). Chemical grouts may contain various inert fillers to modify physical properties, such as consistency, heat generation, and volume. Chapter 4 describes materials for chemical grouting.

Polyurethane chemical grouts are commonly used to repair leaking cracks, some of which could be leaking a significant amount of water. These grouts are semiflexible



Fig. 5.2.1.2—Horizontal gravity feed of crack.

and can accommodate some change in crack width without failure. Polyurethane chemical grouts can treat cracks 0.005 in. (0.12 mm) and greater in width and are injected at high pressures. Chemical grouts are generally not suitable for structural repairs.

Chemical grout should be injected from the surface or interior of the member in the same general manner as cement grouts with the exception that, for unfilled grouts, the port sizes may be 1/8 or 1/4 in. (3 or 6 mm) in diameter, and the port devices may be mechanically anchored or cemented into place (ACI 224.1R; ACI 503.4; ACI 503.6R; ICRI 340.1).

5.2.2.2 Cement grouting—Grout may be injected into an opening from the surface of a concrete member or through holes drilled to intersect the opening in the interior of the member (Fig. 5.2.2.2). When grout is injected from the surface, short entry holes or ports, a minimum of 1 in. (25 mm) in diameter and a minimum of 2 in. (50 mm) deep, are drilled into the opening. The surface of the opening is sealed between ports with a portland cement or resinous mortar. Depending on anticipated grouting pressures, short pipe nipples can be cemented into holes for the grout hose connections. If the ports are drilled after sealing the openings, a hand-held, cone-shaped fitting on the grout hose may be adequate for pressure less than 50 psi (350 kPa). Where cracks or openings extend through a structural member, such as a wall, the opening is typically sealed and ported on both sides. When appearance is not a factor, openings may often be sealed with caulking or with cloth fabric that passes water



Fig. 5.2.2.1—Chemical grouting.



Fig. 5.2.2.2—Cement grouting.

or air yet retains 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, ports should not be spaced farther apart than the desired depth of grout penetration. Before grouting, the openings should be flushed with clean water, subject to the same procedure that is to be used in grouting. Flushing is necessary 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 interconnections, and to familiarize the grouting crew with the existing conditions.

Grouting is started at one end of a horizontal crack or at the bottom of a vertical crack and continues until grout is observed at the port adjacent to or above the one being pumped. When this occurs, grout is then injected into the next port and continues until grout again shows at the adja-

cent open hole. Each flowing port should be plugged before moving to the next injection location. Progress should be monitored on the far side of the member, if accessible, and ports or valves should be closed as necessary. Grouting is typically started with a relatively thin grout, thickened as quickly as possible to the heaviest consistency that can be pumped without blockage, as determined by the grouting operators.

5.2.2.3 Polymer and polymer-cement grouting—Polymer grouting is typically used to repair dormant cracks 0.25 in. (6 mm) and greater in width. Polymer grouts can be hand-applied and packed into large cracks, regardless of orientation, or they can be poured into cracks in horizontal slabs. Polymer grouts bond well to concrete and have low shrinkage, resulting in a watertight repair in dormant cracks. Similar to epoxy resins, polymer grouts are suitable for cracks requiring structural repair. If polymer grouts are used in cracks that experience movement, it can be expected that parallel cracks may appear adjacent to the repaired cracks. Materials of varying consistencies are readily available to repair cracks in vertical, overhead, and horizontal applications.

Polymer-cement grout is a mixture consisting primarily of cement; fine aggregate; water; and a polymer such as acrylic, styrene-acrylic, styrene-butadiene, or a water-borne epoxy. The consistency of this material can be adjusted from a stiff material suitable for hand-packing large cracks on overhead and vertical surfaces to a pourable consistency suitable for gravity feeding cracks in horizontal slabs. These materials may provide better physical properties such as bond strength, compressive strength, and flexural strength than conventional water-based cement grouts that do not use polymers. The potentially high shrinkage characteristics of polymer-cement grouts, however, may make it difficult to obtain a watertight repair when compared with polymer grouts. Additionally, these materials are not as chemically resistant as polymer grouts.

5.2.3 Sealants—Polyurethane sealants, also known as urethane sealants, are formulated for use in sealing both active and dormant cracks and joints in concrete. Sealants are commonly used to mitigate the leakage of water through cracks and joints (Fig. 5.2.3). Various products are available in numerous colors for use in horizontal or vertical surfaces, as well as in conditions of continuous water immersion. These sealants can be used in relatively wide cracks and joints, generally up to 2 in. (50 mm). Additionally, the movement capability of sealants can accommodate normal dimensional changes in concrete due to structural and thermal stresses.

Silicone sealants typically have higher elongation properties than polyurethane sealants, making them suitable for wider and active joints up to 3 to 4 in. (75 to 100 mm). These sealants are highly resistant to UV light, but should not be used for immersion applications. All sealants, both polyurethane and silicone, are not suitable for structural repairs. Additionally, sealants require proper joint design (refer to [ACI 224.1R](#)) to ensure that the materials perform properly,



Fig. 5.2.3—Sealant application.

and installation may involve the routing or widening of smaller width cracks and joints.

5.2.4 Strip-and-seal systems—Strip-and-seal systems consist of a flexible sheet, such as synthetic rubber, that spans over a crack or joint and is adhered to the structure on each side of the crack or joint using a suitable adhesive. These systems are designed to span over the crack or joint to prevent leakage of water. These systems may be used to treat cracks or joints of any width, even though the crack or joint is not actually filled or repaired. The elongation properties of these systems are excellent, making them appropriate for use on active cracks and joints. Some strip-and-seal systems are highly resistant to UV light; thus, they do not chalk or weather. Some systems have a high resistance to aggressive chemicals. Generally, these systems do not require any special worker skills or equipment to install.

Strip-and-seal systems are not suitable for structural repair; they are designed to be applied to the positive-pressure side of a structure, thus preventing leakage of liquid into or out of the structure.

Strip-and-seal systems are particularly suited for stopping water leaks that may otherwise be difficult to seal with other materials. These systems may be applied vertically, horizontally, or overhead. Refer to [ACI 546.3R](#) for additional information on strip-and-seal systems.

5.3—Concrete replacement

5.3.1 Top surface (horizontal repair—partial and full depth)—Horizontal surface repair is common on elevated slabs and slabs-on-ground. Delaminated areas are located by sounding the concrete surface. Performance of top surface repairs relies on adequate surface preparation; refer to [Chapter 3](#). Concrete surface moisture conditions should be as specified by the material manufacturer before placing the repair material. Top surface repair may also extend full depth (Fig. 5.3.1), requiring formwork to perform the repair. Cast-in-place concrete should be cured in compliance with [ACI 301](#) and proprietary repair materials should be cured in accordance with manufacturer’s recommendations. Refer to [RAP-7](#) for additional information on repair of horizontal concrete surfaces.

5.3.2 Form and pour (horizontal repair)—The form-and-pour placement technique is a multistep process consisting



Fig. 5.3.1—Horizontal repair at full depth.

of surface preparation of repair area, formwork construction, and placement of repair materials into a confined-space formwork (Fig. 5.3.2a). Benefits derived from formed repairs include improved curing due to the protection and moisture retention provided by the formwork. Watertight forms should be used to allow for pre-soaking of substrate and to avoid leakage when using fluid repair mortars. Repair materials are placed in the cavity between the formwork and the prepared substrate with buckets, pumps, chutes, or buggies (Fig. 5.3.2b).

The form-and-pour technique allows the use of many different cast-in-place repair materials. Placement can be either a partial- or full-depth repair. Depending on the consistency of the repair material, consolidation is accomplished by vibration or rodding. When a repair material has extremely high slump or is self-consolidating, additional steps to consolidate the repair material may not be required. High-powered vibrators should not be used to consolidate fluid repair materials or material with a slump of greater than 7 in. (175 mm). Refer to **ACI RAP-4** for additional information on surface repair using form-and-pour techniques.

5.3.3 Form and pump (vertical and overhead repair—partial depth)—Form-and-pump is a repair method to replace damaged or 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. The form-and-pump repair procedure involves constructing formwork and pumping repair material into the cavity confined by the formwork and the concrete substrate. The repair materials are mixed and pumped via a discharge line into the formwork until the confined cavity is filled. Formwork should be designed and constructed to resist hydrostatic pressure induced by placement and consolidation of the repair material.

The cavity and formwork design should also provide for venting of air. Venting can be accomplished by the removal profile of the prepared concrete, vent tubes, or drilled holes in the existing concrete. Filling the cavity should begin at



Fig. 5.3.2a—Form-and-pour method.

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 and continues until the cavity is completely filled (**Emmons 1993**). Once filled the forms should be pressurized to 20 to 80 psi (0.14 to 0.55 MPa) depending upon the size, geometry, and number of venting ports; the repair material is consolidated around the reinforcing steel and driven into the crevices of the prepared substrate to improve bond.

Success of this repair methodology is dependent on the use of proper pumping equipment. Three commonly used pump technologies are rotor stator, peristaltic, and swing tube piston. The peristaltic and swing tube piston types are capable of pumping repair materials containing the large aggregate necessary for deep or mass placements. Before work begins, a pre-job meeting and mockup should take place to assure proper equipment usage. Refer to **ACI RAP-5** and **ACI 309R** for additional information on surface repair using the form-and-pump technique.

5.3.4 Hand applied (vertical and overhead repair)—The hand application method can be used to repair deteriorated concrete or to resurface vertical and overhead concrete surfaces. Trowel application of repair materials by hand is suitable for shallow surface repairs, especially in areas with limited or difficult access.

5.3.4.1 Troweling—The hand-troweling of repair materials can be used for shallow and limited areas of repair. Trowels or other suitable tools are used to transport and press the repair material into the prepared substrate to develop contact and bond with the existing concrete. 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



Fig. 5.3.2b—Form-and-pour method.

systems should not be used when reinforcing steel is exposed and undercut due to the difficulty of consolidation of the repair material around and behind the reinforcing steel. The paste of the repair material is commonly used as the bonding material or scrub coat; refer to 5.3.9. Where multiple layers are needed to obtain the total thickness of the repair, intermediate surfaces should be roughened to help bond subsequent layers. For most applications, the prepared surface of the repair area should be saturated surface-dry (SSD) at the time of material application.

A wide variety of proprietary repair mortars and concretes have been modified with chemicals and thixotropic agents. The placement techniques recommended by the material manufacturers do not always conform to accepted placement techniques for portland-cement mortar and concrete. This is particularly true for thin sections, such as 1/8 in. (3 mm), and for vertical and overhead applications. The addition of chemicals and thixotropic agents that permit deviations in placement techniques may compromise some performance properties, including shrinkage, bond strength, and coefficient of thermal expansion. The engineer and contractor should consult with the manufacturer to ensure that the material's performance capability and limitations meet the project criteria before using these materials.

Successful use of trowel-applied repairs is dependent on the surface preparation and the skill of the installer. Proper troweling technique should be used to prevent the entrapment of air at the bonding surface, which can cause reduced bond strength. Additionally, proper curing of portland cement repair mortar is important so that the repair material does not dry before hydration is complete. Special curing provisions are advisable for some proprietary repair materials or where accessibility is difficult (ACI 308R and 308.1). Refer to ACI RAP-6 for additional information on vertical and overhead spall repair by the trowel-applied method.

5.3.4.2 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 w/cm , properly proportioned and applied repairs exhibit low shrinkage, good strength, durability, and watertightness.



Fig. 5.3.5—Shotcrete method.

Due to the extremely low water content of the mortar, curing becomes even more critical to prevent the loss of moisture necessary for cement hydration.

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

5.3.5 Shotcrete (vertical and overhead repair—partial and full depth)—Shotcrete is concrete or mortar pneumatically conveyed at high velocity through a hose onto a surface (Fig. 5.3.5). 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's surface. The shotcrete process is capable of placing repair materials onto vertical and overhead applications without the use of forms and can routinely convey 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 shotcrete 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. Both processes place repair materials for normal construction requirements. In addition to conventional portland-cement concrete, the shotcrete process can also be used to place polymer-cement concrete, fiber-reinforced concrete (FRC), steel or synthetic fibers, and concrete containing silica fume and other pozzolans. **ACI 506R** provides information on the two shotcrete processes and their proper application. Refer to **ACI RAP-12** for additional information on concrete repair by shotcrete application.

The application of repair materials by the shotcrete process should be considered when access to the site is difficult or where significant areas of overhead or vertical repairs are required. The elimination of formwork provides economy when compared to other repair techniques. 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 6**.

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. An ACI-certified, or approved equal, nozzle operator should be used for the shotcrete operations. A properly trained nozzleman is necessary to assure proper consolidation of the shotcrete around the reinforcing bars and to avoid rebounded sand pockets that can lead to reflective cracking.

5.3.6 Low-pressure spraying (vertical and overhead repair—partial depth)—Low-pressure spray repair materials are typically supplied as prepackaged mortars. The spray is applied using small concrete pumps or heavy-duty grout pumps to force the low-slump mortar through a hose. Air is added at the nozzle to impel the mortar. Bond with the prepared substrate is achieved through a combination of proper surface preparation, low-velocity impact, and the material properties of the prepackaged mortar. Prepackaged low-slump mortars should not be rapid-setting products commonly used for hand applications.

Materials should be applied in wet-on-wet lifts. When conducting the repairs, consideration should be given to ambient and substrate temperatures. Work stoppages should be avoided when material is in the hose. Refer to **ACI RAP-3** for additional information on spall repair by low-pressure spraying.

5.3.7 Preplaced aggregate (vertical and overhead repair—partial depth)—Preplaced-aggregate concrete (PPAC) involves the placement of coarse aggregate in a formed cavity. A mixture of portland cement and sand or other appropriate repair material is then pumped into the formed repair to fill the voids. Concrete drying shrinkage is reduced because of point-to-point aggregate contact and reduced paste volume. Preplaced-aggregate concrete can also be used for horizontal repairs as long as the grout is injected from the bottom of the repair either by pumping from underneath the repair area or using a tremie pipe and withdrawing it as grout fills the area.

Preplaced-aggregate concrete is frequently used on underwater repair projects. The concrete mortar is pumped from the bottom of the repair, displacing the water as it rises (**ACI 546.2R**). Information on the preplaced-aggregate method for concrete construction is provided in **ACI 304.1R**. In addition, refer to **ACI RAP-9** for additional information on spall repair by the preplaced-aggregate method.

5.3.8 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 repairs underwater. These repairs are presented in **ACI 546.2R**. Placing concrete under water by tremie and by pump is covered in detail in **ACI 304R**.

5.3.9 Bonding repair materials to substrate—Bonding materials are used to enhance the bond of repair materials to the existing prepared concrete substrate. Repair materials may or may not require a separate bonding material. In either case, the success of the repair depends on achieving intimate and continuous contact between the repair material and the prepared substrate. This can be achieved by vibration, pneumatic application, high fluidity, and troweling pressure in conjunction with maintaining an optimum surface profile and moisture condition.

In cases where a bonding material is to be used, application of the bonding material to the prepared substrate should be timed with the placement of the repair material. Bonding materials applied to substrates may begin setting or curing prematurely, creating a bond breaker with the repair material. Cement-based bonding materials are sprayed or broomed, whereas epoxy- and latex-based systems are brushed, rolled, broomed, or sprayed (Fig. 5.3.9). Whether the repair material is self-bonding or a bond material is used, tests should be conducted to ensure that adequate bonding is achieved. In-place tensile pull-off tests should be performed to evaluate the bond of repair material to the prepared substrate (**ASTM C1583** and **ICRI 210.3**). This testing provides for the evaluation of the adequacy of surface preparation, evaluates the tensile strength and bond strength, and assesses the failure location (**5.6**).

5.4—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. A licensed design professional (LDP) should be consulted for any reinforcement repairs.

After deteriorated and damaged concrete is removed, it is necessary to expose the reinforcing steel, evaluate its condition, and prepare the reinforcement for repair if required. Proper inspection and preparation of the reinforcement helps to assure satisfactory long-term performance of the repair solution.

It is not uncommon that a concrete repair involves replacing only deteriorated concrete at spalls or delaminations. This approach often leaves chloride-contaminated concrete surrounding the repair area, creating a highly conducive environment for continued corrosion of the reinforcement. Such repairs may actually promote corrosion of



Fig. 5.3.9—Application of bonding material before overlay application.

the reinforcing steel in the surrounding concrete. Deterioration of concrete surrounding a repair area is known as the halo or anodic ring effect and is discussed more extensively in Sections 3.2.3.1 and 6.2.5.

