

ACI 562M-13

(metric)

**Code Requirements for Evaluation,
Repair, and Rehabilitation of
Concrete Buildings (ACI 562M-13)
and Commentary**

An ACI Standard

Reported by ACI Committee 562



American Concrete Institute®



First Printing
May 2013

American Concrete Institute®
Advancing concrete knowledge

Code Requirements for Evaluation, Repair, and Rehabilitation of Concrete Buildings and Commentary

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 www.concrete.org/committees/errata.asp. 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
U.S.A.
Phone: 248-848-3700
Fax: 248-848-3701

www.concrete.org

ISBN-13: 978-0-87031-813-9
ISBN: 0-87031-813-6

Code Requirements for Evaluation, Repair, and Rehabilitation of Concrete Buildings (ACI 562M-13) and Commentary

An ACI Standard

Reported by ACI Committee 562

Lawrence F. Kahn, Chair

Keith E. Kesner, Secretary

Tarek Alkhrdaji
James Peter Barlow
F. Michael Bartlett
Randal M. Beard
Casimir Bognacki
Eric L. Edelson
Garth J. Fallis
Paul E. Gaudette
Fred R. Goodwin
Carl J. Larosche
Marjorie M. Lynch

Tracy D. Marcotte
James E. McDonald
Myles A. Murray
Antonio Nanni
Kelly M. Page
Jay H. Paul
Randall W. Poston
Halil Sezen
Constadino Sirakis
Gene R. Stevens

Voting subcommittee members
Greggery G. Cohen
Kevin Conroy
Venkatesh Kumar R. Kodur
Yasser Korany
Andrzej S. Nowak
Predrag L. Popovic
J. Gustavo Tumialan

Consulting member
Peter Emmons

This code provides minimum requirements for the evaluation, repair, rehabilitation, and strengthening of existing concrete buildings and, where applicable, nonbuilding structures.

This code does not provide complete design procedures or construction means and methods. The code comprises both prescriptive and performance requirements. Commentary is provided for both the prescriptive and performance requirements, and is intended to provide guidance to the licensed design professional and referenced sources for additional information on the material presented in the code provisions.

The code and commentary is intended for use by individuals who are competent to evaluate the significance and limitations of its content and recommendations, and who will accept responsibility for application of the material it contains.

The materials, processes, quality control measures, and inspections described in this code should be tested, monitored, or performed as applicable only by individuals holding the appropriate ACI Certifications or equivalent.

CONTENTS

Introduction, p. 3

Chapter 1—General, p. 5

- 1.1—Scope, p. 5
- 1.2—Applicability, p. 6
- 1.3—Jurisdictions without a general existing building code, p. 7
- 1.4—Administration and enforcement, p. 8
- 1.5—Responsibilities, p. 8
- 1.6—Contract documents, p. 9
- 1.7—Maintenance and monitoring, p. 10

Chapter 2—Notation and definitions, p. 11

- 2.1—Notation, p. 11
- 2.2—Definitions, p. 11

ACI 562M-13 supersedes ACI 562-12 (provisional), was adopted February 20, 2013, and was published May 2013.

Copyright © 2013, 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.

Chapter 3—Referenced standards, p. 15

Chapter 4—Basis for compliance, p. 17

- 4.1—General, p. 17
- 4.2—Compliance method, p. 17
- 4.3—Preliminary evaluation, p. 17

Chapter 5—Loads, load combinations, and strength reduction factors, p. 19

- 5.1—General, p. 19
- 5.2—Load factors and load combinations, p. 19
- 5.3—Strength reduction factors for repair design, p. 20
- 5.4—Strength reduction factors for evaluation, p. 20
- 5.5—Load combinations for structures repaired with external reinforcing systems, p. 21

Chapter 6—Evaluation and analysis, p. 22

- 6.1—Requirements for structural evaluation, p. 22
- 6.2—Structural assessment, p. 22
- 6.3—Material properties, p. 23
- 6.4—Test methods to determine or confirm material properties, p. 26
- 6.5—Structural analysis of existing structures, p. 29
- 6.6—Structural serviceability, p. 30
- 6.7—Structural analysis for repair design, p. 30
- 6.8—Strength evaluation by load testing, p. 31

Chapter 7—Design of structural repairs, p. 32

- 7.1—General, p. 32
- 7.2—Strength and serviceability, p. 32

- 7.3—Behavior of repaired systems, p. 32
- 7.4—Bond, p. 34
- 7.5—Materials, p. 35
- 7.6—Design and detailing considerations, p. 36
- 7.7—Repair using supplemental post-tensioning, p. 39
- 7.8—Repair using fiber-reinforced polymer (FRP) composites, p. 40
- 7.9—Performance under fire and elevated temperatures, p. 41

Chapter 8—Durability, p. 44

- 8.1—General, p. 44
- 8.2—Cover, p. 45
- 8.3—Cracks, p. 46
- 8.4—Corrosion and deterioration of reinforcement and metallic embedments, p. 47
- 8.5—Surface treatments and coatings, p. 49

Chapter 9—Construction, p. 50

- 9.1—Stability and temporary shoring requirements, p. 50
- 9.2—Temporary conditions, p. 51
- 9.3—Environmental issues, p. 51

Chapter 10—Quality assurance, p. 52

- 10.1—Inspection, p. 52
- 10.2—Testing of repair materials, p. 53
- 10.3—Construction observations, p. 54

Chapter 11—Commentary references, p. 55

INTRODUCTION

The purpose of this code is to provide minimum material and design requirements for the evaluation, repair, and rehabilitation of structural concrete members.

This page intentionally left blank.

CODE**COMMENTARY****CHAPTER 1—GENERAL****1.1—Scope**

1.1.1 The scope, purpose, applicability, exclusions, interpretation, principles, language, and units of measure are defined in this chapter.

1.1.2 The “existing building code” refers to the code adopted by a jurisdiction that regulates existing buildings.

1.1.3 The “current building code” refers to the general building code adopted by a jurisdiction that regulates new building design and construction. The “original building code” refers to the general building code adopted by a jurisdiction at the time the existing building was constructed.

1.1.4 The “design basis code” is the general building code or the original building code under which the evaluation, repair, and rehabilitation are implemented. If a jurisdiction has adopted a general existing building code, the design basis code shall be determined in accordance with Chapter 4. If a jurisdiction has not adopted a general existing building code, 1.3 applies.

1.1.5 This code is intended to supplement the evaluation requirements of the general existing building code.

1.1.6 This code provides minimum material and design requirements for the repair of damaged, deteriorated, or deficient structural concrete members and systems repaired in accordance with the design basis code. Structural repair includes restoring or increasing one or more of the following: strength, stiffness, ductility, and durability of existing members.

1.1.7 This code supplements the general existing building code and shall govern in all matters pertaining to the evaluation, repair, rehabilitation, and strengthening of concrete members and concrete sections of composite members in existing concrete buildings, except wherever this code is in conflict with the requirements in the general existing

1.1C—Scope

1.1.2C The general existing building code establishes the limit to which a repair and rehabilitation can occur in accordance with the original building code. Above these limits, the repair and rehabilitation is in accordance with the general building code. The general existing building code in the United States is usually based on the International Existing Building Code (IEBC) developed by the International Code Council (ICC). The IEBC is revised every 3 years and was first published in 2003.

1.1.3C The general building code establishes the design requirements for construction materials. The general building code in the United States is usually based on the International Building Code (IBC) published by the ICC. The IBC is revised every 3 years and was first published in 2000. For the design and construction of concrete structures, the IBC and legacy codes reference ACI 318, Building Code Requirements for Structural Concrete and Commentary, with exceptions and additions.

1.1.4C The general existing building code establishes limits to which a repair and rehabilitation can occur in accordance with the original building code. Above these limits, the repair and rehabilitation is in accordance with the general building code.

1.1.5C This code provides evaluation procedures for existing concrete structures. It also provides material and design requirements that allow the licensed design professional to bring existing concrete structures in compliance with building codes written for new construction.

CODE**COMMENTARY**

building code. Wherever this code is in conflict with requirements in other referenced standards, this code shall govern.

1.1.8 Provisions for seismic resistance

1.1.8.1 Evaluation of seismic resistance and rehabilitation design shall be in accordance with the general existing building code.

1.1.8.2 Where rehabilitation for seismic resistance is not required by the general existing building code, voluntary rehabilitation for seismic resistance shall be permitted.