5.4.1 Inspection of reinforcing steel—Following concrete removal, reinforcing steel should be cleaned as described in 3.3.3 and carefully inspected. Deteriorated or damaged reinforcement may need to be replaced or supplemented as directed by the LDP. Project specifications and drawings should include criteria for the repair or replacement of exposed reinforcement where required, and provide splice lengths and mechanical connection criteria as required by ACI 562.

Cleaned reinforcing steel may corrode lightly between the time it is cleaned and the time that repair material 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 typically required. If the rust is loosely bonded so as to inhibit bond between the steel and the repair material, the reinforcing bars should be cleaned again before the repair material is placed. A protective coating may also be applied to the reinforcement after the initial cleaning has been completed; refer to 5.4.2.3.

Two types of reinforcement commonly used in concrete structures are mild reinforcing and prestressing steel. Different repair procedures are warranted for each type of reinforcement and depend on the condition of the exposed reinforcement and its impact on structure integrity.

5.4.2 Mild reinforcing steel—For mild reinforcing steel, repair alternatives can involve the replacement or supplementing of deteriorated and damaged reinforcement. The method selected is an engineering decision based on the function of the reinforcement and the required structural impact to the reinforced member. Refer to 4.6 for different types of new reinforcing steel available.

5.4.2.1 Replacement—One method of repairing reinforcement is to cut out the deteriorated or damaged area and splice in replacement bars. Great care should be taken when removing reinforcing steel that is under stress. Avoid cutting of stressed reinforcement when possible. Unloading of members may be necessary to relieve stress in reinforcement to remove it safely. Replacement of reinforcement is common for large repair areas where deteriorated or damaged reinforcement is totally removed full-length and replaced with new reinforcing steel of the same size and length at the same location of the removed reinforcement. It may be feasible to leave deteriorated or damaged reinforcement in place if one is lap splicing with additional new reinforcing steel adjacent to the deteriorated or damaged area of reinforcement (4.4.2.2). Replacement sections of new reinforcement should match the size of the existing reinforcement and be welded or coupled to the cut ends of the existing reinforcement. If welded splices are used, welding should be performed in accordance with ACI 318 and the American Welding Society (AWS D1.4). All welding should be performed by an AWS-certified welder 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 the bars are not always accessible. Caution should be taken for welded splices on bars larger than No. 8 (No. 25) because the heated embedded bars could expand and crack the surrounding concrete. Special attention is necessary when welding bars adjacent to unbonded or bonded prestressing steel. 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.

5.4.2.2 Supplemental reinforcement—This alternative is often selected when the reinforcement has lost significant cross-sectional area, the existing reinforcement is inadequate, or the existing member is 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 by the LDP on a case-by-case basis following ACI 562 criteria. The deteriorated or damaged reinforcement to remain should be cleaned in accordance with 3.3.3 and the concrete removed to allow placement of the supplemental bar beside the existing bar with adequate cover and a minimum of 3/4 in. (19 mm) space below bars. The length of the supplemental bar should 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 of the two bars, as specified in ACI 318. Supplemental bars may also be lap welded to the existing bars in accordance with AWS D1.4. When supplemental reinforcement is placed to strengthen a section, the techniques discussed in Chapter 7 should be considered. Supplemental reinforcement can also be installed to control shrinkage of the repair material, restore structural capacity, and provide mechanical anchorage of the concrete repair materials if a bond failure occurs. Mechanical anchorage is particularly important for repairs to building façades and



Fig. 5.4.2.3—Coating reinforcing steel.

overhead repairs. For the installation of supplemental reinforcement, embedment into the existing concrete may be required. The effectiveness of grouts to embed supplemental reinforcing steel in hardened concrete should be evaluated (Best and McDonald 1990).

5.4.2.3 Coating of reinforcement—New uncoated bars and existing bars that have been cleaned may be coated with an epoxy, polymer-cement slurry, zinc-rich epoxy, cementitious coating, latex cement, or acrylic latex for protection against corrosion (Fig. 5.4.2.3). Some materials may require multiple coats, but in general the coating should be applied at a total thickness less than 12 mils (0.3 mm) to minimize loss of bond development at the deformations. Reinforcing bars that have lost original deformations as a result of corrosion and cleaning will have reduced bond with most repair materials. Coating of these bars with materials such as epoxy further reduces their bond with repair materials. Care should be taken during the coating process to avoid spillage onto the substrate concrete, as epoxy or zinc-rich coatings act as a bond breaker between the new repair material and the existing concrete. Materials such as polymer-cement coatings may contain corrosion inhibitors for further protection of the reinforcing steel and may also act as bonding materials for bonding the new repair material to the existing concrete. Embedded galvanic anodes are also used for protection against corrosion instead of coatings.

5.4.3 Prestressing steel—Prestressing steel in structural members is of two basic types: bonded and unbonded. Deterioration or damage to prestressing steel can result from impact, overload, corrosion, or fire. Fire can soften cold-worked, high-strength prestressing steel, altering its strength and hardness, resulting in possible failure. Repair options for bonded strands are different than those for unbonded strands and wires. For external repair and strengthening options, refer to Chapter 7.



Fig. 5.4.3.2a—Failure of unbonded tendon.

5.4.3.1 Bonded strands—For prestressed bonded strands, only the exposed and damaged or deteriorated section is detensioned during repair. The repair procedure requires replacing the damaged or deteriorated section with new strand connected to undamaged or nondeteriorated ends of the existing strand. The new strand section for the exposed length should be post-tensioned to achieve the force level of the existing bonded strand. Refer to ACI 423.8R for further information on the repair of grouted multi-strand and bar tendon systems.

5.4.3.2 Unbonded systems—Unbonded systems include buttonhead (wires) and monostrand (strand) tendons. Strands are installed inside sheathings (paper wrapping for wire systems) embedded in the concrete member. Strands are protected against corrosion by the sheathing, corrosion-inhibiting material such as grease, or both. However, if the grease did not completely fill the annular space between the strand and sheathing, the grease did not contain adequate rust-inhibiting properties or the sheathing was loose-fitting, unbonded strands or wires are susceptible to corrosion and deterioration. Corrosion of the strand or wires and anchorages has been the primary cause of unbonded tendon failure (Fig. 5.4.3.2a). Shoring in the span being repaired and adjacent spans up to several bays away may be required before and during unbonded system repair. Unbonded strands and wires may need to be detensioned before repair and retensioned after repair to restore the structural integrity of the member.

Unbonded tendons (monostrand system) can be tested to determine the existing force in the strands and to assess their ability to support the design load. This can be performed at the live end by attaching a chuck and coupler to an exposed end of the strand, following the removal of the grout pocket and performing a lift-off test. This test may require at least 1.5 in. (40 mm) of rust-free strand at the grout pocket. An in-place strand tension test can also be performed to measure the tensile forces in the strands (Fig. 5.4.3.2b). PTI DC80.3-12/ICRI 320.6 (PTI/ICRI 2012) provides guidance for the evaluation of unbonded post-tensioned concrete structures.

To perform a repair, the deteriorated portion of a tendon is exposed by removing the concrete and sheathing. The strand or wires are then detensioned and cut on both sides of the

deteriorated portion where sound strand or wires are evident. Use caution when detensioning and cutting the tensioned or partially tensioned strands or wires. The installation of a lock-off device (monostrand system) may be necessary to maintain forces in the strand beyond the repair area and enable the repair to be completed. The deteriorated portion



Fig. 5.4.3.2b—In-place tension testing of unbonded tendon.

of the tendon that is removed and replaced with a new section is then spliced to the existing tendon at the location of the cuts. The repaired strand or wires are then stressed to provide the required forces in the tendon (Fig. 5.4.3.2c). Tendon repair techniques use both buttonhead and monostrand hardware/equipment, as well as combination systems, depending on the type of unbonded system in the structure. Encapsulated anchorages and other waterproofing devices should be considered for unbonded system repairs to provide added corrosion protection (Fig. 5.4.3.2d).

Removal of a deteriorated or damaged unbonded strand (monostrand system) from the sheathing is possible, but sometimes difficult. Strands of the same diameter should be used for the repair, if possible. When the tendon is removed for replacement, it may be necessary to insert a smaller-diameter strand of higher-strength material that is capable of achieving the required stressing force in the tendon.

Refer to **ACI 423.4R** and PTI DC 80.3-12/ICRI 320.6 (**PTI/ICRI 2012**) for further information on the repair of unbonded post-tensioned concrete structures.

5.5—Anchorage

Anchors are often used in conjunction with reinforcement repair to prevent dislodging of the reinforcing steel from the substrate concrete in the event of a bond failure. Anchors are commonly used in the repair of overhead and vertical surfaces, such as façades, to ensure that the repair area does not dislodge from the structure. Noncorrosive anchors, such as stainless steel anchors, are often specified for repair areas susceptible to corrosion, especially in areas of inadequate concrete cover.

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

5.5.1 Post-installed anchors—Post-installed anchor systems use anchors installed in predrilled holes. These



Fig. 5.4.3.2c—Tendon splices in place (left) and being restressed (right).

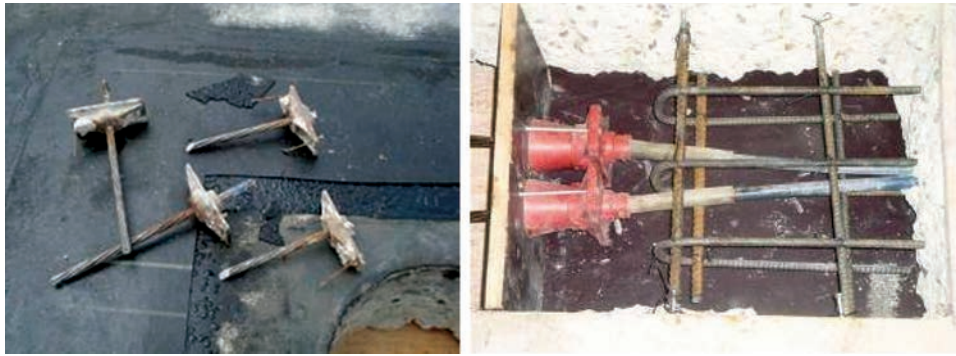


Fig. 5.4.3.2d—Unbonded tendon anchors removed (left). Encapsulated replacement anchors shown secured in place prior to recasting in concrete (right).

anchors can be divided into two general types: bonded and expansion anchors.

5.5.1.1 Bonded anchors—Bonded anchors include both grouted (headed bolts or a variety of other shapes installed with a cementitious grout) and chemical anchors (typically threaded rods installed 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 along the wall of the drilled hole (McDonald 1989). Chemical systems such as epoxies, polyesters, and vinyl esters, have different setting and performance characteristics that should be considered when specifying to meet design intent. The LDP should be diligent when specifying chemical anchors in an underwater or moist environment by verifying that the chemical adhesive has been tested and satisfactorily performs in a representative environment. Epoxy anchors may have significant limitations when used at elevated temperatures, such as in the case of a fire, particularly if the anchor is in tension. The use of epoxy anchors for supporting sustained loads should be avoided because of creep issues (FHWA 2008). Performance characteristics of adhesive anchoring systems should be evaluated before using adhesive anchors in sustained tensile-load applications (McDonald 1998; NTSB 2007).

5.5.1.2 Expansion anchors—Expansion anchor systems, sometimes called mechanical anchors, include torque and 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 undercut anchors, strength is derived from keying into an undercut at the bottom of a drilled hole. Refer to ACI 355.2 and ACI 318 for additional information on post-installed mechanical anchors in concrete.

5.5.1.3 Anchorage of reinforcement into existing substrate—When repair design requires anchors for additional or supplemental reinforcement, or additional reinforcement is anchored into the substrate concrete, care should be taken to thoroughly clean the inside of drilled



Fig. 5.5.1.3—Coring.

holes. Two methods of drilling holes are rotary percussion and core drilling (Fig. 5.5.1.3). Each method requires a different means of cleaning and preparation.

a) **Rotary percussion**—This method uses electric or pneumatic drills for hole drilling that typically create large amounts of dust and concrete fines. Care should be taken to remove all dust and fines using compressed air, vacuums, sandblasting, or water blasting. Standard pneumatic drills use a hollow drill rod through which the cuttings are continuously expelled from the hole, resulting in a clean, rough-surfaced hole. In addition, the use of a chisel-tipped bit will produce relatively larger cuttings. Conversely, electric hammer drills do not expel the cuttings, but rather beat them

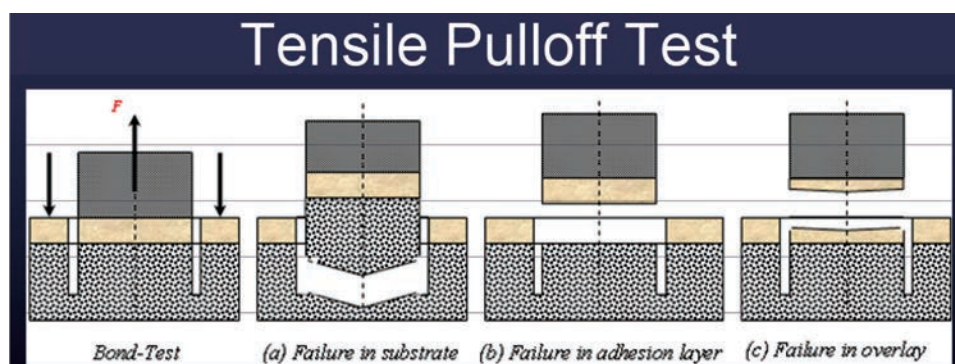


Fig. 5.6.4—Pull-off testing.

up into a fine dust that adheres to the wall of the hole, requiring further cleaning. If water blasting is used, care should be taken to not leave slurry residue inside the hole. This method creates a slightly roughened profile inside the hole.

b) *Core drilling*—Core-drilled holes create several conditions that need to be addressed before installing bonded anchors. Core drilling produces slurry that may compromise the bonding surface when it dries within the drilled hole. Additionally, cored holes typically create a smoother surface than rotary percussion-drilled holes. The combination of water blasting, vacuuming, air-blasting, and/or sandblasting may be necessary to remove the slurry and roughen the inside surface of cored holes.

5.5.2 Cast-in-place anchors—Cast-in-place anchor systems include embedded, non-adjustable anchors of various types and shapes, bolted connections, and adjustable anchors set in place before placing concrete.

5.5.3 Anchor strength—Anchor strength and long-term performance depend on a variety of factors that should be evaluated, including:

- a) Yield and ultimate material strength;
- b) Hole diameter and drilling system used;
- c) Embedment length;
- d) Annular gap between the anchor and the drilled hole for post-installed anchors;
- e) Concrete strength and condition;
- f) Type and direction of load application (static, dynamic, tension, shear, bending, or combined loading);
- g) Spacing to other anchors and edges;
- h) Temperature (for chemical and epoxy anchors);
- i) Hole cleaning;
- j) Mode of failure of the anchor system (concrete breakage, steel breakage, slip or pullout);
- k) Environmental conditions for moisture and corrosion resistance and creep.

Reference Appendix D of **ACI 318-11**.

Site testing for verification of performance should be conducted 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, the manufacturer's installation instructions should be followed to ensure proper anchor performance.

5.6—Quality control and assurance

Quality control and assurance procedures are necessary during and after placement of the repair material. Various testing and evaluation methods (**ACI 228.2R** and **ICRI 210.4**) can provide valuable information relative to the success of the repair. Performance monitoring of repairs should also be considered. Some common test methods and evaluation techniques performed during and after repairs are outlined in the following.

5.6.1 Mockups—Sample repairs should be performed so that the owner, contractor, and LDP can agree on repair procedure, quality of repairs, and the post-repair aesthetics before starting the repair program. This step will reduce unrealistic expectations as to the performance and appearance of the repair areas.

5.6.2 In-place testing—Testing should be performed for quality control of repairs and may include strength testing, delamination, and debonding identification, and internal cracking and void detection of the repair materials. Test methods that establish the presence of deterioration or damage can also be used to determine that the identified condition is no longer present after repair.

5.6.3 Coring and laboratory testing—Coring of existing repair areas may be used to perform accelerated aging or petrographic analysis for assessing durability of the concrete repair material. Physical tests on cores can also be used to determine strength characteristics of the repair.

5.6.4 Pull-off testing (*ASTM C1583* and *ICRI 210.3*)—Tests can be performed to assess the quality of bond and determine bond strength of the repair material (Fig. 5.6.4). These tests can be used to determine if repairs meet specified bond strength requirements and verify that surface preparation is adequate.