1.1.9 This code is not intended for repair of nonstructural concrete or for aesthetic improvements, except if failure of such repairs would result in an unsafe condition.

1.1.10 Licensed design professional

1.1.10.1 All references in this code to the licensed design professional shall be understood to mean persons who possess the knowledge and skills to interpret and properly use this code and are licensed in the jurisdiction where this code is being used. The licensed design professional for a project is responsible for and in charge of the evaluation or repair design, or both.

1.1.10.2 The licensed design professional is permitted to require evaluation, design, construction, and quality assurance that exceed the minimum requirements of this code.

1.2—Applicability

1.2.1 The requirements of this code are applicable when performing evaluation, repair, rehabilitation, and strengthening of existing concrete buildings and concrete portions of other existing buildings.

1.2.2 This code shall govern the evaluation, repair, rehabilitation, and strengthening of nonbuilding concrete structures when required by the building official.

1.1.8C Conditions for evaluation of seismic resistance and repair are provided in ACI 369R, ASCE/SEI 31, and ASCE/SEI 41. Significant improvements to a building's seismic resistance can be made using repair techniques that provide less than those detailing and reinforcement methods required for new construction. As an example, providing additional reinforcement to confine concrete in flexural hinging regions will increase the energy dissipation and seismic performance even though the amount of confinement reinforcement may not satisfy the confinement requirements for new structures (Kahn 1980; Priestley et al. 1996; Harris and Stevens 1991).

Components of the seismic-force-resisting system that require strength and ductility should be identified. Force-controlled (nonductile) action is acceptable for some classifications of components of the seismic-force-resisting system (ASCE/SEI 41). The strength requirement of 7.1 is applicable to these force-controlled components. ASCE/SEI 41 and ACI 369R provide information on rehabilitation for seismic resistance. Seismic-resisting components requiring energy-dissipating capability should maintain the ability to dissipate energy when repaired. Design and detailing requirements for proper seismic resistance of cast-in-place or precast concrete structures are addressed in ACI 318M and 369R.

1.1.10.1C The licensed design professional is expected to exercise sound engineering knowledge, experience, and judgment when interpreting and applying this code.

1.2C—Applicability

1.2.2C Such structures can include arches, tanks, reservoirs, bins and silos, blast- and impact-resistant structures, and chimneys.

CODE

1.2.3 This code shall govern the evaluation, repair, rehabilitation, and strengthening of building foundation members.

1.2.4 This code governs the evaluation, repair, rehabilitation, and strengthening of soil-supported structural slabs that transmit vertical loads or lateral forces from the structure to the soil.

1.2.5 This code governs the evaluation, repair, rehabilitation, and strengthening of the concrete portions of composite members.

1.2.6 This code governs the evaluation, repair, rehabilitation, and strengthening of precast concrete cladding that transmits lateral loads to diaphragms or bracing members.

1.3—Jurisdictions without a general existing building code

1.3.1 Section 1.3 only applies if a jurisdiction has not adopted a general existing building code.

1.3.2 Sections 4.1 and 4.2 do not apply.

1.3.3 Structures designed by a previous version of ACI 318M shall be evaluated in accordance with Section 4.3 and Chapter 6 of this code to determine if the structure is:

- (a) In compliance with that previous version of ACI 318M;
- (b) Not in compliance with that previous version of 318M but safe;
- (c) Not in compliance with that previous version of 318M and unsafe.

1.3.4 If the structure is determined to be unsafe, modifications shall be in accordance with ACI 318M-11 and this code. If the structure is determined to be safe, modifications shall be in accordance with that previous version of ACI 318M and with this code.

1.3.5 For complete replacement of building members or attachment of new construction to existing building members, the design shall be in accordance with ACI 318M-11 and this code.

COMMENTARY

1.2.3C Foundation elements include spread footings, mat foundations, concrete piles, drilled piers, and caissons embedded in the ground. The design and installation of piling fully embedded in the ground is regulated by the general building code. For repair of such members, the provisions within this code apply if not in conflict with the general building code. For the portions of concrete piling in air or water, or in soil not capable of providing adequate lateral restraint throughout the piling to prevent buckling, the provisions of this code govern where applicable.

1.3C—Jurisdictions without a general existing building code

1.3.4C If the original construction properly implemented the design and detailing requirements of a previous version of ACI 318M, there is no need to modify an existing building to satisfy ACI 318M-11, unless the building is deemed unsafe. The principal goal of structural rehabilitation is to ensure that the structural system is safe. Whereas one or more members may not conform to current structural design requirements, those deficiencies may not result in an unsafe structural system. Evaluation of the entire existing structure is used to determine the safety of the whole.

CODE

1.4—Administration and enforcement

1.4.1 The licensed design professional shall specify that all materials used for repair, rehabilitation, or strengthening shall satisfy governing regulatory requirements at the time the work is implemented.

1.4.2 *Approval of special systems of design or construction*—Sponsors of any repair design or construction system, which does not conform to this code but which has been shown to be adequate by successful use, analysis, or testing, shall have the right to present the data on which their repair is based to the building official or to a board of examiners appointed by the building official. This board shall be composed of competent engineers and shall have authority to investigate the data so submitted, to require tests, and to formulate rules governing repair design and construction of such systems to meet the intent of this code. These rules, when approved by the building official and promulgated, shall be of the same force and effect as the provisions of this code.

1.5—Responsibilities

1.5.1 Licensed design professionals are responsible for the evaluation, design, detailing, and specifying material requirements, establishing criteria for executing the work, preparing documents to perform the work, and specifying a quality assurance program.

1.5.2 The licensed design professional shall notify the owner of any maintenance requirements after the work is completed in accordance with 1.7.

1.5.3 The licensed design professional shall report identified unsafe structural conditions to the owner and to jurisdictional authorities, in accordance with local ordinances.

COMMENTARY

1.4C—Administration and enforcement

1.4.2C New methods of design, new materials, and new uses of materials for repair and rehabilitation should undergo a period of development before being specifically covered in a code. Hence, good systems or components might be excluded from use by implication if means are not available to obtain acceptance. For systems considered under this section, specific tests, load factors, strength reduction factors, deflection limits, and other pertinent requirements should be set by the board of examiners and should be consistent with the intent of this code. Provisions of this section do not apply to model analysis used to supplement calculations or to strength evaluation of existing structures.

1.5C—Responsibilities

1.5.1C At times, contractors employ a licensed design professional to design certain temporary elements such as shoring or bracing. Although the design of temporary elements is generally not performed by the licensed design professional responsible for the repair design for the project, the licensed design professional may provide these services if requested by the contractor and approved by the owner as long as there is no conflict of interest.

The licensed design professional may include a statement in the contract documents that the contractor is responsible for all means and methods to implement the repairs consistent with the contract documents prepared by the licensed design professional.

During the evaluation and repair process, the licensed design professional should request that the owner provide all available information regarding the building's condition, plans, previous engineering reports, and disclose the presence of any known hazardous materials in the repair area and any other pertinent information to the parties involved in the work. This information may require that remedial measures be taken before or during the construction process, which should be considered when determining the scope of work.

1.5.2C Good practice is for the licensed design professional to discuss with the owner that development of maintenance requirements should be within the scope of the licensed design professional's services.

1.5.3C Unsafe structural conditions may be observed at the start of an evaluation process or, if hidden, become evident during the repairs. The licensed design professional performing the evaluation is responsible for informing the owner about the presence of unsafe structural conditions found. When required, these conditions, with recommenda-

CODE

1.6—Contract documents

1.6.1 The contract documents shall convey necessary information to perform the work.

1.6.2 Calculations pertinent to design shall be filed with the contract documents when required by the building official. Analyses and designs using computer programs shall be permitted provided design assumptions, user input, and computer-generated output are submitted. Model analysis shall be permitted to supplement calculations.

1.6.3 The licensed design professional shall provide the owner with copies of evaluation reports, project documents, field reports, and other project documents produced by the licensed design professional in addition to documenting the location of the completed repairs to the extent of the licensed design professional's contractual obligations.

COMMENTARY

tions for remediation, should be reported to the proper jurisdictional authorities by the licensed design professional or owner. The owner should take measures to limit the potential risk associated with exposure to these conditions. The remediation of risk may require immediate evacuation, limiting access to portions of a structure, the installation of shoring, securement or removal of delaminated concrete, or other structural remedies.