CHAPTER 6—PROTECTIVE SYSTEMS

Protective systems consist of materials and methods which extend the service life of concrete structures by reducing their exposure to moisture and other factors, reducing corrosion of metals embedded in concrete, increasing surface abrasion or impact resistance, or improving the structure's resistance to other deleterious influences. Protective systems limit the intrusion of moisture, chloride ions, and contami-

nants into the concrete by using surface treatments or minimize corrosion by electrochemical principles.

6.1—Introduction and selection factors

The primary reason for providing a protection system is to extend the life of the structure or subsequent repairs. A repair program should determine the causes for concrete deterioration or failures such that the protection systems can be designed to improve the performance of the repairs as well as the original concrete by moderating the underlying causes of concrete deterioration.

There are a multitude of methods and materials available to protect concrete. Refer to International Concrete Repair Institute (ICRI) 320.2R or ICRI 330.1 for additional information. The following factors should be included in the decision-making process for the appropriate protection system.

6.2—Typical problems that can be mitigated with protection systems

When developing the protection system for a project, the factors affecting the performance of the original concrete and completed repairs should be evaluated. Misplaced reinforcing steel, low concrete cover, high chloride content, low-quality concrete, cracking, and carbonation are some factors that contribute to corrosion of embedded reinforcing steel. A protective system should be designed to reduce the effects of these factors and should minimize the effects of detrimental conditions that could result in a failure of the repairs or contribute to other concrete deterioration. The following are some common problems that should be considered in a repair program.

6.2.1 Low-quality concrete 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. High-quality, durable, properly consolidated concrete with adequate cover provides an environment that should protect the embedded reinforcing steel for years without any protection system before repairs are required. Conversely, low-quality concrete with deficiencies such as excessive cracking and honeycombing, 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 types of deterioration of the structure. It is best to remove deteriorated concrete as part of a repair program, when and where possible. A properly selected protection system can improve the durability of low-quality concrete, enhance the performance of good concrete, and extend the life of repairs.

6.2.2 Misplaced reinforcing steel—Misplaced reinforcing steel is a major contributing factor to corrosion (ACI 222R). This commonly occurs at reveals on walls or columns, in exposed slabs with variable thickness at the edge and at hooked bars perpendicular to exposed concrete slab edges, if the reinforcing steel is placed too close to the surface. The repair program may provide for the lack of cover over the reinforcement. This can be accomplished with build-outs, where appropriate. Barrier coatings on the reinforcing steel or concrete surface, frequently in combination with other

techniques, can provide added protection against corrosion. Cathodic protection, chloride extraction, and corrosion-inhibiting additives in repair materials may also be useful to provide added protection against corrosion.

6.2.3 Water penetration—Water may penetrate into concrete by hydrostatic pressure, moisture vapor pressure, capillary action, wind-driven rain, or any combination of these. Cracks, porous concrete, structural defects, or improperly designed or constructed joints all contribute to the penetration of moisture into concrete. Water penetration into or through concrete can contribute to corrosion of reinforcement, freezing-and-thawing damage, leakage into the interior of the structure or occupied levels beneath decks, contamination of potable water, contamination of groundwater from wastewater or hazardous wastes, and possible structural damage.

6.2.4 Carbonation—Carbonation is the reduction of the alkalinity of concrete caused by the absorption of carbon dioxide and moisture into the concrete. In normal concrete, the reinforcing steel is protected by the naturally high alkalinity of the concrete around the reinforcement, typically at a pH greater than 12. At this pH, 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 sufficient alkalinity is maintained. When carbonation occurs, the alkalinity falls and, once it is below a pH of approximately 10, the embedded reinforcing steel is subject to corrosion. Because carbonation occurs from the exposed surface of the concrete inwards, bars close to the exterior surface are subject to the effects of carbonation. Barrier coatings may reduce the rate of additional future carbonation. 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.

The carbonation of concrete that contains chlorides may cause the release of chloride ions from Friedel's salt. Friedel's salt is an anion exchange mineral. It has affinity for anions such as chloride and is capable to retain them to a certain extent in its crystallographical structure. This release of chloride ions may create a chloride front ahead of the carbonation front. Also, background chlorides reduce the passivation of steel such that concrete containing chloride, even below the chloride threshold, requires a higher pH for passivation of steel. Carbonated structures that are also contaminated with chlorides may therefore experience more severe corrosion.

6.2.5 Anodic ring (halo effect)—The anodic ring (halo effect) refers to corrosion of reinforcing steel, and subsequent concrete deterioration, in the parent concrete around the perimeter of a concrete repair. This type of deterioration is the result of corrosion of existing reinforcement extending from the parent concrete into repair concrete caused by chemical differences between the parent concrete and the repair concrete. It occurs because steel is present in two distinctly different environments, setting up conditions that can result in differences in electrical potential between the new and the parent concrete. Corrosion and concrete delami-

nation typically occurs in the parent concrete adjacent to the repair.

Reinforcement corrosion, due to the anodic effect, is a difficult problem to solve. Sealers or barrier coatings help slow down the process but they 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, in addition to high labor costs, challenges with field application that can reduce their effectiveness. Problems can include access to coating the bar undersides and dripping of coatings, which later act as bond-breakers on substrate surfaces. Passive cathodic protection techniques such as the use of embedded galvanic anodes around the perimeter of repairs can delay the onset of the anodic ring effect.

6.2.6 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. To a lesser extent, it can also be a concern on buildings exposed to acid rain or other 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).

6.3—Total system concept

When selecting a protection system for a particular structure, factors affecting the performance of that system on the structure should be considered. Depending on the type of structure, some of the possible factors to consider include:

- a) Protective materials;
- b) Interface of protective materials to concrete surface;
- c) Surface and bulk properties of the concrete;
- d) Structural considerations;
- e) As-built construction considerations such as placement of reinforcement;
- f) Exposure of concrete to the elements;
- g) Stability of ground-supporting concrete;
- h) Groundwater pressures;
- i) Chemical exposures;
- j) Exposure to abrasion, erosion, and cavitation;
- k) Exposure to potable water;
- l) Exposure to wastewater or hazardous wastes;
- m) Environmental considerations.

6.3.1 Economic factors—Evaluation of various protection systems should consider their life-cycle costs. The protection system with the lowest initial cost may actually be more expensive when the costs of future repairs are evaluated over the projected life of the structure.

6.3.2 Service record—Proven systems with documented track records and field performance histories generally provide expected results. New systems in the market typically have limited track records and the field performance may be more difficult to predict.

6.3.3 Appearance—Protective systems may alter the appearance of a structure and mockups can be used to assess the appearance. Appearance should be considered when selecting an appropriate repair and protection system.

6.3.4 Construction quality monitoring—A quality control program should be established and implemented during the installation of the protection system.

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

6.3.5.1 Protective systems that are in contact with potable water should meet the testing requirements of National Science Foundation (NSF)/ANSI 61, as appropriate for the size limits of the repair.

6.3.6 Compatibility and bond—The surface conditions should be reviewed when selecting a protective system. Factors to consider include the compatibility of the repair and substrate materials with the materials of the protective system as well as the ability of the protection material to bond to existing coatings or repair materials. The bond of any remaining existing coatings should be evaluated in accordance with ICRI 210.3 or ASTM C1583. Surface preparation requirements as well as the need to remove existing coatings or sealers may influence the selection of a new system. Job-site testing, laboratory testing, or both, should be performed before installing a new protection system over existing coatings or sealers (ICRI 310.2R).

6.3.7 Durability and performance—The anticipated performance and service life of a protection system, coupled with the owner's requirements, should be considered before selecting a system. Among the factors to consider are substrate movement, thermal expansion and contraction, freezing and thawing, exposure to wind-driven rain, carbon dioxide exposure, ultraviolet exposure, temperature variations, moisture variations, acid rain, abrasion, chemical exposure, and any other potentially destructive conditions.

6.3.8 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 protection materials are toxic and could cause 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 but who are in direct proximity, spaces with open flames, potential for sparks from nearby electrical service, and other hazards are issues that should be considered. The contractor should follow proper safety procedures at all times. The manufacturers should be part of the process when decisions are made regarding installation and the safe use of their products. Material safety data sheets (MSDS) should be furnished with all supplied materials and made available to the owner.

6.3.9 Resistance to chemical attack—Review chemical resistance, testing, and performance records for protective systems that are required to resist exposure to corrosive chemicals or gases to verify that they can perform

adequately. References include [ACI 350.1](#); [ACI 515.2R](#); Portland Cement Association (PCA) IS001 ([PCA 1997](#)); Society for Protective Coating (SSPC) [PA 7](#) and [SSPC-Paint 38](#); [SSPC TR 5](#), [TU 2](#), and [TU 10](#); as well as specific manufacturer's recommendations.

6.4—Surface treatments

Surface treatments are intended to provide protection to the concrete. They include materials applied to the surface of concrete, reinforcing steel, or both. Although the materials and procedures discussed in this section are important components of a protection system, they are not the only components that should be evaluated to achieve an adequate protection system for a repair program. Most of the protection systems for repaired concrete require at least one, or in some cases several, of the surface treatment classifications discussed in [6.5.2](#) and [6.6](#) to [6.9](#).

6.4.1 Uses—Surface treatments include horizontal and 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 by reducing or preventing further moisture, chemical, and chloride intrusion.

6.4.1.1 Precautions—Compatibility among the intended treatment, repair materials, and the existing concrete substrate, with moisture, chemical and chloride intrusion, should be considered. Compatibility of materials in contact with each other should be evaluated to avoid bond or performance failure of the surface treatment materials or the underlying substrate.

Surface treatments often rely on bond to perform adequately. Encapsulating concrete between non-breathing surface treatments should be avoided. Manufacturer's recommendations and limitations should be followed.

In addition, other factors that could influence the effectiveness of the surface treatment 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 protection system.

6.4.1.2 Properties—The properties of surface treatments affecting their selection are discussed in the following sections and are summarized in Table 6.4.1.2a. 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 6.4.1.2b gives references to standards for sealers and coatings.

6.4.1.3 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, concrete repairs should be completed and allowed to cure. Most manufacturers require a minimum curing period of 28 days for cast-in-place concrete. Product manufacturers should be consulted for the required curing time and application conditions for specific products.

Details of terminations at expansion and contraction joints, door and window openings, drains, and curbs should be reviewed and installed properly. It is imperative to ventilate any flammable or noxious fumes that may be produced. Specifications should include installation requirements.

6.4.1.4 Performance characteristics—Surface treatment systems, and the materials used in those systems, have their 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, determine the required system characteristics for the project. The system should then be selected based on its ability to satisfy project requirements. The following are many of the primary system characteristics that should be reviewed when selecting a protection system. Table 6.4.1.2b lists most of the tests commonly used to evaluate performance characteristics.

a) *Water permeability*—Resistance to water absorption is a critical factor in protective systems, as water can facilitate the penetration of chemical or salt solutions causing corrosion of the reinforcing steel. Water permeability is a measurement of the quantity of water that can pass through a surface treatment over a period of time and can be measured using RILEM Test No. II.4.

b) *Moisture vapor transmission*—Surface treatments should resist water intrusion, but they should also allow concrete to dry, particularly if placed over new concrete repairs or on the inside face of foundation walls or slabs-on-ground. Moisture vapor transmission is the quantity of water vapor that can pass through a protective system over a period of time. References include [ASTM E96](#) and [ACI 302.2R](#).

c) *Crack bridging*—The protection system should include treatment of cracks that could provide paths for moisture or other liquids into the concrete substrate. Surface treatment products included in [6.5.2](#) have varying crack bridging abilities. Some products have the ability to fill small dormant cracks but are not suitable to bridge active cracks. Certain products are formulated to bridge small, active 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.

d) *Skid and slip resistance*—Skid and slip resistance are measures of the frictional characteristics of the surface. Penetrants generally do not affect the skid resistance of concrete because they are located at or beneath the surface where they are subject to wear to the same extent as the

Table 6.4.1.2a—Summary of surface treatments

Types	Generic classification(s)	Installation requirements	Durability characteristics	Performance characteristics
Penetrating sealers Surface sealers	Alkyl-alkoxy-silane/siloxanes Epoxy/urethane/methacrylate	Surface free of pretreatments; Sprayed, brushed, or rolled; Ventilation required. Clean, dry, and sound surface; Sprayed, brushed, rolled, or squeegeed.	Improves resistance to freezing and thawing; Reduces salt penetration; Reduces rate of corrosion; Reapplication required. Variable UV radiation resistance; Prevents moisture from penetrating cracks; Reapplication required.	Improved resistance to water absorption and reinforcement corrosion; Does not bridge cracks; Breathable. Seals cracks; Improved resistance to water absorption and reinforcement corrosion; Does not bridge active cracks; Water vapor barrier.
Corrosion inhibitors	Various types	Clean, dry surface free of pretreatments; Ventilation may be required; Generally above freezing.	Reduces rate of corrosion if sufficient material penetrates the concrete to the reinforcing steel.	Reduces rate of corrosion if sufficient material penetrates the concrete to the reinforcing steel.
Coatings (high-build and elastomeric)	Epoxy Urethane Rubberized asphaltic	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. Flexible materials bridge small cracks; Water vapor barrier.
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.

concrete surface. Surface sealers and high build coatings may make the concrete surface less skid-resistant, whereas membrane systems with embedded aggregate and overlays have the potential to create more skid-resistant surfaces. Refer to Table 6.4.1.2a.

e) *Appearance*—Most surface treatments alter the concrete appearance. Some systems, which can mask the contrast between the original concrete and the repairs, can be advantageous if appearance is an important factor.

f) *Carbonation resistance*—Some surface treatments are effective at resisting the penetration of carbon dioxide and can provide protection against carbonation.

g) *Volatile organic compound characteristics*—Federal and state regulations have set allowable VOC levels that may not allow for use of certain surface treatments in some areas.

6.4.2 Surface treatment classifications—The surface treatment classifications discussed in this section are commonly used for concrete repair and protection against reinforcement corrosion. Because of the number or combination of products and techniques, it is impossible to cover all of them. In addition, new products and methods to protect concrete are continually being developed and introduced to the market. Surface treatments in this section are grouped into the following general classifications: penetrating sealers, surface-applied corrosion inhibitors, surface sealers, high-build coatings, elastomeric coatings, and overlay general.

6.4.2.1 Penetrating sealer

a) *Description*—Penetrating sealers are materials that, after application, are generally contained within the near-surface region of the concrete and include water repellent agents. Depth of penetration varies by the product and with

the properties of the concrete on which the sealer is applied. Depth of penetration is a function of the size of the sealer molecule, the pore structure in the concrete, and the moisture content of the concrete. While deep penetration may be desirable, especially for surfaces subjected to abrasion, it is not the most important criterion for sealer effectiveness.

These sealers are formulated to provide protection as water repellents or as surface hardeners. Penetrating sealers do not have crack-bridging capabilities; however, 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 penetrating sealers.

Such sealers include, but are not limited to, silanes, siloxanes, penetrating epoxies, and methacrylates. Some sealers penetrate but are not effective damp-proofers.

b) *Uses*—The products in this classification have the capability to reduce the intrusion of water, chlorides, and, in some cases, mild chemicals. Many permit the transmission of water vapor.

c) *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 resistance, wear resistance, and abrasion resistance are generally good when compared with coatings or membrane systems

6.4.2.2 Surface-applied corrosion inhibitors—Typically applied corrosion inhibitors in concrete repair were first

Table 6.4.1.2b—Testing of sealers and coatings for reinforced concrete

A. Protective properties	
1. Artificial or accelerated weathering:	ASTM D1653 and ASTM E96
2. Water absorption:	ASTM C67 and ASTM C642
3. Chloride ingress:	AASHTO T 259
B. Durability	
1. Natural weathering:	ASTM D4141
2. Artificial or accelerated weathering:	ASTM D2565
3. Salt spray resistance:	ASTM B117
4. Freezing and thawing/scaling:	ASTM D2247
5. Resistance to alkali:	ASTM D1647
6. Light stability:	ASTM D333
7. Chalking:	ASTM D4214
8. Water swelling:	ASTM D4214, ASTM D2247
9. Resistance to wind-driven rain:	ASTM E514, Federal Specification TT-C-555B, Federal Specification TT-P-1411A, ASTM D412, ASTM D522, ASTM D2370, ASTM D429, ASTM D2197, ASTM D3359, ASTM D4541, ASTM D2794
10. Coefficient of friction:	ASTM D2047
11. Chemical resistance:	ASTM C267
12. Crack bridging:	ASTM C836
C. Physical properties	
1. Viscosity:	ASTM D2196
2. Hardness:	ASTM D1471, ASTM D2134, ASTM D2240, ASTM D3363
3. Specific gravity	
4. Drying time:	ASTM D1640
5. Nonvolatile content:	ASTM D1353
6. Flash point:	ASTM D56, ASTM D93, ASTM D3278

introduced in the early 1990s. To be effective, these materials must migrate through the concrete to the reinforcing steel so that they can mitigate the steel corrosion. The effectiveness and life expectancy of these materials is unknown, and likely vary considerably with different types of inhibitors, the properties of the concrete, and site conditions. Users report a wide variation in their effectiveness, which presents the question as to what parameters relate to successful use for any given formulation. A number of studies of their effectiveness have been completed with variable results (Berke et al. 1992; Jones et al. 1999; Elsner 2000; Elsner et al. 1997, 1999; Mietz et al. 1997; Transportation Research Board 2001). Unfortunately, there is no easy or universal method of determining if the corrosion inhibitor has penetrated to the level of the reinforcing steel because each product has its own detection method. Manufacturers should include in their literature the corrosion inhibitor type, recommended uses, application rates, constraints, anticipated effectiveness, penetration detection method, and monitoring method.

6.4.2.3 Surface sealers

a) *Description*—Surface sealers are products that create a film of 10 mils (0.25 mm) or less in thickness that generally adhere to the concrete surface. The dry film thickness of surface sealers ranges from 1 to 10 mils (0.03 to 0.25

mm). Although these products can 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 typically reflect through the sealer. Although surface sealers do not have significant crack-bridging capabilities, some might fill dormant cracks, reducing the penetration of moisture through those cracks.

Such products include epoxies, urethanes, methacrylates, and acrylic resins. Certain oil-based or latex-based paints, 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 10 mils (0.25 mm) thick (PCA 1995).

b) *Uses*—Products in this classification have the capability to reduce the intrusion of water, chlorides, and, in some cases, mild chemicals. They may also inhibit the transmission of water vapor to varying degrees.

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. Many of these products are affected by UV light exposure and will wear due to 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.

6.4.2.4 High-build coatings

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

Base polymers of such products include, but are not limited to, acrylics, alkyds, styrene butadiene copolymers, vinyl esters, chlorinated rubbers, urethanes, methacrylates, 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 light and exposure. On floors, resistance to abrasion, punctures, and mild chemicals such as salts, grease, oil, and detergents are also important.

In addition to the 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 or lead to critical saturation and rapid deterioration of the concrete by cycles of freezing and thawing.

Epoxy resins are commonly used coating 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. Non-elastomeric high-build coatings generally do not bridge active cracks but may be effective in filling small, dormant 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 structure's 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 permeability to water vapor while providing sufficient protection against liquid water intrusion.

6.4.2.5 Elastomeric coatings

a) *Description*—Elastomeric coatings are surface treatments with thicknesses greater than 30 mils (0.75 mm) and less than 250 mils (6 mm) applied to the surface of the concrete. These products alter the appearance and mask blemishes in the concrete surface.

Elastomeric coatings generally have sufficient thickness and flexibility to bridge narrow, inactive cracks of various widths, even if the crack width fluctuates. Some systems require that cracks wider than 10 to 15 mils (0.25 to 0.375 mm) be routed and sealed before applying the membrane. Elastomeric membranes are typically gray or black, but some manufacturers offer other colors. Colored coatings have a tendency to fade, especially when exposed to ultra-violet light. Refer to ACI 362.1R, 362.2R, and 350.1.

Such products include, but are not limited to, urethanes, acrylics, neoprenes, and asphaltic products. Slabs-on-ground should not receive a coating that is also a vapor barrier. An impervious barrier allows moisture passing from the subgrade or backfill to accumulate under or behind the barrier, causing debonding of the coating or leading to critical saturation and rapid deterioration of the concrete by cycles of freezing and thawing.

b) *Uses*—Elastomeric coatings are generally used as protective or waterproofing systems. Elastomeric coatings can also be used as 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 active or inactive cracks less than 0.01 in. (0.25 mm) wide. Membranes with an aggregate-filled urethane or epoxy resin top coats 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 intended for abrasion and skid

resistance are difficult to keep clean and may wear off, or be damaged by mechanical means such as tears caused by snow plows.

Tests that measure performance characteristics of elastomeric membranes, particularly traffic deck membrane systems, are quite limited. Manufacturers may list a variety of different non-standardized test results in their technical data sheets. When available, 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.

6.4.2.6 Overlays general

a) *Description*—Overlays are products 250 mils (6 mm) or greater in thickness that can be bonded or unbonded to the surface of the concrete. Such products include but are not limited to polymer concrete, portland-cement concrete, epoxies, polyesters, polymer-modified concrete, and other hydraulic-cement concrete overlays. 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 inactive cracks; however, active cracks may mirror through the overlay.

b) *Uses*—Overlays may be used as a wearing course and generally provide protection against the intrusion of water and chloride ions. Overlays may also be used to strengthen or restore structural capacity. Overlays also provide abrasion resistance and may be decorative (ACI 222R; 345R; 548.4; 548.5R).

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

For an overlay to perform properly, the surface to which it is bonded should be clean, sound, and roughened. Laitance, dust, films formed from hydrodemolition concrete removal, and debris should be removed. Concrete placement against construction joints on the concrete substrate should involve proper surface preparation as per manufacturer's recommendations.

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 where there is active cracking or structural movement. Unbonded overlays should be used in these situations instead.

Slabs-on-ground in freezing climates should not receive an overlay that also acts as a vapor barrier. An impervious barrier allows moisture passing from the subgrade or backfill to accumulate under or behind the barrier, debonding the overlay or leading to critical saturation and rapid deterioration of the concrete by cycles of freezing and thawing.

Thin cementitious overlays may be susceptible to plastic-shrinkage cracking because of their high surface-to-volume ratio, which promotes rapid evaporation under accelerated drying conditions such as low relative humidity and wind. Also, because these concretes typically have low w/cm , 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 accelerated drying conditions.

Cracking in bonded overlays may be caused by tearing the surface due to late finishing operations; plastic shrinkage due to rapid drying; differential movement between the deck and the overlay due to temperature differences; or drying shrinkage or existing cracks propagating 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](#), [ICRI 210.3](#), and [ASTM C1583](#), should be specified.

6.4.2.6.1 Common overlay systems

a) *Unbonded*—Unbonded 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, asphaltic concrete, or concrete are generally used for this purpose ([ACI 546.1R](#)).

Asphalt and asphaltic concrete overlays are porous and do not provide waterproofing by themselves. The absence of an effective waterproofing system below an unbonded asphaltic or concrete system can promote deck deterioration due to freezing and thawing, as well as lead to corrosion-related damage due to penetration of water and chlorides into the concrete.

These systems are typically used on bridge decks, plaza decks, parking garages, and over-occupied spaces.

b) *Bonded portland-cement concrete (PCC) overlay*—Bonded PCC overlays are typically relatively thin layers of 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 concrete overlays are sometimes fortified with fibers and other additives ([4.2.1](#) and [4.2.5](#)). Low-slump concrete overlays are also included in this group ([4.2.7](#)). Bonded PCC 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 1.5 in. (40 mm) to almost any reasonable thickness, depending on the purpose it is intended to serve.

A PCC 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, fire, and the repair of deteriorated pavements.

Low-slump concrete overlays are PCC overlays that have modified mixture proportions to produce a denser, more durable concrete. They have good bond characteristics to a properly prepared substrate and increased durability because of a lower w/cm . Portland-cement concrete overlays are less expensive than modified concrete systems. The performance of low-slump concrete overlays depends on workmanship and effective use of conventional materials. Generally, the performance of these systems has been good. Local bond failures have been reported, but these have typically been attributed to inadequate surface preparation or other construction deficiencies ([ACI 222R](#); [546.1R](#); [PCA 1996](#)). Low-slump concrete overlays and other PCC 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.

c) *Latex-modified concrete (LMC) overlays*—Latex-modified concrete is similar to conventional concrete except that it contains a polymer dispersion (latex) and less water. The rate of 15 percent 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/cm , good durability, good bonding characteristics, and a high degree of resistance to penetration by chloride ions ([4.2.5](#)).

There have been problems associated with finishing LMC overlays. Hot weather and wind cause rapid drying, making finishing difficult and promoting shrinkage cracking. Mixing, placing, and finishing should be completed within 30 minutes 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 minutes of mixing after the latex has been added. As such, it is generally not practical to mix LMC in concrete trucks because the LMC will likely lose its workability before it can be discharged and placed. Longer periods of mixing entrap air and subsequently increase 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, or texturing, is applied too soon, the grooves will 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 4.3.1](#).

Latex-modified concrete overlays, prepared with a rapid-hardening cement such as calcium sulpho aluminate dicalcium silicate, can be used when it is desirable to place traffic on the overlay with as little as 3 hours of cure ([Sprinkel 2011](#)). Latex-modified concrete 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.

d) *Polymer concrete overlays*—Polymer concrete is a class of composite materials that includes a wide group of mortars and concretes. 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. Whereas more than one may be used for most applications, some are more suitable for specific situations. Refer to **ACI 548.5R** and **Section 4.3.2**.

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 cycles of freezing and thawing 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 within acceptable limits (**ACI 548.1R**; **ACI 548.5R**).

An important advantage of polymer concrete compared to other overlay systems is its rapid cure characteristic. 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 depend on application temperatures (**ACI 548.1R**; **ACI 548.5R**). Another disadvantage is that polymer concrete generally has a different thermal expansion coefficient than PCC.

Polymer-concrete overlays are generally used as a wearing course in damp-proofing or decorative systems. Polymer-concrete overlays have been used as wearing and protective barriers for bridge decks and parking garage decks (**ACI 548.8**; **ACI 548.10**; **Fowler and Whitney 2011**).

e) *High-performance concrete overlays*—High-performance concrete is designed to have improved physical properties with the addition of chemical admixtures, supplemental cementitious materials, and other modifiers such as microfibers or macrofibers, fly ash, silica fume, and blast-furnace slag. Chemical admixtures may include high-range water-reducing admixtures, shrinkage-reducing admixtures, and expansive agents. The improved properties are generally related to improved workability or placeability in the short term or durability and resistance to normal types of deterioration in the long term. They include lower permeability to moisture and chemicals, reduced shrinkage, higher strength, crack resistance, and resistance to freezing and thawing.

Silica-fume concrete is a normal portland-cement concrete containing silica fume as a supplemental cementitious material. Silica-fume concrete is commonly 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 (**4.2.1.3**).

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 autogenous 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. Adding silica fume to the mixture makes it sticky and, thus, more difficult to finish. The use of a high-range water-reducing admixture may be required.

6.5—Concrete surface preparation and installation requirements

Certain basic conditions should be met before application of the protective system. Of these conditions, surface preparation is one of the most important.

6.5.1 Completion of concrete repairs—Concrete repairs should be completed and cured typically 28 days before application of a protective system (**ACI 302.2R**). Some product manufacturers claim that certain products can be installed on moist surfaces and in considerably less time than 28 days.

6.5.2 Surface preparation—The surface should be clean and dry in most cases, but manufacturer’s recommendations should also be followed. A damp surface may be preferable for hydraulic-cement concrete (HCC) overlays, as this may enhance the cure. The substrate should generally be sound, and all surface contamination, including poorly bonded existing coatings, should be removed. The bond of existing coatings, which may have deteriorated due to their age, should be evaluated. Surface preparation should remove all poorly bonded existing coatings, which could result in poor adhesion of a new coating.

Accepted techniques include scarification, brushing or grinding, abrasive blasting, and shotblasting. Dust, debris, and microfractured or bruised concrete resulting from surface preparation should be removed before applying the surface treatment. Refer to **Chapter 3** for more detail on surface preparation. **ICRI 310.2R** provides guidelines for the preparation of concrete substrates that could be used to define a standard of acceptability for some protection systems.

6.5.3 Surface profiles for liquid-applied membranes and other coatings—Surfaces should be relatively smooth, typically in the range of **ICRI 310.2R** concrete surface profile of CSP 3 to CSP 5 (**ICRI 310.2R**; **SSPC-SP13/NACE 6**). Most liquid-applied products are not intended to level off or hide imperfections, although trowel grade materials can sometimes be used for this purpose. The surface profile can compromise the protective qualities of coatings. Surface profiles should be considered when applying coatings to assure that the minimum dry film thickness is maintained. This may entail the application of additional material and could be an issue for exposed membranes or coatings where appearance is a concern. Proper surface preparation should be implemented, including the repair of shallow delaminations, surface scaling, and aggregate popouts; grinding of rough surfaces; or the treatment of any other surface defects as required to achieve proper product performance.

6.5.3.1 Surface quality—For liquid-applied membranes and other coatings, the quality of the concrete surface should be evaluated before installation. Not only should the surface be free of contaminants that could affect the performance of the protection system, it should also have adequate strength, a pH that is compatible with the product to be installed, and as other physical and chemical properties required by the manufacturer for the selected protection system. Refer to **SSPC-SP13/NACE 6** for an overview of the tests commonly used to check the surface quality.

The membrane or coating manufacturer should be consulted to establish criteria for acceptability of the prepared surface.

6.5.4 Ventilation requirements—Ventilation requirements and humidity conditions should be considered when selecting products so that proper application and curing can occur safely and in accordance with requirements. Many surface sealers and penetrants have a high solvent content that, because of evaporation during curing, will require ventilation to remove the vapors from the structure to ensure the safety of construction personnel and other occupants.

6.5.5 Temperature limitations—Most products have installation limitations related to the substrate temperatures, rate of temperature change, product temperature, and mixing and setting times. These limitations should be considered when scheduling work.

6.6—Joints

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

Protection systems for joints include the sealing of cracks, contraction joints, expansion joints, and construction joints.

6.6.1 Types of joints

a) **Cracks**—Cracks occur in concrete because of shrinkage, thermal changes, structural-related stresses, and long-term strain shortening. Before selecting a sealant, the reason for the cracking should be determined. Active or moving cracks should be identified. In some instances, structural bonding of a crack may be required to restore structural integrity, whereas for most active cracks, restraint across the crack should be avoided because future movement will cause a rigid repair to crack again or create a new crack nearby.

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

c) **Expansion (isolation) joints**—Expansion joints prevent crushing and distortion including displacement, buckling, and warping, and provide separation between abutting concrete structural units. 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 configuration of the structure or its settlement (**ACI 504R**). Expansion joints are made by providing a gap between abutting structural units. This gap is often bridged by some sort of mechanical or elastomeric joint or other device to allow movement.

d) **Construction joints**—Construction joints are made before and after interruptions in concrete placement or through positioning of precast units. Locations are typically predetermined to limit the work that can be done at one time to a convenient size with the least impairment to the strength of the finished structure. Construction joints 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 solidly bonded together 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**).

6.6.2 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, as discussed in **4.3** and **6.4.2.5**. **ACI 504R** discusses different techniques and materials to seal joints, and **ACI 503.4** discusses epoxy materials.

6.7—Cathodic protection

6.7.1 Description—Reinforcing steel in concrete is generally protected from corrosion by a passive oxide film formed by contact with alkaline cement. When aggressive ions, such as chlorides, contaminate the concrete around the reinforcing steel, or if dissimilar metals are present, the passive oxide film may be weakened or destroyed, and accelerated 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, an electric current flows from the anodic area to the cathodic area, which results in corrosion at the anodic areas. Unless mitigated, 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 all of the embedded reinforcing steel cathodic, thereby preventing further corrosion of the steel. This can be accomplished by electrically connecting the reinforcing steel to another electrically conductive material 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 called sacrificial, galvanic, or passive systems. The metal that is used to protect the steel is less noble or more prone to corrosion than the steel. Examples are, zinc, magnesium, and aluminum alloys. The sacrificial metal oxidizes in place of the steel and the steel is protected

(NACE 01105). An impressed current cathodic protection (ICCP) system incorporates an external power supply to force a small electric current to flow to the reinforcing steel to counteract the current created by the corrosion process. A highly corrosion-resistant metal such as platinum or titanium typically serves as the anode in most ICCP systems. Refer to ACI 222.3R and NACE 01102 for additional information on this subject.