1.6C—Contract documents

1.6.1C As necessary, the contract documents should indicate:

- (a) Name and date of issue of the building code and supplements to which the evaluation, repairs, rehabilitation, or strengthening conforms.
- (b) Loads and other demand-related criteria.
- (c) Design assumptions including specified properties of materials used for the project and the strength requirements at stated ages or stages of the construction.
- (d) Details and notes indicating the size, configuration, reinforcement, anchors, repair materials, preparation requirements, and other pertinent information to implement the repairs, strengthening, or rehabilitation of the structure.
- (e) Magnitude and location of prestressing forces.
- (f) Anchorage details for prestressing reinforcement.
- (g) Anchorage length of reinforcement and length of lap splices.
- (h) Type and location of mechanical or welded splices of reinforcement.
- (i) Shoring or bracing necessary before, during, and at completion of the evaluation, repair, rehabilitation, or strengthening projects.
- (j) Quality assurance program including special inspections.

1.6.3C Documentation of the protection and repairs that have been carried out, including any test results, and instructions on inspection and maintenance to be undertaken during the remaining design service life of the repaired part of the concrete structure should be provided to the owner. The extent and type of records should be consistent with the agreement recorded in the contract documents. It is good practice for the owner to keep documentation of repairs, inspection, monitoring, and investigations as a record retention plan developed for future reference.

CODE**1.7—Maintenance and monitoring**

1.7.1 Maintenance recommendations shall be documented by the licensed design professional as part of the repair design.

1.7.2 Repairs, inspections, quality assurance reports, and tests required by the licensed design professional shall be documented and submitted to the owner.

COMMENTARY**1.7C—Maintenance and monitoring**

1.7.1C A maintenance protocol that addresses project-specific conditions should be established as part of the repair design that includes inspections and period of time between inspections, after completion of the repair installation. Maintenance and frequent preventative approaches that occur early in the service life of the structure generally result in improved service life with less interruption and a lower life-cycle cost (Tuutti 1980; ACI 365.1R). A licensed design professional should provide recommendations to the owner on inspection and maintenance to be undertaken during the remaining design service life of the repair material or the repaired part of the structure.

A maintenance protocol, including periodic inspections after completion of the repair that addresses project-specific conditions, provides the most effective method to ensure the durability of the repair design. A maintenance protocol should be provided in the as-built or close-out contract documents. Maintenance of the repair can be incorporated in the instruction manuals from the licensed design professional, contractor, or product manufacturers.

1.7.2C The licensed design professional should provide the owner with copies of reports and other project documents produced by the licensed design professional in addition to documenting the location of completed repairs.

CODE

COMMENTARY

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A_k	= load or load effect resulting from extraordinary event
c	= depth of neutral axis, mm
D	= dead load acting on the structure
d_t	= distance from extreme compression fiber to centroid of extreme tension reinforcement, mm
\bar{f}_c	= average core strength modified to account for the diameter and moisture condition of the core, MPa
f'_c	= specified concrete compressive strength, MPa
f_{ceq}	= equivalent specified concrete strength used for evaluation, MPa
f_y	= specified yield strength of steel reinforcement, MPa
\bar{f}_y	= average yield strength value for steel reinforcement, MPa
f_{yeq}	= equivalent yield strength of steel reinforcement used for evaluation, MPa
k_c	= coefficient of variation modification factor
L	= live load acting on the structure
n	= number of sample tests
S	= snow load acting on the structure
T_g	= glass transition temperature, °C
U	= required strength to resist factored loads or related internal moments and forces
U_{ex}	= resistance of the structural component without strengthening
V	= coefficient of variation determined from testing
ε_t	= net tensile strain in the extreme tension reinforcement at nominal strength
ε_y	= yield strain of steel reinforcement
ϕ	= strength reduction factor
λ	= modification factor related to unit weight of concrete

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, “ACI Concrete Terminology,” <http://terminology.concrete.org>. Definitions provided here complement that resource.

bond—1. adhesion of the applied materials to reinforcement or other surfaces against which it is placed, including friction due to shrinkage and longitudinal shear in the concrete and repair materials engaged by the bar deformations. 2. adhesion or cohesion between layers of a repair area or between a repair material and a substrate produced by adhesive or cohesive properties of the repair material or other supplemental materials throughout the service life of the repair.

compatible—the ability of two or more materials to be placed in contact or in sufficiently close proximity to interact with no significant detrimental results.

composite construction—a type of construction using members produced by combining different materials (for example, concrete and structural steel); members produced

2.2C—Definitions

Additional repair-related definitions are provided by “ICRI Concrete Repair Terminology,” <http://www.icri.org/GENERAL/repairterminology.aspx>.

CODE

COMMENTARY

by combining cast-in-place and precast concrete, or cast-in-place concrete elements constructed in separate placements but so interconnected that the combined components act together as a single member and respond to loads as a unit.

connector steel—steel elements, such as reinforcing bars, shapes, or plates embedded in concrete or connected to embedded elements to facilitate concrete member connectivity. The purpose of connector steel is to transfer load, restrain movement, and provide stability.

dangerous—any concrete building, structure, or portion thereof that meets any of the conditions described below shall be deemed dangerous:

1. The building or structure has collapsed, has partially collapsed, has moved off its foundation, or lacks the necessary support of the ground.
2. There exists a significant risk of collapse, detachment or dislodgement of any portion, member, appurtenance, or ornamentation of the concrete building or structure under service loads.

demand-capacity ratio—ratio of required strength to design strength.

design basis code—legally adopted code requirements under which the evaluations, repairs, and rehabilitations are designed and constructed.

design criteria—codes, standards, loads, displacement limits, materials, connections, details, and protections used in the design of mandated and voluntary work.

durability—ability of a material or structure to resist weathering action, chemical attack, abrasion, and other conditions of service and maintain serviceability over a specified time.

effective area of concrete—cross-sectional area of a concrete member that resists axial, shear, or flexural stresses.

effective area of reinforcement—cross-sectional area of reinforcement assumed to resist axial, shear, or flexural stresses.

equivalent cover—a system to supplement insufficient concrete cover to improve durability or fire protection to that equivalent to the minimum cover specified in the design basis code.

evaluation process—process of determining the in-place condition of a structure. The process may include field and laboratory testing to determine material properties and the extent of any deleterious processes. Engineering calculations may be required to determine the existing structure capacity and demand. The goal of the evaluation process is an assessment of the current or in-place condition of the structure.

existing building—building for which a legal certificate of occupancy has been issued. For buildings that are not covered by a certificate of occupancy, existing buildings are those that are complete and permitted for use.

glass transition temperature—midpoint in transition over which a polymer resin changes from a glassy state to a viscoelastic state as measured pursuant to ASTM D4065. $T_g - 15^\circ\text{C}$ is the glass transition temperature minus 15°F .

Commentary: dangerous—this definition is from the International Existing Building Code (IEBC 2012).

CODE

COMMENTARY

in-place condition—existing condition of a structure, member, connection, and component sizes and geometry, material properties, and damage from aging or other event.

jurisdictional authority—person or entity that has legal control over the applicable building code and permitting procedures for a structure. An example of a jurisdictional authority is the local building official.

nonstructural concrete—any element made of plain or reinforced concrete that is not part of a structural system required to transfer gravity load, lateral load, or both, along a load path to the ground.

objectives of the remedial work—restore, maintain, or enhance the performance level of the existing structure for its intended use and life expectancy, which for the design requires specific design criteria and implementing remedial work.

owner—corporation, association, partnerships, individual, or public body or authority with whom the contractor enters into an agreement and for whom the work is provided. The owner is the party in legal possession of the structure.

renovation—repair and remodeling of a building to restore it to its original serviceable condition.

repair process—1. complete process of evaluating an existing structure; the designing, replacing or correcting deteriorated, damaged, or faulty materials, components, or elements of a structure.

2. procedure of evaluating, designing, and constructing a structure to increase its load-carrying capacity beyond its current capacity or restoring the damaged structure to its original design capacity.

repair reinforcement—reinforcement used to provide additional strength, ductility, confinement, or all, to the repaired member.

retrofit—modification of an existing member or structure to increase its strength, ductility, or both.

service life—estimate of the remaining useful life of a structure based on the current rate of deterioration or distress, assuming continued exposure to given service conditions without repairs.

design service life (of a building, component, or material)—the period of time after installation or repair during which the performance satisfies the specified requirements when routinely maintained but without being subjected to an overload or extreme event.

economic service life—time in service until replacement of the structure (or part of it) is economically more advantageous than keeping it in service.

expected service life (of a building component or material)—the period of time after installation or repair during which the performance satisfies the specified requirements when routinely maintained but without being subjected to an overload or extreme event.

functional service life—time in service until the structure no longer fulfills the functional requirements or becomes obsolete due to change in functional requirements.

technical service life—time in service until a defined unacceptable state is reached, such as spalling of concrete, safety level below acceptable limits, or failure of elements.

CODE

COMMENTARY

serviceability—adequate structural performance under service loads.

stability, global—stability of the overall structure with respect to uplift, overturning, sway instability, or sliding failure.

stability, local—the stability of an individual member or part of an individual member in an existing structure.

strengthening—process of increasing the load-resistance capacity of a structure or a portion thereof.

structural concrete—plain or reinforced concrete in a member that is part of a structural system required to transfer gravity loads, lateral loads, or both, along a load path to the ground.

structural repair—to replace, correct, or strengthen deteriorated, damaged, or understrength load-resisting members and nonstructural members that, if failed, would result in an unsafe condition.

temporary bracing—temporary supplemental members added to an existing structure to prevent local or global instability during evaluation and repair construction.

unsafe—a structure or individual structural member that is dangerous or that constitutes a fire hazard.

upgrade—to improve the quality, effectiveness, or performance of a structure or its structural or nonstructural components (for example, seismic upgrade of a building by increasing the strength or deformation capacity of columns).

Commentary: unsafe—this definition is adapted from the IEBC (2012).

CODE**COMMENTARY****CHAPTER 3—REFERENCED STANDARDS***American Concrete Institute*

ACI 216.1M-07—Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies

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

ACI 437-13—Code Requirements for Load Testing of Existing Concrete Structures and Commentary

ACI 440.6M-08—Specification for Carbon and Glass Fiber-Reinforced Polymer Bar Materials for Concrete Reinforcement

American Institute for Steel Construction

ANSI/AISC 360-10—Specification for Structural Steel Buildings

American Welding Society

D1.4/D1.4M:2011—Structural Welding Code—Reinforcing Steel

ASTM International

ASTM A15—Specification for Billet-Steel Bars for Concrete Reinforcement (withdrawn 1969)

ASTM A16—Specification for Rail-Steel Bars of Concrete Reinforcement (withdrawn 1969)

ASTM A61—Specification for Deformed Rail Steel Bars for Concrete Reinforcement with 60,000 psi Minimum Yield Strength (withdrawn 1969)

ASTM A160—Specification for Axle-Steel Bars for Concrete Reinforcement (withdrawn 1969)

ASTM A185-07/A185M-07—Standard Specification for Steel Welded Wire Reinforcement, Plain, for Concrete

ASTM A370-12—Standard Test Methods and Definitions for Mechanical Testing of Steel Products

ASTM A408—Specification for Special Large Size Deformed Billet-Steel Bars for Concrete Reinforcement (withdrawn 1968)

ASTM A431—Specification for High-Strength Deformed Billet-Steel Bars for Concrete Reinforcement with 75,000 psi Minimum Yield Strength (withdrawn 1968)

ASTM A432—Specification for Deformed Billet Steel Bars for Concrete Reinforcement with 60,000 psi Minimum Yield Point (withdrawn 1968)

ASTM A497/A497M-07—Standard Specification for Steel Welded Wire Reinforcement, Deformed, for Concrete

ASTM A615-09b/A615M-09b—Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement

ASTM A616/A616M-96a—Standard Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement (withdrawn 1999)

ASTM A617/A617M-96a—Standard Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement (withdrawn 1999)

Both current, past and withdrawn standards are referenced. Standards that are referenced in the design basis code are applicable for the evaluation of existing structures. These standards may have been withdrawn by the developing organization; however, they provide information on the materials used at the time of original construction. Refer to 4.3.3 and Chapter 6.

CODE**COMMENTARY**

ASTM A706/A706M-09b—Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement

ASTM A955/A955M-11—Standard Specification for Deformed and Plain Stainless Steel Bars for Concrete Reinforcement

ASTM A1061/A1061M-09—Standard Test Methods for Testing Multi-Wire Steel Strand

ASTM C42/C42M-12—Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete

ASTM C823/C823M-12—Standard Practice for Examination and Sampling of Hardened Concrete in Constructions

ASTM C1583/C1583M-04—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) (withdrawn 2013)

ASTM D4065-06—Standard Practice for Plastics: Dynamic Mechanical Properties: Determination and Report of Procedures

ASTM E329-11c—Standard Specification for Agencies Engaged in Construction Inspection, Testing, or Special Inspection

American Society of Civil Engineers

ASCE/SEI 7-10—Minimum Design Loads for Buildings and Other Structures

ASCE/SEI 31-03—Seismic Evaluation of Existing Buildings

ASCE/SEI 41-06—Seismic Rehabilitation of Existing Buildings

CODE**COMMENTARY****CHAPTER 4—BASIS FOR COMPLIANCE****4.1—General**

The licensed design professional shall determine, at the start of a project, the design basis code for the project's compliance based on criteria set forth in the general existing building code.

4.2—Compliance method

4.2.1 The licensed design professional shall design a repair or rehabilitation in accordance with the prescriptive method, the work area method, or the performance method. The compliance method selected and the design basis code shall be used exclusive of the other methods.

4.3—Preliminary evaluation

4.3.1 The licensed design professional shall review plans, construction data, reports, and other available documents to perform the preliminary evaluation as deemed necessary by the licensed design professional.

4.3.2 For the purpose of performing a preliminary evaluation, the licensed design professional is permitted to either assume a design basis code or analyze the building in accordance with 4.3.3 and 4.3.4.

4.3.3 Existing site conditions shall be visually or otherwise assessed as deemed necessary by the licensed design professional to verify existing structural geometry and conditions. The licensed design professional shall make a determination if substantial structural damage has occurred.

4.3.4 The design strength of the existing structure, with the impact of deficiencies considered, shall be determined considering in-place geometric dimensions and material properties. When material properties are not immediately available, a preliminary evaluation shall be completed using reasonable assumptions for material properties as described in Chapter 6.

4.1C—General

Structures constructed under previously adopted codes or prior to the adoption of a building code may not satisfy all current building code requirements. The International Building Code (IBC) and International Existing Building Code (IEBC) contain specific requirements that determine when existing structures should be upgraded to satisfy the requirements of the general building code. Local ordinances may also require that a structure be upgraded to satisfy the current ACI 318M. These requirements should be reviewed at the start of a project.

4.3C—Preliminary evaluation

4.3.3C Substantial structural damage refers to damage in a structure that results in a significant decrease in either the gravity or lateral load resistance of a structure. The IEBC provides a definition for substantial structural damage. In most building codes, when substantial structural damage has occurred, the structure is required to be repaired sufficiently to satisfy structural design requirements for new construction, except that the seismic design criteria are generally reduced.

4.3.4C Strength calculations should be based on in-place conditions and should include an assessment of the loss of strength due to deterioration mechanisms. Guidelines for assessing in-place conditions include ACI 201.2R, ACI 214.4R, ACI 228.1R, ACI 228.2R, ACI 364.1R, ACI 437.1R, ASCE/SEI 11, ASCE/SEI 31, ASCE/SEI 41, ATC 20, and ATC 45, as well as The Concrete Society Technical Report 68 (2008). Material properties can be assessed by testing as described in these references. When material test results are initially unavailable, historical properties based on typical values used at the time of construction can be used in preliminary evaluation.

The assessment of existing structures should initially focus on critical gravity-load-resisting members such as columns/walls or members that are expected to have limited

CODE

COMMENTARY

ductility, followed by an assessment of the lateral-load-resisting system.

Assessing fire damage and other deterioration mechanisms that result in a change in material properties (such as compressive strength or modulus of elasticity) should include an evaluation of the effect of the damage on the material properties and the impact of the damage on the performance of the existing structure. Examples of deterioration mechanisms that result in possible changes in material properties include corrosion of steel reinforcement, thermal damage, concrete reactions such as alkali-aggregate, and freezing and thawing.