6.7.2 Applications—Cathodic protection can be used to protect almost any type of reinforced concrete structure.

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

- a) Cathodic protection does not replace corroded steel;
- b) Cathodic protection using impressed-current systems should not be used for general use on prestressed or post-tensioned concrete structures. Hydrogen embrittlement of the high-strength steels may occur;
- c) Post-tensioned structures may be cathodically protected using passive sacrificial systems after an analysis by a corrosion engineer;
- d) 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 installation of a cathodic protection system;
- e) For an ICCP system to function, the reinforcing steel should remain completely isolated from any contact with the anode system. Electrical contact between the anode and the reinforcing steel will create a short circuit and compromise the operation of the system (SHRP-S-337).

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

6.7.4 Types of sacrificial passive cathodic systems

- a) *Zinc hydrogel anode*—Zinc sheet anodes, precoated with a conductive hydrogel adhesive, are applied to the concrete surface. Various coatings or architectural layers 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. Enclosure during spraying may be desired to address toxicity and environmental concerns. This system has been used in both marine and nonmarine environments. A chemical activator or humectant may be applied to the metallized surface to increase current output and corrosion protection for structures in nonmarine environments.
- c) *Zinc anode jackets and overlays*—Zinc mesh may be installed in concrete jackets in marine tidal zones. Distributed zinc anodes may be installed in concrete jackets, overlays, and in holes drilled into the existing structure. Anodes may be chemically activated to increase current output and corrosion protection for structures in nonmarine environments.

- d) *Embedded galvanic anodes*—Embedded galvanic anodes generally 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 and along joints to protect adjacent areas from corrosion due to the anodic-ring effect. The anodes can also be installed on a grid pattern or in drilled holes to provide more overall protection (ACI RAP-8).

6.7.5 Types of ICCP systems—All ICCP systems contain an anode, a DC power source, and connecting cables. In addition to 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.

- a) *Mesh-type noble metal anodes*—A mesh of a noble metal anode is fixed to the concrete with multiple pins and then covered with a cementitious material.
- b) *Ribbon-type noble metal anodes*—A ribbon of a noble metal anode. Generally, these anodes are grouted in slots with a suitable material.
- c) *Conductive coatings*—Carbon-filled conductive coating/mastic has been used as a surface-applied anode. Some of these systems have failed prematurely if exposed to wet conditions.
- d) *Conductive ceramic anodes*—Conductive ceramic anodes are installed and grouted into drilled holes.

6.8—Chloride extraction

Chloride extraction can be accomplished by an electrochemical process.

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

The benefits of ECE include reduced corrosion activity, extended service life, and elimination of the need to remove and replace chloride contaminated concrete. Electrochemical chloride extraction can be applied to 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 222.3R and NACE 01101 for additional information.

6.8.2 Electrochemical chloride extraction application—To remove chlorides from a reinforced concrete structure, an anode embedded in an electrolyte media, typically a poultice similar to paper maché, is applied to the concrete surface. 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 the anode 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 around the negatively

charged reinforcing steel. At the end of the treatment period, the anode and the electrolyte medium 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 zone around the steel. Therefore, when the process is terminated and the installation is removed, the reinforcing steel is in a low-chloride, highly-alkaline concrete environment. The result is a strong repassivation of the embedded reinforcing steel. Corrosion of the reinforcing steel should be significantly reduced. Protection of the concrete may be recommended to keep external chlorides from infiltrating the concrete.

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

6.8.3 Limitations—Electrochemical chloride extraction can be used to mitigate corrosion on many types of structures; however, the following should be observed:

- a) Electrochemical chloride extraction does not replace corroded steel;
- b) Electrochemical chloride extraction should not be used on prestressed or post-tensioned concrete elements as hydrogen embrittlement of the high-strength steels may occur;
- c) The reinforcing steel should be electrically continuous for ECE to function;
- d) The effect of the removal may be different in areas close to reinforcing steel than it is in areas further away from the reinforcing steel.

6.9—Realkalization

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

6.9.1 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 222.3R** and **NACE 01104** for additional information on this subject.

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

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

6.9.3 Limitations—Limitations for realkalization are similar to those for ECE.

CHAPTER 7—STRUCTURAL REPAIR AND STRENGTHENING

7.1—General

Repairs that serve to reestablish the capacity of a member are deemed to be structural. This chapter discusses considerations when repairs are intended to be structural to both restore and enhance the capacity of concrete members. Prior to repairing structural members, a structural analysis may be required to determine if the members are overloaded or underdesigned for the service loads and for ultimate loads. The analysis should consider serviceability and strength and should include consideration of the causes of the structural failure or degradation; refer to **ACI 562**. Based on the evaluations and analytical results, the engineer should decide whether repair is intended to restore the original load capacity only or whether repair is intended to strengthen the member beyond its original capacity, possibly due to additional service loads or code upgrades. **ACI 562** provides guidance for structural evaluation and repair design.

Sections 7.2 through 7.7 outline several acceptable alternatives for strengthening structural members without completely replacing them. These sections concentrate on construction methods used for strengthening, including placement of reinforcement within existing concrete sections as well as placement of new reinforcement outside of the existing member. In all cases, the goal is to strengthen the member to resist stresses caused by flexure, shear, torsion, and axial forces so that it meets the required minimum strength.

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

7.2.1 Repair of cracks by epoxy injection—In cases where repair is needed to restore the original capacity without further increasing the member's structural strength capacity, epoxy injection is commonly used, provided the crack is not actively moving or overly contaminated with dirt or mineral deposits (Fig. 7.2.1). Because the epoxy tensile bond to the concrete substrate is stronger than the concrete's tensile strength, subsequent loads applied to the injected concrete section may result in failures at the same load as that of the original uncracked member section. Resin injection by itself is therefore not a strengthening method, but is considered a restoration method that returns the section to its original precracked strength. Refer to 4.3, 5.2, **ACI 503R**, and International Concrete Repair Institute (ICRI) 210.1.

Strengthening of cracked members may be accomplished by installing additional reinforcement across the crack plane in combination with resin injection. Frequently, internal or external reinforcement is installed in combination with epoxy injection to strengthen a member beyond its original capacity.

7.2.2 Advantages and typical uses—Injection can be successfully performed on cracks as narrow as 0.13 mm (0.005 in.) in width using epoxy injection resins conforming to the appropriate grade, type, and class of **ASTM C881**. Narrower cracks, down to approximately 0.05 mm



Fig. 7.2.1—Epoxy injection of crack in wall.

(0.002 in.), can be injected with epoxy or other polymer systems having a viscosity below 200 cps (ACI 503R and ACI 224.1R). Applying negative pressure to the crack with a vacuum pump can increase the penetration of resin into long or narrow cracks.

7.2.3 Disadvantages—Special considerations should be made for bond strength when elevated in-service temperatures are anticipated. Epoxies lose stiffness and strength when exposed to fire or sustained elevated temperatures above approximately 120 to 150°F (50 to 65°C).

Although some epoxies are moisture-insensitive and thus suitable for injection into wet or damp cracks, epoxies may not be appropriate for injection into cracks that have water actively flowing through them. The specifier should obtain independent test results to verify the moisture sensitivity of the specific epoxy proposed for use, depending on the actual installation conditions. Even epoxies labeled water-insensitive may develop milky white bond lines during curing if injected into excessively wet cracks, indicating that the epoxy is being affected by the moisture. Verification that the epoxy is completely water-insensitive should be made by injecting the test epoxy into prewetted cracks, then evaluating a cored sample of the cured epoxy injected into the crack. Some epoxies absorb water, which reduces the epoxy stiffness and increases creep deformations.

7.2.4 Other considerations—Shrinkage cracking that occurs in slabs or walls as a result of restraint at support ends may develop into full-depth cracks. This type of cracking may result from volume change during concrete curing (drying) or from wide thermal fluctuations. The cause of cracking should be considered when selecting an epoxy injection repair and eliminated if possible. The slab or wall shrinkage cracking described above may recur after injecting the cracks unless end restraints are removed to accommodate slab or wall movement.

Concrete temperature at the time of repair should be considered. Variations in thermal conditions can result in a buildup of internal stresses at the epoxy bond line. Injecting the crack while the member is near the midrange of its anticipated service temperature range minimizes the risk

of re-cracking from thermal stress fluctuations. In addition, cured epoxy properties such as creep resistance and modulus of elasticity should be considered.

7.3—External reinforcement (encased and exposed)

7.3.1 Description—External reinforcement may consist of externally bonded fiber-reinforced polymer (FRP) materials, steel bars and strands, steel brackets, steel plates, steel rolled sections or reinforced concrete jackets placed on the outside of an existing concrete member. The new reinforcement is typically encased with concrete, shotcrete, mortar, plaster, waterproofing or fireproofing, or it may be left exposed and protected from corrosion with a coating; or FRP, which is generally left exposed. Refer to ACI 440.2R for more information regarding repair and strengthening using FRP. Other types of reinforcement include deformed steel bars, welded-wire reinforcement, steel rolled sections, or specially fabricated brackets. ICRI 330.1 provides guidance on several strengthening methods, including externally bonded systems, post-tensioning, section enlargement, and supplemental supports.

Where members have been damaged by overload, erosion, abrasion, or chemical attack, the deteriorated or cracked concrete should be removed and new reinforcement installed around and adjacent to the remaining concrete. In these cases, the new reinforcement is encased in concrete, trowel-applied mortar, or with shotcrete to replicate the original member profile or even to enlarge the size of the original member to provide additional protective cover. 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 3. Epoxy, other chemical adhesives, or portland-cement concrete can be used to bond the new reinforcement to the substrate, or the reinforcement may be mechanically fastened to the existing concrete.

7.3.2 Advantages and typical uses—Placement of external reinforcement may be a more appropriate method for repair and strengthening where obstructions limit access of equipment needed to remove existing concrete and to install internal reinforcement. Additionally, if the concrete requires repair and the placement of 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 concrete beams and girders may be provided by bonding FRP sheets or strips to the member. Alternately, deformed reinforcing bars or plates may be bonded to the surface of the member with shotcrete, cast-in-place concrete, epoxy, or polymer concrete. Anchors may be required in either repair method to ensure composite action.

Steel plates may be attached to existing girders using bolts, as illustrated in Fig. 7.3.2a.

Using a structural adhesive requires adequate surface preparation on both the steel and concrete, as well as selecting an adhesive that is suitable to bond the steel to the concrete.

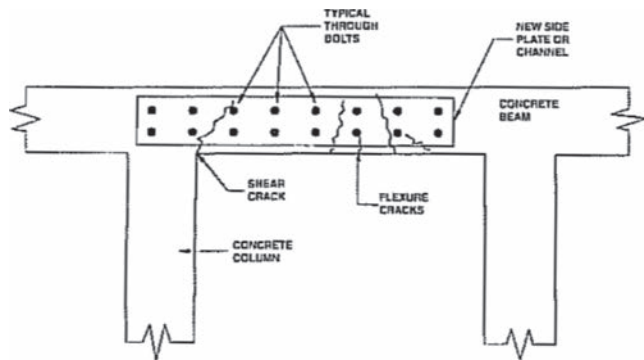


Fig. 7.3.2a—External reinforcement.



Fig. 7.3.2b—Column jacketing.

Abrasive blasting of both the steel plate and concrete surface is an excellent preparation method, but surface cleaning by other mechanical means or water blasting using a minimum of 5000 psi (35 MPa) may be adequate.

Beams, girders, columns, and walls can be strengthened by placing longitudinal reinforcing bars and stirrups or ties around the members, and then encasing the members with shotcrete or cast-in-place concrete (Fig. 7.3.2b). The concrete bonds the new reinforcement to the existing member. Because the added concrete increases the size of the member and adds strength and stiffness, consideration should be given to the additional weight of the member resulting from such additions. Further, the added size of the member increases the member's stiffness, which may alter the force distribution in the structure.



Fig. 7.3.2c—FRP reinforcement.



Fig. 7.3.2d—FRP reinforcement.

Both masonry and reinforced concrete walls have been strengthened by adding layers of externally bonded FRP, or by adding reinforcing bars or welded-wire reinforcement to the face of the wall and using shotcrete to bond the new reinforcement to the wall. Often, new dowels are embedded in holes drilled into the wall to enhance the load transfer between the existing wall and the reinforcement.

Beams, columns, and walls have also been strengthened using FRP materials installed with bonding resin or mortar (Fig. 7.3.2c). These systems should be designed in accordance with the recommendations of **ACI 440.2R** and the manufacturer's recommendations. Banding and strengthening with externally bonded FRP materials can be a cost-effective method used to strengthen structural members requiring seismic upgrade modifications (Fig. 7.3.2d). Refer to ACI 440.2R. Strengthening with externally bonded FRP materials is suitable for use where service loads are increased, provided ductility is considered along with proper provisions for fireproofing and waterproofing.

7.3.3 Limitations—Because the stiffness of most repaired members is increased when plates or concrete encasement are

used, load distribution in the structure is altered and should be analyzed. New 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 brittle material behavior of FRPs, the ultimate behavior of the FRP-strengthened system should be carefully evaluated to ensure satisfactory performance. Refer to [ACI 440.2R](#) for guidance.

Surface preparation is critical if bonding is required for composite action. [Chapter 3](#) 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 and above their glass-transition temperature, which may be as low as 120°F (50°C). Where required, appropriate fire protection should be provided. Proper detailing and protection is especially critical when using FRP materials for strengthening because they are generally adhered to substrates with epoxies. Some adhesives are moisture-sensitive and, when applied to damp or moist surfaces, do not properly bond. Bond tests should be conducted in accordance with [ASTM D7522](#). Pultruded FRP plates should be tested in accordance with [ASTM D3039](#), while FRP impregnated and wet laid-up

FRP should be tested in accordance with [ASTM D7565](#). [ACI 440.8](#) material specifications provide test methods and quality assurance guidelines for FRP.

7.4—External post-tensioning

7.4.1 Description—Girders and slabs may be strengthened in both flexure and shear by the addition of external tendons, rods, or bolts that are tensioned after installation. Figure 7.4.1a illustrates external post-tensioning examples; the external reinforcement may be straight or harped. Figure 7.4.1b illustrates external reinforcement on a precast concrete tee.

7.4.2 Advantages and typical uses—The use of high-strength prestressing steel reinforcement effectively reduces the amount of reinforcement required for repair and strengthening. The prestressing may produce upward camber in members, which helps to reduce service load deflection while increasing both shear and flexural strength. Increased shear resistance results from precompression of the concrete member; and the flexural strength is increased due to the addition of longitudinal reinforcement. The external 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 the equipment required to stress the strands or rods. The post-tensioning force should be determined in accordance with PCI, PTI, and ACI guidelines, including PTI DC80.3-12/ICRI 320.6 ([PTI/ICRI 2012](#)).

7.4.3 Limitations—Placement and stressing of prestressing strands and rods is often difficult in congested buildings and interior locations. If proper positioning of stressing equipment is impossible or impractical, turnbuckles or nuts on rods may be used to tension the steel, depending on the forces required, but the tension in the steel is more difficult to accurately measure using these methods.

Employing prestressing to close or reduce the size of cracks may not be possible, as debris often collects in the open crack. Cleaning and filling the crack with epoxy or other appropriate material may be necessary before tensioning, in order to have more effective load transfer across the plane of the crack.



Fig. 7.4.1a—External post-tensioning of prestressed girder.



Fig. 7.4.1b—External post-tensioning of precast tee member prior to repair with shoring in place (left), and after reinforcement is placed and encased at the center of the tee stem (right).



Fig. 7.5.1—Collar around top of column – reinforcement in place (left) and after concrete has been cast (right).

Providing adequate anchorage of the reinforcement is important; however, this may be difficult to accomplish. Use of steel grillages on the far side of columns and at the end of beams, as shown in Fig. 7.4.1a, is advantageous because such anchorage uses bearing forces and permits stressing of the entire length of a member, including the end connections. Bolted or bonded side anchors are viable alternate anchorages, but the anchors should be designed for the resulting eccentric forces. Concrete members beyond the anchor regions should be checked for strength and to assure that restraint to post-tensioning deformations of the beam section does not cause cracking between the end of the post-tensioned section and the adjoining elements.

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

7.5—Jackets and collars

7.5.1 Description—Jacketing is a process where a section of an existing structural member is increased in size by encasement in concrete. A steel reinforcement cage is constructed around the entire perimeter of the structurally deficient section of a beam or column onto which shotcrete or cast-in-place concrete is placed. Collars are jackets that surround only a portion of the height of a column or pier and typically are used to provide increased support to slabs or beams. A shear collar is shown in Fig. 7.5.1.

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.

7.5.2 Advantages and typical uses—Jacketing is particularly applicable for 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 and steel sections against further deterioration and for strengthening. Permanent forms are advantageous in marine environments where added protection from weath-

ering, abrasion, and chemical pollution is desired. Collars are effective in providing new capitals on existing columns for supporting increased loads on floor slabs or beams. The collar provides increased shear capacity for the slab, and decreases the effective length of the column. Collars may help satisfy architectural constraints better than jacketing the column for its full height.

7.5.3 Limitations—Both jackets and collars require that all deteriorated concrete be removed, structural cracks repaired, existing reinforcement cleaned, and surfaces prepared. This preparation is required so that the newly placed materials properly bond with the existing structure. When jackets are used under water, such preparation is expensive and difficult. The applicability of jackets and collars, however, is widespread and 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 of the repair.

7.5.4 Installation details—The forming techniques used 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. 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.

7.6—Supplemental members

7.6.1 Description—Supplemental members are new columns, beams, braces, or infilled walls installed to permanently support damaged structural members or to support additional loads, as illustrated in Fig. 7.6.1. The supplemental members are typically placed below the failed or deflected areas to stabilize the structural framing.

7.6.2 Advantages and typical uses—Supplemental members may be quickly installed and therefore are suitable temporary emergency repair solutions. Typically,

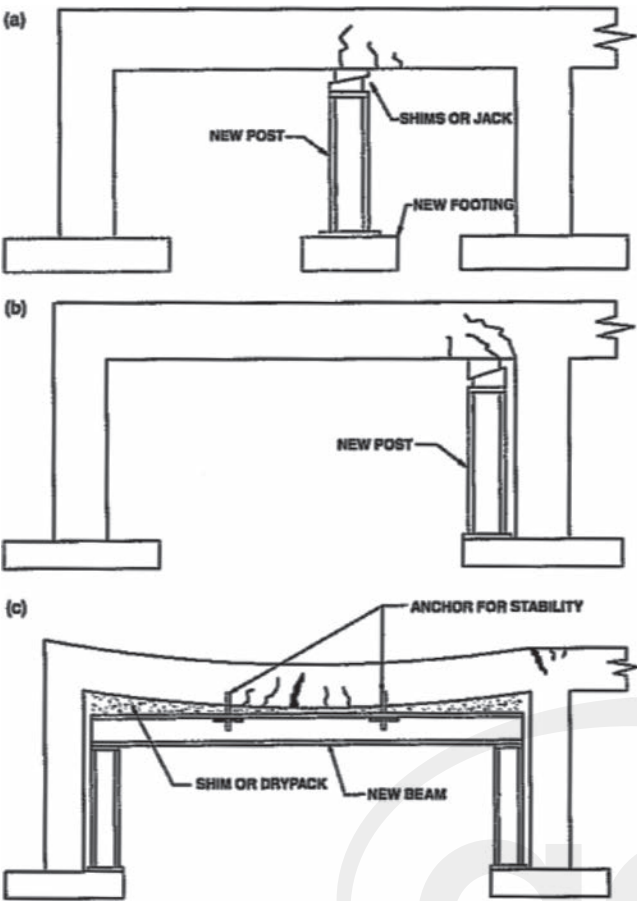


Fig. 7.6.1—Supplemental members.

new members are installed to support seriously damaged, cracked, or deflected flexural members. Often, the use of supplemental members may be the most economical repair alternative.

7.6.3 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 may be distracting. The use of cross bracing, infilled walls, or other means of providing resistance to lateral forces may be needed to augment the structural capacity of the original structure. Such bracing may further restrict interior space utilization. Loads and stresses in the existing structure may not be relieved by the supplemental members unless special procedures such as jacking and/or shoring are used to transfer loads; thus, these aspects should be considered. The supplemental members may cause a redistribution of loads and forces that overstresses nearby existing members, foundations, or both.

7.6.4 Installation details—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 if lifting existing structural members to avoid load-redistribution overstress of beams or adjacent support members.

In Fig. 7.6.1(a), the new post supports a beam to increase its flexural capacity. The new post requires adequate foundation support. As the single-span beam has become continuous with the addition of the post, the negative and positive moment regions should be assessed, considering that adequate flexural reinforcement may not exist in critical negative moment regions.

Figure 7.6.1(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 an existing column are more economical than collars. The post may be positioned eccentrically on the existing footing. If so, the footing should 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 7.6.1(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 with grout. To provide lateral stability to the supplemental member, it may be necessary to mechanically anchor it to the existing slab. If the existing slab is supported on beams, the new beam could be supported on the existing beams instead of adding new posts.

Figures 7.6.4a and 7.6.4b are photographs of supplemental members installed in place.

Special considerations for repair of structural elements

7.7—Repair of concrete columns

Repair procedures for concrete columns should consider the compressive loads on the column(s) to be repaired. Structural columns, by their nature, are designed to carry vertical and sometimes lateral loads and bending moments. Specifically, the self-weight of the structure plus the superimposed dead and live loads supported by the column during the repair procedure should be considered when designing concrete column repairs.

7.7.1 Concrete column repair categories—Repairs to concrete columns typically fall into one of two categories: 1) surface or cosmetic repairs designed to address local deterioration; or 2) structural repairs that enhance or reestablish the load-carrying capacity of the affected column(s).

When concrete columns require repair to address surface deterioration, compressive loads have redistributed to the unaffected portions of the column. In such instances, if the deterioration does not significantly reduce the cross section, conventional concrete repair procedures can successfully address the damage without the need to unload the column via shoring or jacking.

When column deterioration is extensive enough to affect the load-carrying capacity of the column, then unloading the column may be required such that, once the repair is complete, the reintroduced loads are shared uniformly across the entire cross section of the column. If it is not possible to remove the load from the column, then a supplemental column may provide an alternate load path to be used in



Fig. 7.6.4a—Supplemental members.



Fig. 7.6.4b—Supplemental members.

conjunction with the repair of the existing column, assuming that the repair material does not undergo sufficient shrinkage to cause a redistribution of the load or have insufficient bond to the substrate to provide load transfer.

7.7.2 Column repair strategies—Column repair strategies may vary with each condition but include:

- a) Patching the column using repair materials compatible with the original concrete;
- b) Encasement or enlargement of the column cross section;
- c) Confinement using steel plates or FRP materials;
- d) Addition of shear collars to increase the shear capacity of floor slabs and to decrease effective column length;
- e) Addition of a steel plate assembly to increase moment capacity;
- f) Supplemental columns.

In addition to the aforementioned repair strategies, one or more of the following protection strategies may be employed, depending on the condition of the column:

- a) The application of a chemical or barrier protection system;
- b) Cathodic protection;
- c) Realkalization of the concrete adjacent to reinforcing steel; or
- d) Chloride extraction.

7.7.3 Column repair parameters

7.7.3.1 Unloading columns—Unless the column is partially or fully 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 gravity load, particularly if the repair experiences any drying shrinkage. It may be difficult and expensive to unload columns, especially in tall buildings; however, the benefits of unloading may outweigh costs if the structural repairs are required to be fully effective. A properly constructed repair will assist in resisting gravity loads and lateral loads; however, the structural engineer should consider the effect of significant deformation of the column prior to engagement of the repaired section, when shoring is not used to unload prior to repair.

7.7.3.2 Redistribution of load—Redistribution of vertical load has already occurred in the vicinity of the column damage (due to corroded reinforcing steel, concrete cracking, spalling, and/or delamination). The designer should account for this in evaluating the remaining cross section to determine if the redistribution of load has overstressed the column locally. The level of overstress may indicate a need to at least partially relieve the column of load before the repair operation.

7.7.3.3 Supplemental vertical reinforcing steel—Supplemental vertical column reinforcement should ideally be placed within the existing column cage; however, this is difficult to accomplish without cutting the column ties. Cutting existing column ties may result in buckling loaded vertical reinforcement; therefore, it is typically prudent to place additional reinforcing steel bars outside the existing ties, installing additional new ties to provide confinement of the new vertical reinforcing bars.

7.7.3.4 Concrete removal—Removal of concrete within a column cage, inside the vertical reinforcement and ties, especially over a significant portion of the height of the vertical reinforcing bars, is a major concern. When too much concrete is removed from inside the cage, the vertical bars could buckle, even if the column ties are left intact. In addition, the removal of any concrete in a loaded column during the repair process may cause even further load redistribution across the remaining concrete and reinforcing bars. Analysis should define the limit of such concrete removal to prevent a compression failure of the column.

7.7.3.5 Corroded reinforcing steel—Typically, it is unnecessary to remove corroded reinforcing bars where reduced cross-sectional area occurs, even when additional reinforcing bars are installed to replace the lost cross-sectional area. Proper lap lengths for the added reinforcing bar should be provided above and below the corroded portion of the reinforcing bar. If adequate length for lap splicing cannot be accomplished, the existing bars may be welded to a

supplemental bar or to a steel plate or angle to join sections of existing reinforcement to effectively transfer load. Welds should be made according to American Welding Society (AWS D1.4), with due consideration to requirements of preheating and the effects to surrounding concrete from such preheating and welding. Any corroded reinforcement that is left in place should be thoroughly cleaned by sandblasting, to bare metal if possible, to reduce the likelihood of future corrosion.

7.7.3.6 Supplemental reinforcing steel—When the supplemental vertical reinforcing bars are placed outside the existing ties, the column dimensions should be increased to provide adequate cover. Hairpin ties are often added to laterally support the supplemental vertical bars. The hairpin ties may be epoxy bonded into holes drilled into the existing column in accordance with Appendix D of ACI 318.

7.7.3.7 Corroded ties—When column deterioration results from corroded ties, it is important that repairs include appropriate measures to laterally confine the vertical bars. This can be accomplished by adding hairpin ties anchored into the concrete. It is often necessary to build out the column concrete to provide adequate cover over the supplemental ties. Potential buckling problems in the vertical bars may be avoided with this technique.

7.7.3.8 Low-strength concrete—Where low concrete strength results in insufficient load-carrying capacity, several alternatives are available:

- a) Shore the column and remove and replace the low-strength concrete with repair concrete to match the required strength of the column;
- b) Shore the column and increase the size of the column using additional vertical bars, ties, and concrete with strength compatible with the low-strength concrete in the column, thereby increasing the area of the column and increasing its load capacity;
- c) Wrap the column with externally bonded FRP materials to provide confinement such that the strength of the confined material is equivalent to that required;
- d) Install a supplemental column.

7.8—Repair of concrete beams, girders, and joists

Similar to concrete columns, repair procedures for concrete beams, girders, and joists must consider the existing structural loads on the members to be repaired. Concrete beams, girders, and joists are normally designed to carry shear loads and bending moments. The self-weight of the structure plus any superimposed dead and live loads that cannot be removed before repair should be considered when designing repairs for concrete beams, girders, and joists.

7.8.1 Concrete beam, girder, and joist repair categories—Repairs typically fall into one of two categories: 1) surface or cosmetic repairs designed to address local deterioration; or 2) structural repairs that enhance or reestablish the load-carrying capacity of the affected members.

When concrete beams, girders, or joists require repair due to surface deterioration, structural loads may be less critical, given that loads have redistributed to the undisturbed cross section of the member. If the deterioration does not signifi-

cantly reduce the cross section, conventional concrete repair procedures can successfully address the damage.

When deterioration of the member is significant, then unloading the member may be required such that once the repair is complete, the entire cross section of the beam, girder, or joist carries the reintroduced loads. If it is not possible to remove all loads from the member, then the repair concrete and any added reinforcement will only carry loads added after the repair is completed. The area of concrete and original reinforcement, reduced to account for any deterioration, should be checked to confirm that the unrepaired section is capable of carrying all dead and live loads superimposed on the member after the repair is completed. Repair design should also consider the capacity of the unrepaired section to carry required dead and live loads during repair.

7.8.2 Repair strategies—Beam, girder, and joist repair strategies may vary with each condition but include:

- a) Patching the member using repair materials compatible with the original concrete
- b) Adding additional mild steel reinforcement, often accompanied by enlargement of the member cross section;
- c) Supplemental exterior reinforcement using steel plates, or FRP materials;
- d) External prestressing, and
- e) Supplemental columns to reduce the span of the affected member.

In addition to the aforementioned repair strategies, one or more of the following protection strategies may be employed, depending on the condition of the beam, girder, or joist:

- a) The application of a chemical or barrier protection system;
- b) Cathodic protection;
- c) Realkalization of the concrete adjacent to reinforcing steel;
- d) Chloride extraction.

7.8.3 Repair parameters

7.8.3.1 Unloading beams, girders, and joists—Unless the member is unloaded by transferring some or all existing loads by shoring and jacking before performing the repair, it is unlikely that the new repair will carry any load, except those loads that are applied after the repairs are completed, particularly if the repair experiences any drying shrinkage. If the member cannot be unloaded, the repair should be designed considering that the original portion of the member's net cross-sectional area is reduced to account for deterioration of the concrete and reduced areas of reinforcing steel due to corrosion. The analysis should also consider all dead loads and live loads in place at the time of repair, and the portion of superimposed dead and live loads carried by the original, unrepaired member.

7.8.3.2 Preloading simple span prestressed beams, girders, and joists—Repairs made to the precompressed tension zone, bottom-of-the-center section, of simple span prestressed beams, girders, and joists, including double-tees, are likely to crack prematurely if the members are not preloaded before repair. The preload should be calculated such that, with the preload force and dead loads on the member at the time of repair, the precompression load in the

repair area is eliminated. This will allow the repair concrete to be compressed to the same level as the surrounding concrete once the preload is removed. If this is not done, the tension stress under full design load will be significantly higher in the repair concrete than in the original concrete surrounding the repair.

7.8.3.3 Redistribution of the load—Redistribution of shear and bending forces has already occurred in the vicinity of the corroded reinforcing steel and resulting delamination. The designer should be aware of this and carefully evaluate the remaining cross section to determine if the redistribution of load has overstressed the member locally. If so, it may be necessary to at least partially relieve the member of load via shoring and jacking before the repair operation.

7.8.3.4 Concrete removal—The removal of concrete in a compression zone or in areas with high shear forces should be analyzed to establish the limit of concrete removal to avoid potential overstress and sudden failure.

7.8.3.5 Corroded reinforcing steel—Typically, it is unnecessary to remove corroded reinforcing bars with reduced cross-sectional area if additional reinforcing bars are added to replace the lost cross-sectional area. Any corroded reinforcement that is left in place should be thoroughly cleaned by abrasive blasting, to bare metal if possible to reduce the likelihood of future corrosion. Proper lap splice lengths for the added reinforcing bar should be provided on each end of the corroded portion of the reinforcing bar that is supplemented. If adequate length for lap splicing cannot be accomplished, the existing bars may be mechanically coupled or welded to a supplemental bar or to a steel plate or angle to effectively join sections of existing reinforcement to develop their required strength. The welds should be made according to AWS D1.4 with careful consideration to requirements of preheating and the effects to surrounding concrete from such preheating and welding.

7.9—Repair of concrete structural slabs

Considerations for repair of concrete structural slabs are similar to those for beams, girders, and joists, with the additional consideration of punching shear. If repairs are made to a flat slab or flat plate system in the vicinity of a column, punching shear stresses should be considered and shoring of the slab may be required. Like concrete beams, consideration should be given to the existing loads on the slab at the location of the repair; however, unlike beams, significant redistribution of the load is possible and may likely have already occurred. Especially when large repair areas are anticipated in slabs, shoring should be considered not only in the vicinity of the slab repair, but also extending to adjacent bays.

7.9.1 Concrete structural slab repair categories—Repairs typically fall into one of two categories: 1) surface or cosmetic repairs designed to address local deterioration; or 2) structural repairs that enhance or reestablish the load-carrying capacity of the slab.

When concrete slabs require repair due to surface deterioration, structural loads are redistributed to the undamaged portion of the slab. If the deterioration does not significantly

reduce the cross section, conventional concrete repair procedures can successfully address the damage.

When deterioration of the slab is significant, then unloading the slab may be required so that once the repair is complete, the entire slab will carry the reintroduced loads. If it is not possible to remove the load from the slab, then the repair concrete and any added reinforcement will carry only loads added after the repair is completed. The area of concrete and original reinforcement, reduced to account for any deterioration, should be checked to confirm that it is capable of carrying all dead and live loads on the slab at the time of repair, in addition to its proportional share of all dead and live loads superimposed on the slab after the repair is completed.