CODE

COMMENTARY

CHAPTER 5—LOADS, LOAD COMBINATIONS,
AND STRENGTH REDUCTION FACTORS

5.1—General

5.1.1 Applicable loads used in: (a) the assessment of the existing structure; and (b) the design of repairs or rehabilitation, shall be determined to verify code compliance.

5.1.2 When ACI 318M-11 is the design basis code for the evaluation or repair design, the load factors, load combinations, and strength reduction factors presented in this chapter shall be used.

5.1.3 It is not permitted to use load factors and load combinations from the original design building code with strength reduction factors from this chapter. It is not permitted to use load factors and load combinations from this chapter with strength reduction factors from the original design building code.

5.1.4 Loads during all phases of the construction and repair process shall be used to design shoring during repairs or other temporary construction. If the building is occupied during the construction period, loading shall be in accordance with ASCE/SEI 7. If the building is unoccupied during the construction period, loads shall be in accordance with ASCE/SEI 37.

5.1.5 When assessing an existing structure or designing repairs or rehabilitation, consideration shall be given to forces including, but not limited to, prestressing, crane loads, vibration, impact, shrinkage, temperature changes, creep, and unequal settlement of supports.

5.1.6 When this code is the design basis code, loads shall be determined in accordance with ASCE/SEI 7, except that seismic loads shall be determined in accordance with ASCE/SEI 41.

5.2—Load factors and load combinations

5.2.1 Design of the repair shall account for existing loads on the structure; the effects of load removal; and the sequencing of load application, including construction and shoring loads, during the repair process.

5.2.2 Structural members and connections, whether being designed for a repair or being structurally evaluated, shall have design strengths at all sections at least equal to the required strengths calculated for factored loads and forces in such combinations as stipulated in this code.

5.1C—General

5.1.2C Load factors, load combinations, and strength reduction factors are intended to achieve acceptable levels of safety against failure based on the accuracy of the strength prediction model and on maximum expected loads during the service life of the structure.

In some instances, a building may need to be upgraded to satisfy current building code requirements in accordance with the provisions of Chapters 1, 4, and 6. Applicable loads are determined in accordance with the building code and standards such as ASCE/SEI 7, ASCE/SEI 37, and ASCE/SEI 41.

5.1.3C Mixing of load factors and load combinations from one code with strength reduction factors from a different code may result in an inconsistent level of reliability and safety.

5.1.5C These factors may not have been specified in the original building code and therefore should be considered in the evaluation and design of repairs.

5.1.6C The provisions of ASCE/SEI 41 will require the licensed design professional to refer to ASCE/SEI 7 for select information.

5.2C—Load factors and load combinations

5.2.2C The basic requirement for strength design or evaluation is expressed as

design strength \geq required strength

$$\phi(R_n) \geq U$$

CODE

COMMENTARY

5.2.3 Required strength U shall be at least equal to the effects of factored load combinations as specified in ASCE/SEI 7 for the applicable gravity, wind, and reduced or unreduced seismic loads from the design basis code.

5.2.4 For post-tensioned anchorage zone design, a load factor of 1.2 shall be applied to the maximum prestressing jacking force.

5.3—Strength reduction factors for repair design

5.3.1 Design strength provided by a member; its connections to other members; and its cross sections, in terms of flexure, axial load, shear, and torsion, shall be taken as the nominal strength calculated in accordance with requirements and assumptions of this code, multiplied by the strength reduction factors ϕ in 5.3.2 and 5.3.4.

5.3.2 The strength reduction factor ϕ shall be as given in the following:

- Tension-controlled sections (steel tensile strain at failure greater than $2.5\varepsilon_y$, where ε_y is the yield strain).....0.90
 - Compression-controlled sections (tensile strain at failure less than or equal to ε_y)
 - (a) Members with spiral reinforcement0.75
 - (b) Other reinforced members0.65
- For sections in which the net tensile strain in the extreme tension steel at nominal strength, ε_t , is between the limits for compression-controlled and tension-controlled sections, linear interpolations of ϕ shall be permitted.
- Shear and torsion0.75
 - Bearing on concrete (except for post-tensioned anchorage zones and strut-and-tie models).....0.65
 - Post-tensioned anchorage zones0.85
 - Strut-and-tie models and struts, ties, nodal zones, and bearing areas in such models.....0.75

5.3.3 Development lengths do not require a ϕ -factor.

5.3.4 For flexure, compression, shear, and bearing of structural plain concrete, ϕ shall be 0.60.

5.4—Strength reduction factors for evaluation

5.4.1 If the required structural element dimensions and location of reinforcement are determined in accordance with

The design strength is the nominal strength multiplied by the strength reduction factor ϕ . The required strength U is expressed in terms of factored loads, which are the product of specified loads multiplied by load factors specified in 5.2.3.

5.2.4C The load factor of 1.2 applied to the maximum tendon jacking force results in a design load that exceeds the typical prestressing yield strength. This compares well with the maximum attainable jacking force, which is limited by the anchor efficiency factor. For jacking loads less than the maximum tendon jacking force, or for jacking loads on nonmetallic prestressing tendons, design of the anchorage for 1.2 times the anticipated jacking force is appropriate, given that the jacking load is controlled better than typical dead loads.

5.3C—Strength reduction factors for repair design

5.3.2C For a steel yield strength of 420 MPa, the steel tensile strains corresponding to the tension- and compression-controlled limits are 0.005 and 0.002, respectively. Because the compressive strain in the concrete at nominal strength is typically assumed to be 0.003, the net tensile strain limits for compression-controlled members may also be stated in terms of the ratio c/d_t , where c is the depth of the neutral axis at nominal strength, and d_t is the distance from the extreme compression fiber to the centroid of extreme tension reinforcement. The c/d_t limits for compression- and tension-controlled sections are 0.6 and 0.375, respectively. The 0.6 limit for compression-controlled sections applies to sections reinforced with Grade 420 steel and to prestressed sections.

5.4C—Strength reduction factors for evaluation

5.4.1C Strength reduction factors given in 5.4.1 are larger than those specified in 5.3.1. These increased values

CODE

Chapter 6, and material properties are determined according to Section 6.3.5, it shall be permitted to increase ϕ from those specified in 5.3, but ϕ shall not be more than:

Tension-controlled section	1.0
Compression-controlled sections:	
Members with spiral reinforcement	0.9
Other reinforced members.....	0.8
Shear and/or torsion.....	0.8
Bearing on concrete.....	0.8

5.4.2 If an evaluation of members with no observed deterioration is based on historical material properties as given in Tables 6.3.1a through 6.3.1c, the ϕ -factors not greater than those given in 5.3 shall apply.

5.5—Load combinations for structures repaired with external reinforcing systems

5.5.1 For repairs achieved with unprotected external reinforcing systems, the required strength U of a structure without repair shall be at least equal to the effects of factored loads in Eq. (5.5.1).

$$U_{ex} \geq 1.2D + 0.5L + A_k + 0.2S \quad (5.5.1)$$

where U_{ex} is the factored resistance of the structure without repair; D , L , and S are the specified dead, live, and snow loads, respectively, calculated for the rehabilitated structure; and A_k is the load or load effect resulting from the extraordinary event. For cases where the design live load acting on the member to be strengthened has a high likelihood of being present for a sustained period of time, a live load factor of 1.0 shall be used in place of 0.5 in Eq. (5.5.1).

COMMENTARY

are justified by the improved reliability due to the use of accurate field-obtained material properties, actual in-place dimensions, and well-understood methods of analysis. They have been deemed appropriate for use in ACI 318M-11, Chapter 20, and have had a lengthy history of satisfactory performance.

5.5C—Load combinations for structures repaired with external reinforcing systems

5.5.1C For repairs achieved with external reinforcing systems, such as externally bonded fiber-reinforced polymer, externally bonded steel plates, or external post-tensioning systems, the unrepaired structure should maintain a minimum strength should failure of the repair system occur due to extraordinary events. Fire, impact, and blast are considered examples of extraordinary events. Wind and earthquake forces are not considered extraordinary events, as they are unlikely to cause damage to the unprotected external reinforcing systems. The minimum limit of Eq. (5.5.1) will allow the structure to maintain sufficient structural strength until the damaged repair system has been repaired. Examples of cases where the live load is present for a sustained period of time include library stack areas, heavy storage areas, warehouses, and other occupancies with a live load exceeding 5 kN/m².