7.9.2 Repair strategies—Structural slab repair strategies may vary with each condition but include:

- a) Patching the member using repair materials compatible with the original concrete; shotcrete is often used on the underside of slabs for this purpose;
- b) Adding additional mild steel reinforcement, often accompanied by enlargement of the member cross section when installed on the bottom of the slab, and often cut into the slab and bonded with epoxy when installed on the top of the slab;
- c) Supplemental exterior reinforcement using steel plates or FRP materials;
- d) Addition of collars or brackets below the slab to increase the punching shear perimeter and, thus, increase the shear capacity of the slab.

In addition to the above repair strategies, one or more of the following protection strategies may be employed, depending on the condition of the slab:

- a) The application of a chemical or barrier protection system;
- b) Cathodic protection;
- c) Realkalization of the concrete adjacent to reinforcing steel;
- d) Chloride extraction.

7.9.3 Repair parameters

7.9.3.1 Unloading structural slabs—Unless the slab is unloaded by transferring all existing loads to shoring before performing the repair, it is unlikely that the new repair will carry any load, except those loads that are applied after the repairs are completed, particularly if the repair exhibits any drying shrinkage. If the slab cannot be unloaded, the repair must be carefully designed. The portion of the remaining slab should be checked using net cross-sectional area of concrete accounting for deterioration of the concrete and reduced areas of reinforcing steel due to corrosion, and before any supplemental reinforcement or patching concrete is added. The check should account for all dead loads, live loads in place at the time of repair, and its portion of superimposed dead and live loads.

7.9.3.2 Load redistribution—Redistribution of shear and bending forces has already occurred in the vicinity of the corroded reinforcing steel and resulting delamination. The designer should be aware of this and carefully evaluate the remaining cross section to determine if the redistribu-

tion of load has overstressed the member locally. If so, it might be necessary to at least partially relieve the member of load before the repair operation. In addition, the designer should carefully consider the redistribution of load whenever repairs are made to cantilevered slabs, either on the cantilever itself or on the backspan. Shoring is often required when performing repairs on cantilevered slabs.

7.9.3.3 Concrete removal—The removal of concrete in a compression zone or in areas with high shear forces is a major concern. When too much concrete is removed from a compression or high shear zone, the remaining concrete could become overstressed and fail, often suddenly. The designer of the repair should carefully consider the amount of concrete that will be removed, any prestress forces that may exist, and the amount of corroded reinforcement that is lost, when determining the need for shoring.

7.9.3.4 Corroded reinforcing steel—Cutting through existing corroded reinforcement that is under stress will result in redistribution of stress, transferring loads to other parts of the structure. Because of this, it is generally not advisable to remove corroded reinforcing bars with reduced cross-sectional area even if additional reinforcing bars are added to replace the lost cross-sectional area. An exception is when shoring and jacking has been installed so as to unload the member and reinforcement. Proper lap lengths for the added reinforcing bar should be provided on each side of the corroded portion of the reinforcing bar that is supplemented. The supplemental bar(s) should be spliced to the deteriorated portion of the corroded reinforcement to replace the lost cross-sectional area. If adequate length for lap splicing cannot be accomplished, the existing bars may be mechanically spliced or welded to a supplemental bar or to a steel plate or angle to join sections of existing reinforcement. The welds should be made according to AWS D1.4 with careful consideration to requirements of preheating and the effects to surrounding concrete from such preheating and welding. Any corroded reinforcement that is left in place should be thoroughly cleaned by abrasive blasting, to bare metal if possible, to reduce the likelihood of future corrosion.

CHAPTER 8—REFERENCES

AASHTO

M-235-03 (2012)—Standard Specification for Epoxy Resin Adhesives

T 259-02 (2012)—Standard Method of Test for Resistance of Concrete to Chloride Ion Penetration

T 260 -97 (2011)—Standard Method of Test for Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials

American Concrete Institute

201.1R-08—Guide for Conducting a Visual Inspection of Concrete in Service

201.2R-08—Guide to Durable Concrete

207.3R-94(08)—Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions

210R-93(08)—Erosion of Concrete in Hydraulic Structures

210.1R-94—Compendium of Case Histories on Repair of Erosion-Damaged Concrete in Hydraulic Structures (withdrawn)

211.1-91(09)—Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete

212.3R-10—Report on Chemical Admixtures for Concrete

222R-01(10)—Protection of Metals in Concrete against Corrosion

222.3R-11—Guide to Design and Construction Practices to Mitigate Corrosion of Reinforcement in Concrete Structures

223R-10—Guide for the Use of Shrinkage-Compensating Concrete

224.1R-07—Causes, Evaluation, and Repair of Cracks in Concrete Structures

224.3R-95(13)—Joints in Concrete Construction

228.2R-13—Report on Nondestructive Test Methods for Evaluation of Concrete in Structures

232.1R-12—Report on the Use of Raw or Processed Natural Pozzolans in Concrete

232.2R-03—Use of Fly Ash in Concrete

234R-06—Guide for the Use of Silica Fume in Concrete

237R-07—Self-Consolidating Concrete

301-10—Specifications for Structural Concrete

302.2R-06—Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials

304R-00(09)—Guide for Measuring, Mixing, Transporting, and Placing Concrete

304.1R-92—Guide for the Use of Preplaced Aggregate Concrete for Structural and Mass Concrete Applications (withdrawn)

304.2R-96(08)—Placing Concrete by Pumping Methods

304.5R-91(97)—Batching, Mixing, and Job Control of Lightweight Concrete (withdrawn)

304.6R-09—Guide for Use of Volumetric-Measuring and Continuous-Mixing Concrete Equipment

305R-10—Guide to Hot Weather Concreting

306R-10—Guide to Cold Weather Concreting

306.1-90(02)—Standard Specification for Cold Weather Concreting

308.1-11—Specification for Curing Concrete

308R-01(08)—Guide to Curing Concrete

309R-05—Guide for Consolidation of Concrete

311.1R-07—ACI Manual of Concrete Inspection (SP-2) (Synopsis)

311.4R-05—Guide for Concrete Inspection

311.5-04—Guide for Concrete Plant Inspection and Testing of Ready-Mixed Concrete

318-11—Building Code Requirements for Structural Concrete and Commentary

345R-11—Guide for Concrete Highway Bridge Deck Construction

350.1-10—Specification for Tightness Testing of Environmental Engineering Concrete Structures and Commentary

355.1R-91—Report on Anchorage to Concrete (withdrawn)

355.2-07—Qualification of Post-Installed Mechanical Anchors in Concrete and Commentary

362.1R-12—Guide for the Design of Durable Concrete Parking Structures

362.2R-00(13)—Guide for Structural Maintenance of Parking Structures

364.1R-07—Guide for Evaluation of Concrete Structures before Rehabilitation

364.3R-09—Guide for Cementitious Repair Material Data Sheet

423.4R-98—Corrosion and Repair of Unbonded Single Strand Tendons

423.8R-10—Report on Corrosion and Repair of Grouted Multistrand and Bar Tendon Systems

437R-03—Strength Evaluation of Existing Concrete Buildings

439.3R-07—Types of Mechanical Splices for Reinforcing Bars

440R-07—Report on Fiber-Reinforced Polymer (FRP) Reinforcement for Concrete Structures

440.1R-06—Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars

440.2R-08—Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures

440.3R-12—Guide Test Methods for Fiber-Reinforced Polymers (FRP) Composites for Reinforcing or Strengthening Concrete and Masonry Structures

440.8-13—Specification for Carbon and Glass Fiber-Reinforced Polymer (FRP) Materials Made by Wet Layup for External Strengthening of Concrete and Masonry Structures

503R-93—Use of Epoxy Compounds with Concrete (withdrawn)

503.4-92(03)—Specifications for Repairing Concrete with Epoxy Mortars

503.6R-91—Guide for the Application of Epoxy and Latex Adhesives for Bonding Freshly Mixed and Hardened Concretes (withdrawn)

503.7-07—Specification for Crack Repair by Epoxy Injection

504R-90—Guide to Joint Sealants for Concrete Structures (withdrawn)

506R-05—Guide to Shotcrete

506.1R-08—Guide to Fiber-Reinforced Shotcrete

506.3R-91—Guide to Certification of Shotcrete Nozzlemen (withdrawn)

515.1R-85—Guide to the Use of Waterproofing, Damp-proofing, Protective, and Decorative Barrier Systems for Concrete (withdrawn)

515.2R-13—Guide to Selecting Protective Treatments for Concrete

544.1R-96(09)—Report on Fiber Reinforced Concrete

544.3R-08—Guide for Specifying, Proportioning, and Production of Steel Fiber-Reinforced Concrete

544.4R-88(09)—Design Considerations for Steel Fiber Reinforced Concrete

546.1R-80—Guide for Repair of Concrete Bridge Superstructures (withdrawn)

546.2R-10—Guide to Underwater Repair of Concrete

546.3R-14—Guide for the Selection of Materials for the Repair of Concrete

548.1R-09—Guide for the Use of Polymers in Concrete

548.2R-93—Guide for Mixing and Placing Sulfur Concrete in Construction (withdrawn)

548.3R-09—Report on Polymer Modified Concrete

548.4-11—Specification for Latex-Modified Concrete (LMC) Overlays

548.5R-94(98)—Guide for Polymer Concrete Overlays

548.8-07—Specification for Type EM (Epoxy Multi-Layer) Polymer Overlay for Bridge and Parking Garage Decks

548.10-10—Specification for Type MMS (Methyl Methacrylate Slurry) Polymer Overlays for Bridge and Parking Garage Decks

562-13—Code Requirements for Evaluation, Repair, and Rehabilitation of Concrete Buildings and Commentary

RAP-1-03(09)—Structural Crack Repair by Epoxy Injection

RAP-2-03(09)—Crack Repair by Gravity Feed with Resin

RAP-3-03(09)—Spall Repair by Low-Pressure Spraying

RAP-4-03(09)—Surface Repair Using Form-and-Pour Techniques

RAP-5-03(09)—Surface Repair Using Form-and-Pump Techniques

RAP-6-03(09)—Vertical and Overhead Spall Repair by Hand Application

RAP-7-05(10)—Spall Repair of Horizontal Concrete Surfaces

RAP-8-05(10)—Installation of Embedded Galvanic Anodes

RAP-9-05(10)—Spall Repair by the Preplaced Aggregate Method

RAP-12-10—Concrete Repair by Shotcrete Application

RAP-13-10—Methacrylate Flood Coat

RAP-14-10—Concrete Removal Using Hydrodemolition

ASTM International

A185-79—Standard Specification for Welded Steel Wire Fabric for Concrete Reinforcement

A497-79—Standard Specification for Welded Deformed Steel Wire Fabric for Concrete Reinforcement

A615/A615M-07—Standard Specification for Deformed and Plain Carbon Steel Bars for Concrete Reinforcement

A616/A616M-96a—Standard Specification for Rail Steel Deformed and Plain Bars for Concrete Reinforcement (withdrawn 1999)

A617/A617M-96a—Standard Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement (withdrawn 1999)

A706/A706M-01—Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement

A767/A767M-09—Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement

A775/A775M-07b(2014)—Standard Specification for Epoxy-Coated Steel Reinforcing Bars

A780/780M-09—Standard Practice for Repair of Damaged and Uncoated Areas of Hot-Dip Galvanized Coatings

B117-90—Standard Test Method of Salt Spray (Fog) Apparatus

C67-13a—Standard Test Methods of Sampling and Testing Brick and Structural Clay Tile

C94/C94M-14—Standard Specification for Ready-Mixed Concrete

C125-13b—Standard Terminology Relating to Concrete and Concrete Aggregates

C157/C157M-08—Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete

C267-01(2012)—Standard Test Methods for Chemical Resistance of Mortars, Grouts, and Monolithic Surfacing and Polymer Concretes

C293/C293M-10—Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Center-Point Loading)

C387-04—Standard Specification for Packaged, Dry, Combined Materials for Mortar and Concrete

C469/C469M-14—Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression

C494/C494M-13—Standard Specification for Chemical Admixtures for Concrete

C496/C496M-11—Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens

C531-00—Standard Test Method for Linear Shrinkage and Coefficient of Thermal Expansion of Chemical-Resistant Mortars, Grouts, Monolithic Surfacing, and Polymer Concretes

C597-09—Standard Test Method for Pulse Velocity through Concrete

C642-13—Standard Test Method for Density, Absorption, and Voids in Hardened Concrete

C672/C672M-12—Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals

C685/C685M-11—Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing

C806-12—Standard Test Method for Restrained Expansion of Expansive Cement Mortar

C836/C836M-12—Standard Specification for High Solids Content, Cold Liquid-Applied Elastomeric Waterproofing Membrane for Use with Separate Wearing Course

C845/C845M-12—Standard Specification for Expansive Hydraulic Cement

C878/C878M-14—Standard Test Method for Restrained Expansion of Shrinkage-Compensating Concrete

C881/C881M-13—Standard Specification for Epoxy-Resin-Base Bonding Systems for Concrete

C882/C882M-13a—Standard Test Method for Bond Strength of Epoxy-Resin Systems Used with Concrete by Slant Shear

C900-13a—Standard Test Method for Pullout Strength of Hardened Concrete

C920-14—Standard Specification for Elastomeric Joint Sealants

C928/C928M-13—Standard Specification for Packaged, Dry, Rapid-Hardening Cementitious Materials for Concrete Repairs

C1059/C1059M-13—Standard Specification for Latex Agents for Bonding Fresh to Hardened Concrete

C1107/C1107M-13—Standard Specification for Packaged Dry Hydraulic-Cement Grout (Nonshrink)

C1116/C1116M-03—Standard Specification for Fiber-Reinforced Concrete and Shotcrete

C1152/C1152M-04(2012)^{e1}—Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete

C1218/C1218M-99(2008)—Standard Test Method for Water-Soluble Chloride in Mortar and Concrete

C1240-10—Standard Specification for Silica Fume Used in Cementitious Mixtures

C1383-04(2010)—Standard Test Method for Measuring the P-Wave Speed and the Thickness of Concrete Plates Using the Impact-Echo Method

C1438-13—Standard Specification for Latex and Powder Polymer Modifiers for use in Hydraulic Cement Concrete and Mortar

C1439-13—Standard Test Methods for Evaluating Latex and Powder Polymer Modifiers for use in Hydraulic Cement, Concrete, and Mortar

C1582/C1582M-11—Standard Specification for Admixtures to Inhibit Chloride-Induced Corrosion of Reinforcing Steel in Concrete

C1583/C1583M-13—Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off Method)

C1600/C1600M-11—Standard Specification for Rapid Hardening Hydraulic Cement

D56-05(2010)—Standard Test Method for Flash Point by Tag Closed Cup Tester

D93-13^{e1}—Standard Test Methods for Flash Point by Pensky-Martens Closed Cup Tester

D98-05(2013)—Standard Specification for Calcium Chloride

D333-01(2013)—Standard Guide for Clear and Pigmented Lacquers

D412-06a(2013)—Standard Test Methods for Vulcanized Rubber and Thermoplastic Rubbers and Thermoplastic Elastomers – Tension

D429-14—Standard Test Method for Rubber Property—Adhesion to Rigid Substrates

D522/D522M-13—Standard Test Methods for Mandrel Bend Test of Attached Organic Coatings

D638-10—Standard Test Method for Tensile Properties of Plastics

D695-10—Standard Test Method for Compressive Properties of Rigid Plastics

D1353-13—Standard Test Method for Nonvolatile Matter in Volatile Solvents for Use in Paint, Varnish, Lacquer, and Related Products

D1471-69—Method of Test for Two-Parameter 60 Deg Specular Gloss (withdrawn 1975)

D1623-09—Standard Test Method for Tensile and Tensile Adhesion Properties of Rigid Cellular Plastics

D1640-03(2009)—Standard Test Methods for Drying, Curing, or Film Formation of Organic Coatings at Room Temperature

D1647-89(1996)^{e1}—Standard Test Methods for Resistance of Dried Films of Varnishes to Water and Alkali (withdrawn 2004)

D1653-13—Standard Test Methods for Water Vapor Transmission of Organic Coating Films

D2047-11—Standard Test Method for Static Coefficient of Friction of Polish-Coated Floor Surfaces as Measured by the James Machine

D2134-93(2012)—Standard Test Method for Determining the Hardness of Organic Coatings with a Sward-Type Hardness Rocker

D2196-10—Standard Test Methods for Rheological Properties of Non-Newtonian Materials by Rotational (Brookfield type) Viscometer

D2197-13—Standard Test Method for Adhesion of Organic Coatings by Scrape Adhesion