CODE

COMMENTARY

CHAPTER 6—EVALUATION AND ANALYSIS

6.1—Requirements for structural evaluation

6.1.1 A structural evaluation shall comprise a structural assessment, structural analysis, or both. If the strength of a structure is known, improvements to the strength, serviceability, durability, and fire performance of a structure shall be permitted without performing a structural evaluation.

6.1.2 A structural evaluation shall be performed if, during the preliminary evaluation, as described in Section 4.3, it is determined that an existing member, portions of a structure, or entire structure exhibit signs of deterioration, structural deficiency, or behavior that is inconsistent with available design and contract documents or code requirements in effect at the time of construction. If determined by the structural assessment that the strength of a structure is not in question, structural analysis is not required.

6.1.3 A structural evaluation shall be performed when there is a reason to question the design strength of the member or structure and insufficient information is available to determine if a member, portion, or all of the existing structure is capable of supporting existing or new design loads.

If the strength of a structure is not in question, improvements to the strength, serviceability, durability, and fire performance of a structure may be completed without performing a structural evaluation.

6.1.4 Where repairs are required on an element in a structure, it shall be determined if similar elements throughout the structure also require evaluation.

6.2—Structural assessment

6.2.1 At a minimum, a structural assessment shall include affected structural members identified in Section 6.1 for further assessment.

6.2.2 A structural assessment shall document existing conditions as necessary to perform a structural analysis.

6.1C—Requirements for structural evaluation

6.1.1C Guidance on assessments may be obtained in ASCE/SEI 31 and ACI 364.1R.

6.1.3C The data gathered for determining strength should include the effects of material degradation, such as loss of concrete strength from chemical attack; freezing and thawing; and loss of steel area due to corrosion or other causes. The effect of deterioration on the ductility of the section should be considered in the evaluation. The strength or serviceability of a structure may be compromised by spalling, excessive cracking, large deflections, or other forms of degradation.

6.1.4C If there is no evidence of distress or deterioration of similar elements to those that required repair elsewhere in a structure, there is no need to perform an evaluation of those locations unless conditions are such that life safety is an issue. Life safety may be a concern if there are significant variances from the original design intent such as lower-strength concrete or misplaced reinforcement. In addition, if the similar elements are in an environment that could foster deterioration, then repairs or strengthening may be required, as opposed to a similar element in a less severe environment.

6.2C—Structural assessment

6.2.1C Assessment may include visual, destructive, and nondestructive testing (NDT) where evidence of deterioration is apparent. At a minimum, areas of known deterioration and distress in the structural elements should be identified, inspected, and recorded as to the type, location, and degree of severity. Procedures are referenced in ACI 201.1R, ACI 228.1R, ACI 364.1R, and ACI 437. Engineering judgment may be required when performing a structural assessment. The affected structural elements are not only members with obvious signs of distress but also contiguous elements in the structural system.

6.2.2C Existing conditions which may need to be documented include (a) through (g):

CODE

COMMENTARY

6.2.3 If an analysis is required, the structural assessment shall document the requirements of 6.2.2 and (a) through (c).

- (a) As-measured structural member section properties and dimensions.
- (b) The presence and effect of any alterations to the structural system.
- (c) Loads, occupancy, or usage different from the original design.

6.3—Material properties

6.3.1 Material properties shall be obtained from available drawings, specifications, and other documents for existing construction. If such documents do not provide sufficient information to characterize the material properties, this information shall be obtained from the historical data provided in Tables 6.3.1a through 6.3.1c, or determination of material properties in accordance with the requirements of 6.3.5, or both.

6.3.2 As a minimum, concrete compressive strength and steel reinforcement yield strength shall be determined for the existing structure where a structural evaluation is required. When tests are used to determine material properties, test methods shall comply with the requirements set forth in 6.3.5.

(a) The physical condition of the structural members shall be examined and the extent and location of the degradation.

(b) The adequacy of continual load paths through the primary and secondary structural members to provide for life safety and structural integrity.

(c) As-built information required to determine appropriate strength reduction factors in accordance with Chapter 5.

(d) Structural members' orientation, displacements, construction deviations, and physical dimensions.

(e) Properties of materials and components from available drawings, specifications, and other documents for existing construction; or by testing of existing materials.

(f) Additional considerations, such as proximity to adjacent buildings, load-bearing partition walls, and other limitations for rehabilitation.

(g) Necessary information to assess seismic resistance, seismicity, lateral-force-resisting systems, span lengths, support conditions, building use and type, and architectural features.

The construction documents may not represent as-built conditions. Therefore, the licensed design professional is encouraged to research and verify the material properties obtained from record documents are accurate. Material testing may be required to verify these values. Additional information regarding the extent of the evaluation is provided in 6.3.5C.

6.3C—Material properties

6.3.1C Material properties required for seismic evaluation and rehabilitation are discussed in ASCE/SEI 31 and ASCE/SEI 41. Where the as-built conditions and properties of historical buildings require evaluation and rehabilitation, care should be taken to minimize the impact of design and investigation procedures (U.S. Department of the Interior 1995). Material properties include all physical and chemical properties of the concrete and reinforcement specified, and includes all references to ASTM standards and other methods of determining physical and chemical properties.

6.3.2C Additional factors and characteristics affecting materials that may be required to be evaluated include:

- (a) Ductility of the element based on the stress-strain curves of the material.
- (b) Presence of corrosion of embedded steel reinforcement, including carbonation, chloride intrusion, and corrosion-induced spalling.

CODE

COMMENTARY

Table 6.3.1a—Default compressive strength of structural concrete, MPa*

Time frame	Footings	Beams	Slabs	Columns	Walls
1900-1919	7	14	10	10	7
1920-1949	10	14	14	14	14
1950-1969	17	21	21	21	17
1970-present	21	21	21	21	21

*Taken from ASCE/SEI 41-06.

Table 6.3.1b—Default tensile and yield strength properties for steel reinforcing bars for various periods*

		Structural [†]	Intermediate [†]	Hard [†]				
	Grade	230	280	345	420	450	485	520
	Minimum yield, MPa	230	280	345	420	450	485	520
Year	Minimum tensile, MPa	380	485	550	620	520	550	690
1911-1959		X	X	X	—	X	—	—
1959-1966		X	X	X	X	X	X	X
1966-1972		—	X	X	X	X	X	—
1972-1974		—	X	X	X	X	X	—
1974-1987		—	X	X	X	X	X	—
1987-present		—	X	X	X	X	X	—

*An entry of “X” indicates the grade was available in those years.

[†]The terms “structural,” “intermediate,” and “hard” became obsolete in 1968.

CODE

COMMENTARY

Table 6.3.1c—Default tensile and yield strength properties of steel reinforcement for various ASTM specifications and periods*

			Structural [†]	Intermediate [†]	Hard [†]					
			Grade	230	280	345	420	450	485	520
			Minimum yield, MPa	230	280	345	420	450	485	520
ASTM designation [‡]	Steel type	Year range	Minimum tensile, MPa	380	485	550	620	520	550	690
A15	Billet	1911-1966		X	X	X	—	—		
A16	Rail [§]	1913-1966		—	—	X	—	—		
A61	Rail	1963-1966		—	—	—	X	—		
A160	Axle	1936-1964		X	X	X	—	—		
A160	Axle	1965-1966		X	X	X	X	—		
A185	WWF	1936-present		—	—	—	—	X		
A408	Billet	1957-1966		X	X	X	—	—		
A431	Billet	1959-1966		—	—	—	—	—		X
A432	Billet	1959-1966		—	—	—	X	—		—
A497	WWF	1964-present		—	—	—	—	—	X	—
A615	Billet	1968-1972		—	X	—	—	—	—	X
A615	Billet	1974-1986		—	X	—	X	—		—
A615	Billet	1987-present		—	X	—	X	—		X
A616	Rail	1968-present		—	—	—	X	—		—
A617	Axle	1968-present		—	X	—	—	—		—
A706 [#]	Low-alloy	1974-present		—	—	—	X	—	X	—
A955	Stainless	1996-present		—	X	—	X	—		X

*An entry of "X" indicates the grade was available in those years.

[†]The terms structural, intermediate, and hard became obsolete in 1968.

[‡]ASTM steel is marked with the letter W.