D2240-05(2010)—Standard Test Method for Rubber Property-Durometer Hardness

D2247-11—Standard Practice for Testing Water Resistance of Coatings in 100% Relative Humidity

D2370-98(2010)—Standard Test Method for Tensile Properties of Organic Coatings

D2565-99(2008)—Standard Practice for Xenon-Arc Exposure of Plastics Intended for Outdoor Applications

D2794-93(2010)—Standard Test Method for Resistance of Organic Coatings to the Effects of Rapid Deformation (Impact)

D3039/3039M-14—Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials

D3278-96(2011)—Standard Test Method for Flash Point of Liquids by Small Scale Closed-Cup Apparatus

D3359-09^{e2}—Standard Test Methods for Measuring Adhesion by Tape Test

D3363-05(2011)^{e2}—Standard Test Method for Film Hardness by Pencil Test

D4141/D4141M-14—Standard Practice for Conducting Black Box and Solar Concentrating Exposures of Coatings

D4214-98—Standard Test Methods for Evaluating Degree of Chalking of Exterior Paint Films (withdrawn 2007)

D4258-05(2012)—Standard Practice for Surface Cleaning Concrete for Coating

D4259-88(2012)—Standard Practice for Abrading Concrete

D4260-05(2012)—Standard Practice for Liquid and Gelled Acid Etching Concrete

D4262-05(2012)—Standard Test Method for pH of Chemically Cleaned or Etched Concrete Surfaces

D4417-14—Standard Test Methods for Field Measurement of Surface Profile of Blast Cleaned Steel

D4541-09^{e1}—Standard Test Method for Pull-Off Strength of Coatings Using Portable Adhesion Testers

D4580/4580M-12—Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding

D4788-03(2013)—Standard Test Method for Detecting Delaminations in Bridge Decks Using Infrared Thermography

D6087-08—Standard Test Method for Evaluating Asphalt-Covered Concrete Bridge Decks Using Ground Penetrating Radar

D7522/7522M-09—Standard Test Method for Pull-Off Strength for FRP Bonded to Concrete Substrate

D7565/7565M-10—Standard Test Method for Determining Tensile Properties of Fiber Reinforced Polymer Matrix Composites Used for Strengthening Civil Structures

D7682-12—Standard Test Method for Replication and Measurement of Concrete Surface Profiles Using Replica Putty

E96/E96M-13—Standard Test Methods for Water Vapor Transmission of Materials

E329-14—Standard Specification for Agencies Engages in Construction Inspection, Testing, or Special Inspection

E514/514M-14—Standard Test Method for Water Penetration and Leakage Through Masonry

E965-96(2006)—Standard Test Method for Measuring Pavement Macrotexture Depth Using a Volumetric Technique

E1216-11—Standard Practice for Sampling for Particulate Contamination by Tape Lift

F710-11—Standard Practice for Preparing Concrete Floors to Receive Resilient Flooring

G109-07(2013)—Standard Test Method for Determining Effects of Chemical Admixtures on Corrosion of Embedded Steel Reinforcement in Concrete Exposed to Chloride Environments

American Welding Society (AWS)

D1.4:2005—Structural Welding Code—Reinforcing Steel

Federal Specifications

TT-C-555B-75—Coating, Textured (For Interior and Exterior Masonry Surfaces)

TT-P-1411A-73—Paint, Copolymer-Resins Cementitious (For Waterproofing Concrete and Masonry Walls)

TT-S-001543A-71—Interim Federal Specification, Sealing Compound: Silicone Rubber Base (For Calking, Sealing, and Glazing in Buildings and Other Structures)

TT-S-00227E-69—Interim Federal Specification, Sealing Compound: Elastomeric Type, Multi-Component (For Calking, Sealing, and Glazing in Buildings and Other Structures)

TT-S-00230C-70—Interim Federal Specification, Sealing Compound: Elastomeric Type, Single Component (For Calking, Sealing, and Glazing in Buildings and Other Structures)

International Code Council—Evaluation Service (ICC-ES)

AC125-12—Acceptance Criteria for Concrete and Reinforced and Unreinforced Masonry Strengthening Using Externally Bonded Fiber-Reinforced Polymer (FRP) Composite Systems

International Concrete Repair Institute (ICRI)

120.1-09—Guidelines and Recommendations for Safety in the Concrete Repair Industry

- 130.1R-09—Guide for Methods of Measurement and Contract Types for Concrete Repair Work
- 210.1-98—Guide for Verifying Field Performance of Epoxy Injection of Concrete Cracks
- 210.3-04—Guide for Using In-Situ Tensile Pull-off Tests to Evaluate Bond of Concrete Surface Materials
- 210.4-09—Guide for Nondestructive Evaluation Methods for Condition Assessment, Repair, and Performance Monitoring of Concrete Structures
- 310.1R-08—Guide for Surface Preparation for the Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion
- 310.2R-13—Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, and Polymer Overlays, and Concrete Repair
- 310.3-04—Guide for the Preparation of Concrete Surfaces for Repair Using Hydrodemolition Methods
- 320.2R-09—Guide for Selecting and Specifying Materials for Repair of Concrete Surfaces
- 320.3R-12—Guide for Inorganic Repair Material Data Sheet Protocol
- 330.1-06—Guideline for the Selection of Strengthening Systems for Concrete Structures
- 340.1-06—Guideline for Selecting Grouts to Control Leakage in Concrete Structures

NACE International

- 01101—Electrochemical Chloride Extraction from Steel-Reinforced Concrete—A State-of-the-Art Report
- 01102—State-of-the-Art Report: Criteria for Cathodic Protection of Prestressed Concrete Structures
- 01104—Electrochemical Realkalization of Steel-Reinforced Concrete—A State-of-the-Art Report
- 01105—Sacrificial Cathodic Protection of Reinforced Concrete Elements—A State-of-the-Art Report

National Science Foundation (NSF)

- NSF/ANSI 61-07—Drinking Water System Components—Health Effects

RILEM

- II.4-87—Water Absorption Under Low Pressure (Pipe Method)

The Society for Protective Coating (SSPC)

- PA 7—Applying Thin Film Coatings to Concrete
- SSPC-Paint 38—Single Component Moisture—Cure Weatherable, A
- SSPC-SP13/NACE 6—Surface Preparation of Concrete
- TR 5—Preparation of Protective Coating Specifications for Atmospheric Service
- TU 2—Design, Installation, and Maintenance of Coating Systems for Concrete Used in Secondary Containment
- TU 10—Procedures for Applying Thick Film Coatings and Surfacing Over Concrete Floors

Transportation Research Board (TRB)/Strategic Highway Research Program (SHRP)

- SHRP-S-336—Techniques for Concrete Removal and Bar Cleaning on Bridge Rehabilitation Projects
- SHRP-S-337—Cathodic Protection of Reinforced Concrete Bridge Elements: A State-of-the-Art Report

Authored documents

- Al-Fadhala, M., and Hover, K. C., 2001, "Rapid Evaporation from Freshly Cast Concrete and the Gulf Environment," *Construction & Building Materials*, V. 15, No. 1, Jan., pp. 1-7. doi: 10.1016/S0950-0618(00)00064-7
- Al-Qadi, I. L.; Prowell, B. D.; Weyers, R. E.; Dutta, T.; Gou, H.; and Berke, N., 1992, "Corrosion Inhibitors and Polymers," *Report No. SHRP-S-666*, Strategic Highway Research Program, National Research Council, Washington, DC.
- Berke, N. S.; Dallaire, M. P.; Weyers, R. E.; Henry, M. B.; 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 Conshohocken, PA, 432 pp.
- Best, J., and McDonald, J., 1990, "Evaluation of Polyester Resin, Epoxy and Cement Grouts for Embedding Reinforcing Steel Bars in Hardened Concrete," *Technical Report REMR-CS-23*, USACE, Vicksburg, MS, Apr., 69 pp.
- Elsner, B., 2000, "Corrosion Inhibitors for Steel in Concrete," *International Congress on Advanced Materials*, Keynote Lecture, Materials Week, Munich, Germany.
- Elsner, B.; Buchler, M.; and Bohni, H., 1997, "Corrosion Inhibitors for Steel in Concrete," *Proceedings of EURO-CORR*, Trondheim, Norway.
- Elsner, B.; Buchler, M.; Stadler, F.; and Bohni, H., 1999, "Migrating Corrosion Inhibitor Blend for Reinforced Concrete," *Corrosion*, NACE, V. 55, No. 12.
- Emmons, P. H., 1993, *Concrete Repair and Maintenance Illustrated: Problem Analysis, Repair Strategy, Techniques*, R. S. Means Co., Kingston, MA, 300 pp.
- Federal Highway Administration (FHWA), 2008, "Use and Inspection of Adhesive Anchors in Federal-Aid Projects," Technical Advisory Committee (TA) 5140.30, Oct., 1 p.
- Fowler, D. W., and Whitney, D. W., 2011, "Long Term Performance of Polymer Concrete for Bridge Decks," *NCHRP Synthesis* 423, TRB, Washington, DC.
- Hindo, K. R., 1990, "In-Place Bond Testing and Surface Preparation of Concrete," *Concrete International*, V. 12, No. 4, Apr., pp. 46-48.
- Holland, T. C., 1987, "Working with Silica-Fume Concrete," *Concrete Construction*, V. 32, No. 3, pp., Mar., 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.
- Jones, G.; Lambert, P.; and Wood, I., 1999, "Preventing an Achilles Heel," *Concrete Engineering International*, V. 2, No. 7, Oct., pp. 59-61.
- Kosmatka, S. H., 1994, "Bleeding," *Significance of Tests and Properties of Concrete and Concrete-Making Materials*, ASTM STP 169C, P. Klieger and J. Lamond, eds., ASTM, West Conshohocken, PA, pp. 88-111.

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

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.

Lerch, W., 1957, "Plastic Shrinkage," *ACI Journal Proceedings*, V. 53, Feb, pp. 797-802.

McDonald, J. E., 1987, "Rehabilitation of Navigation Lock Walls: Case Histories," *Technical Report REMR-CS-13*, USACE, Vicksburg, MS, Apr., 288 pp.

McDonald, J. E., 1989, "Evaluation of Vinylester Resin for Anchor Embedment in Concrete," *Technical Report REMR-CS-20*, USACE, Vicksburg, MS, 53 pp.

McDonald, J. E.; Vaysburd, 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.

McDonald, W. E., 1998, "Evaluation of Grouting Materials for Anchor Embedments in Hardened Concrete," *Technical Report REMR-CS-56*, USACE, Vicksburg, MS, Feb., 164 pp.

Mietz, J.; Elsener, B.; and Polder, R., 1997, "Corrosion of Reinforcement in Concrete—Monitoring, Prevention and Rehabilitation," *Publication No. 25*, European Federation of Corrosion (EUROCORR), 64 pp.

National Transportation Safety Board (NTSB), 2007, "Ceiling Collapse in the Interstate 90 Connector Tunnel, Boston, Massachusetts, July 10, 2006," *Accident Report NTSB/HAR-07/02*, Washington, DC, July, 120 pp.

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

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

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

Popovic, P. L., 1998, "Durable Repairs for Corrosion Damaged Concrete," *Proceedings of ICRI Workshop*, Houston, TX.

Project Profile, 1993, "Damaged Bridge Girders Restored," *Concrete Repair Bulletin*, V. 6, No. 4, pp. 23-24.

PTI/ICRI, 2012, "Guide for Evaluation and Repair of Unbonded Post-Tensioned Concrete Structures," PTI DC-80.3-12/ICRI 320.6, Farmington Hills, MI, 54 pp.

Schokker, A. J., 2010, *The Sustainable Concrete Guide – Strategies and Examples*, U.S. Green Concrete Council, Farmington Hills, MI, 89 pp.

Sprinkel, M. M., 1998, "Very-Early-Strength Latex-Modified Concrete Overlay," TAR 99-TAR3, Virginia Transportation Research Council, Charlottesville, VA, 13 pp.

Sprinkel, M. M., 2011, "Rapid Concrete Bridge Deck Overlays," *Frontiers in the Use of Polymers in Concrete*, SP-278, M. Reda Taha, ed., American Concrete Institute, Farmington Hills, MI, pp. 1-10. (CD-ROM) DOI: 10.14359/51682508

Thomas, M. D. A.; Fournier, B.; and Folliard, K. J., 2004, "Protocol for Selecting ASR-Affected Structures for Lithium Treatment," *Report No. FHWA-HRT-04-113*, Federal Highway Transportation Administration (FHWA), Washington, DC, 23 pp.

Thomas, M. D. A.; Fournier, B.; Folliard, K. J.; Ideker, J. H.; and Resendez, Y., 2007, "The Use of Lithium to Prevent or Mitigate Alkali-Silica Reaction in Concrete Pavements and Structures," *Publication No. FHWA-06-133*, Federal Highway Transportation Administration (FHWA), Washington, DC, 50 pp.

Transportation Research Board, 2001, "Evaluation of Penetrating Corrosion Inhibitor System," *Ninth Maintenance Management Conference Proceedings 23*, Transportation Research Board, Washington, DC, pp. 127-135.

Trout, J., 2006, *Epoxy Injection in Construction*, second edition, Hanley-Wood, 85 pp.

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.

U.S. Army Corps of Engineers (USACE), 1989, "Test Method For Determining The Resistance Of Freshly Mixed Concrete To Washing Out In Water," *Handbook for Concrete and Cement*, CRD-C 61-89, USACE, Washington, DC, 3 pp.

U.S. Army Corps of Engineers (USACE), 1993, "Standard Specification for Packaged, Dry, Hydraulic-Cement Grout (Nonshrink)," *Handbook for Concrete and Cement*, CRD C621-93, USACE, Concrete Research Division, Washington, DC, Mar.

U.S. Army Corps of Engineers (USACE), 1995a, "Evaluation and Repair of Concrete Structures," *Engineer Manual*, EM 1110-2-2002, U. S. Army Corps of Engineers, June, Washington, DC.

U.S. Army Corps of Engineers (USACE), 1995b, "Chemical Grouting," *Engineer Manual*, EM 1110-1-3500, USACE, Washington, DC, Jan.

U.S. Army Corps of Engineers (USACE), 2006, "Specification for Antiwashout Admixtures for Concrete," CRD-C 661, USACE, Washington, DC, Mar., 15 pp.

U.S. Army Corps of Engineers (USACE), 2009, "Concrete Rehabilitation for Civil Works," UFGS-03 01 32, USACE, Washington, DC, Nov., 73 pp.

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, Oct., pp. 865-871.

.....



American Concrete Institute
Always advancing

As ACI begins its second century of advancing concrete knowledge, its original chartered purpose remains “to provide a comradeship in finding the best ways to do concrete work of all kinds and in spreading knowledge.” In keeping with this purpose, ACI supports the following activities:

- Technical committees that produce consensus reports, guides, specifications, and codes.
- Spring and fall conventions to facilitate the work of its committees.
- Educational seminars that disseminate reliable information on concrete.
- Certification programs for personnel employed within the concrete industry.
- Student programs such as scholarships, internships, and competitions.
- Sponsoring and co-sponsoring international conferences and symposia.
- Formal coordination with several international concrete related societies.
- Periodicals: the ACI Structural Journal, Materials Journal, and Concrete International.

Benefits of membership include a subscription to Concrete International and to an ACI Journal. ACI members receive discounts of up to 40% on all ACI products and services, including documents, seminars and convention registration fees.

As a member of ACI, you join thousands of practitioners and professionals worldwide who share a commitment to maintain the highest industry standards for concrete technology, construction, and practices. In addition, ACI chapters provide opportunities for interaction of professionals and practitioners at a local level.

American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
Phone: +1.248.848.3700
Fax: +1.248.848.3701

www.concrete.org



American Concrete Institute
Always advancing

38800 Country Club Drive
Farmington Hills, MI 48331 USA
+1.248.848.3700
www.concrete.org

The American Concrete Institute (ACI) is a leading authority and resource worldwide for the development and distribution of consensus-based standards and technical resources, educational programs, and certifications for individuals and organizations involved in concrete design, construction, and materials, who share a commitment to pursuing the best use of concrete.

Individuals interested in the activities of ACI are encouraged to explore the ACI website for membership opportunities, committee activities, and a wide variety of concrete resources. As a volunteer member-driven organization, ACI invites partnerships and welcomes all concrete professionals who wish to be part of a respected, connected, social group that provides an opportunity for professional growth, networking and enjoyment.

Copyright American Concrete Institute
Provided by IHS under license with ACI
No reproduction or networking permitted without license from IHS