[§]Rail bars are marked with the letter R.

^{||}Bars marked with "s!" (ASTM A616) have supplementary requirements for bend tests.

[#]ASTM A706 has a minimum tensile strength of 550 MPa, but not less than 1.25 times the actual yield strength.

(c) Presence of other degradation, such as alkali-silica reaction, sulfate attack, or delayed ettringite formation.

(d) Degradation due to cyclic freezing and thawing.

(e) Internal cracking (especially adjacent to and under previous repairs).

Other tests for material properties, including petrographic examination, are often used. These tests can be highly variable and dependent on the structure, member type(s), and distress mechanism.

Chloride penetration can cause steel reinforcement corrosion, which can lead to cracking and spalling. The depth of a spall will affect the effective area of concrete section. Concrete degradation will affect its compressive strength.

CODE

6.3.3 Nominal material properties shall be determined by (a), (b) or (c):

- (a) Historical material properties provided in Tables 6.3.1a through 6.3.1c.
- (b) Available drawings, specifications, and previous testing documentation.
- (c) Physical testing in accordance with 6.4.

6.3.4 The material properties provided in the original construction test reports or material test reports shall be permitted to be used unless visible deterioration has occurred.

6.3.5 When properties are to be determined by in-place testing, the locations and numbers of material samples shall be defined by the licensed design professional. The number of samples shall not be less than required by the test standard.

6.3.6 If historic data are not given in either Table 6.3.1b or 6.3.1c, the historic default value for yield strength shall be taken as 185 MPa.

6.4—Test methods to determine or confirm material properties

6.4.1 *General*—Destructive and nondestructive test methods used to obtain in-place mechanical properties of materials and member properties shall comply with the requirements of this section. It shall be permitted to determine the compressive strength of sound concrete by taking and testing cores. It shall be permitted to determine steel reinforcement properties by removal of reinforcement samples and destructive testing.

6.4.2 *Core sampling of concrete for testing*—It shall be permitted to determine the compressive strength of sound concrete by taking cores from the members being evaluated.

COMMENTARY

6.3.3C The construction documents may not represent as-built conditions. Therefore, the licensed design professional is encouraged to research and verify using material testing as required, that the material properties obtained from record documents are representative.

6.3.4C When the results of material testing from original construction are available, these results may be used in analysis models. Additional testing may be required to confirm these material test results when degradation has occurred.

6.3.5C The licensed design professional should research and acquire available records from the original construction. Assessment, historical research, and documentation of the geometry, material properties, and detailing used in the construction are invaluable and may be used to reduce the amount of required in-place testing. The data gathered to determine strength should include the effects of material degradation, such as loss of concrete strength from sulfate attack and loss of steel area due to corrosion. The impact of deterioration on the expected strength and ductility of the section also needs to be considered in the evaluation.

The minimum number of tests is influenced by the data available from the original construction, the type of structural system, the desired accuracy, and the quality and condition of the in-place materials. The focus of the prescribed material testing should be on the principal structural members and specific properties needed for analysis. The licensed design professional should determine the appropriate number and type of testing needed to evaluate the existing conditions.

Care should be taken in selecting the location for sampling concrete. Core drilling should minimize damage of the existing reinforcement and should generally occur at locations where the coring will least affect the member strength.

6.4C—Test methods to determine or confirm material properties

6.4.2C Guidelines for core sampling and evaluating core strength data are given in ACI 214.4R.

CODE

Steel reinforcement shall be located by NDT before locating the cores to be extracted.

6.4.3 Concrete—The cores shall be selected and removed in accordance with ASTM C42 and ASTM C823. The equivalent specified concrete strength f_{ceq} shall be calculated using Eq. (6.4.3).

$$f_{ceq} = 0.9\bar{f}_c \left[1 - 1.28\sqrt{\frac{(k_c V)^2}{n} + 0.0015} \right] \quad (6.4.3)$$

where \bar{f}_c is the average core strength, as modified to account for the diameter and moisture condition of the core; V is the coefficient of variation of the core strengths; n is the number of cores taken; and k_c is the coefficient of variation modification factor, as obtained from Table 6.4.3.

6.4.3.1 Quantifications of concrete compressive strength by NDT alone shall not be permitted as a substitute for core sampling and testing. Nondestructive testing shall be permitted when a valid correlation is established with core sample strength test results and nondestructive test results.

6.4.4 Steel reinforcement—The size, number, and location of steel reinforcing bars or elements shall be established. If the original construction documents are not available and if the properties of the reinforcing bars are unknown, historical values provided in 6.3.6 shall be permitted in place of testing. If the grade of material is unknown, the lowest grade provided in Table 6.3.1b for a given historic period shall be used. When the grade of the material is unknown and historical default strength properties are not used, testing is required.

COMMENTARY

6.4.3C The equivalent specified strength determined using this procedure can be used in strength equations with the strength reduction factors from Chapter 5. The strength value obtained using this procedure is an estimate of the 13 percent fractile of the in-place concrete strength. This approach is specified in the Canadian Highway Bridge Design Code (CAN/CSA S6-06 2006) and is based on the approach proposed by Bartlett and MacGregor (1995).

6.4.3.1C ACI 228.1R provides information on NDT methods for assessment of concrete compressive strength and development of statistical correlations between NDT and core test results.

6.4.4C The ASTM designation or age of the structure may be known but the grade of reinforcement is not known. In this case, the lowest grade of reinforcement corresponding to the age or ASTM designation should be used. When no information is available, the licensed design professional may use the lower bound value of 185 MPa instead of testing.

Table 6.4.3—Coefficient of variation modification factor k_c

n	k_c
2	2.4
3	1.47
4	1.28
5	1.20
6	1.15
8	1.10
10	1.08
12	1.06
16	1.05
20	1.03
25 or more	1.02

CODE

6.4.5 Reinforcement sampling and testing—Removal of reinforcement samples and the laboratory destructive testing shall be permitted as a method of determining existing steel reinforcement properties. The yield and tensile strength for steel reinforcement shall be obtained in accordance with ASTM A370.

Steel reinforcement includes square, rectangular, and round bars with and without deformations, prestressing wire, bars, multi-wire strands, and structural shapes. Historically, wire rope and chain have also been used as reinforcement.

6.4.6. A minimum of three sample coupons, taken from different segments of reinforcement, shall be obtained from the members being evaluated. The equivalent specified yield strength f_{yeq} used for analysis shall be calculated by

$$f_{yeq} = (\bar{f}_y - 24)\exp(-1.3k_s V) \quad (6.4.6)$$

where \bar{f}_y is the average yield strength value from the tests, in MPa; V is the average coefficient of variation determined from testing; n is the number of strength tests; and k_s is the steel coefficient of variation modification factor, as obtained from Table 6.4.6.

6.4.7 If the properties of the connector steel are unknown, strength shall be determined by one of the following:

- (a) Testing of coupons taken from the connector steel.
- (b) Documentation giving connector steel properties in the original construction documents.
- (c) Use of historic default values given in 6.3.6.

6.4.8 Coupon specimens for the determination of yield and tensile strengths of structural steel shall be tested in accordance with ASTM A370. A minimum of three specimens shall be taken from representative elements.

COMMENTARY

6.4.5C Generally, the steel reinforcement used in a structure is of a common grade and strength. Occasionally, more than one grade of steel is used in a structure. Historical research from The Concrete Reinforcing Steel Institute (CRSI) (2001) may contain supplemental information on mechanical properties of the reinforcement used in different construction eras.

6.4.6C The equivalent specified yield strength determined using this procedure can be used in strength equations with the strength reduction factors from Chapter 5. The yield strength value obtained using this procedure is an estimate of the 10 percent fractile of the static steel strength. It is assumed that the difference between the yield strength measured during a coupon test and the static yield strength is 24 MPa. This approach is specified in the Canadian Highway Bridge Design Code (CAN/CSA S6-06 2006).

The factors in Table 6.4.6 reflect the uncertainty of the sample standard deviation when it is calculated for a small sample size. They are the 95 percent one-sided tolerance limits on the 10 percent fractile, and they have been reduced by a constant factor to be equal to 1.0 for $n = 30$ specimens.

6.4.7C The historic default value is obtained from ASCE/SEI 41-06.

Table 6.4.6—Steel coefficient of variation modification factor, k_s

n	k_s
3	3.46
4	2.34
5	1.92
6	1.69
8	1.45
10	1.32
12	1.24
16	1.14
20	1.08
25	1.03
30 or more	1.00

CODE

The equivalent specified yield strength f_y of each specimen shall be its reported yield strength. The f_{yeq} used for analysis shall be calculated by

$$f_{yeq} = (\bar{f}_y - 28)\exp(-1.3k_sV) \quad (6.4.8)$$

where \bar{f}_y is the average yield strength value from tests, in MPa; V is the average coefficient of variation determined from testing; n is the number of strength tests; and k_s is the steel coefficient of variation modification factor, as obtained from Table 6.4.6.

6.4.9 The sampling of prestressing steel reinforcement for strength testing shall be required if strength and historical data are not available. Testing of the prestressing reinforcement shall be in accordance with ASTM A1061/1061M.

6.4.10 If welding of reinforcement is required, carbon equivalent shall be determined in accordance with AWS D1.4.

6.5—Structural analysis of existing structures

6.5.1 The gravity and lateral-force-resisting structural systems shall be analyzed for the maximum effects on the affected members. Loads and load combinations shall be determined in accordance with the provisions of Chapter 5.

6.5.2 Analysis of the structure shall use accepted engineering principles that satisfy force equilibrium and the principles of compatibility of deformations and strains.

6.5.3 Analysis shall consider material properties, member geometry and deformation, lateral drift, duration of loads, shrinkage and creep, and interaction with the supporting foundation.

6.5.4 Members shall be analyzed considering the effect of any material degradation, bond loss, and the redistribution of forces in members and in the structural system as a whole.

COMMENTARY**6.5C—Structural analysis of existing structures**

6.5.1C Structural evaluation analyses are conducted to verify strength and serviceability. The analytical methods of 6.7 are used with factored loads to determine strength requirements for a combination of moments, shears, and axial loads of pertinent structural members. A service load level analysis may be required to evaluate serviceability, including deflections and expected crack size and distribution.

6.5.3C The licensed design professional is responsible for determining the appropriate method of analysis. Appropriate methods include linear elastic analysis, nonlinear analysis, and other traditionally accepted engineering analysis methods. If a linear elastic analysis method is used, the effects of cracking and other nonlinear effects should be included in the analysis using engineering approximations.

The analysis may include the effects of the size and member geometry to determine the forces on individual members of a structure. The analysis should consider external effects, including prestressing, material volume changes, temperature variations, and differential foundation movement.

6.5.4C Member deterioration and damage may result in distribution of internal forces different than the distribution of forces for which the structure was originally designed. The strength and integrity of prestressed structures with damaged prestressing reinforcement requires careful consideration to assess the impact of the damage. Redistribution of moments may exceed the moment distribution permitted in ACI 318M-11. The actual state of the structure should be accurately modeled to determine the distribution of forces.

CODE**COMMENTARY**

6.5.5 Analysis shall consider the load path from the load application through the structure to the foundation. Three-dimensional distribution of loads and forces in the complete structural system shall be considered unless a two-dimensional analysis adequately represents the part of the structure being evaluated.

6.5.6 Analysis shall consider the effects of previous repairs and of any previous structural modifications on the behavior of the structure.

6.5.7 The analysis shall be based on available documentation, as-built dimensions, and the in-place properties of the structure including section loss. The assessment of in-place material properties shall be as described in 6.3.

6.6—Structural serviceability

6.6.1 When serviceability problems have been identified pursuant to 6.1.2, the licensed design professional shall perform a serviceability evaluation based on the in-place geometry and properties of the structure.

6.7—Structural analysis for repair design

6.7.1 The structural analysis used for repair design shall consider the structural repair process. The analysis shall consider the effects of the sequence of load application and material removal during all phases of the evaluation and repair process.

6.7.2 Structural modeling shall account for repairs where the materials change through the section.

6.7.3 Section analysis shall use principles of mechanics and shall assume (a), (b), or (c): (a) full composite action with no slip at interfaces between repair materials and existing materials; (b) separate action with full slip between repair

Redistribution of forces may be determined using material nonlinear analysis, by load tests described in ACI 437R, or by linear analysis, which bounds the limits of redistributed forces.

6.5.5C The evaluation of load effects requires consideration of both the load paths through the structure and how the forces are distributed in members.

6.5.6C Modifications to structures in the form of repairs, alterations, or additions may affect the force distribution and load path in a structure. The effect of these modifications should be accounted for in analytical models.

6.5.7C Available documentation may include original drawings, specifications, shop drawings, structural assessments, testing, and geotechnical reports. Deviations between the existing construction and construction documents are to be identified and recorded. If section loss has occurred, a more accurate analysis may be developed by direct measurement of the section, and by calculation of section properties based on actual conditions.

6.6C—Structural serviceability

6.6.1C Structural serviceability problems may include, but are not limited to, deflections, vibrations, leakage, and objectionable cracking. The data gathered for the purpose of determining serviceability should include the effects of material degradation, such as loss of concrete strength from sulfate attack or loss of steel area due to corrosion.

The specific performance criteria and the intended function of an individual structure should be defined. Floor deflection criteria can be found in ASCE/SEI 7. Vibration criteria are given in Murray et al. (1999).

6.7C—Structural analysis for repair design

6.7.1C The construction process may involve the application and removal and replacement of loads. The analysis needs to consider the effects of the application and removal of construction loads to determine the maximum loading during all construction phases. The additional applied loads may be due to prestressing, vibration, material volume changes such as creep and shrinkage, temperature changes, effect of shoring, and unequal deformation of supports.

6.7.3C Depending on the repair construction process and the selection of repair materials, the repair materials and the existing concrete or reinforcement may not act compositely. The analysis should appropriately model the behavior of the correct degree of the expected composite action of the

CODE

and existing materials; or (c) partial composite action with friction at interfaces between repair and existing materials.

6.7.4 *Seismic analysis of repaired structure*

6.7.4.1 The interaction of structural members and nonstructural components that affect the response of the structure to seismic motions shall be considered in the analysis.

6.7.4.2 Existing, repaired, and added supplementary members assumed not to be a part of the seismic-force-resisting system shall be permitted provided their effect on the response of the system is considered and accommodated in the repair design. Consequences of failure of structural members that are not a part of the seismic-force-resisting system and nonstructural components shall be considered.

6.7.4.3 The method of analysis shall consider the structural configuration and material properties after repair.

6.8—Strength evaluation by load testing

6.8.1 Load testing in accordance with ACI 437-13 shall be permitted to supplement the analysis or to demonstrate the strength of the original or repaired structure

6.8.2 Model analysis shall be permitted to supplement analysis.

COMMENTARY

repaired structure. An example of partial composite behavior are beams that contain shear studs to develop nominal strength, yet lack bond between the overlay and substrate. In this situation, the overlay and substrate do not maintain strain compatibility.

6.7.4.3C Procedures for seismic rehabilitation of concrete buildings, including analysis, are provided in ASCE/SEI 41 and supplemented in ACI 369R. These references provide details for forces, rehabilitation methods, analysis and modeling procedures, and seismic rehabilitation design.

6.8C—Strength evaluation by load testing

6.8.1C The licensed design professional may lack information required to determine the strength of deteriorated or repaired structural members. Field assessments, including destructive and in-place testing, can provide some of the information required, but the costs for these assessments can be significant; however, the results of strength evaluation may still be inconclusive due to unknown effects of existing conditions or interaction with the repair. In these cases, load testing can verify the strength of a member and of a structural system and to achieve a reliable estimate of the short-term strength. In such cases, load testing may provide the most effective means of verifying the strength of an existing structure or element. Load testing can also be a valuable tool for evaluating the effectiveness of structural repairs. For example, load testing, as defined in ACI 437-13, can be performed to determine that the service load deflection and cracking are acceptable.

6.8.2C This code permits model analysis to be used to supplement structural analysis and design calculations. Model analysis involves the construction and experimental testing of full or scale models of structure components, assemblages, or systems. Documentation of the model tests and subsequent interpretation should be provided with the related calculations. Model analysis should be performed by an individual having experience in this technique. Guidelines for model analysis are given by Sabnis et al. (1983).

CODE

COMMENTARY

CHAPTER 7—DESIGN OF STRUCTURAL REPAIRS

7.1—General

Repaired structures, structural members, and connections shall be designed to have design strengths at all sections at least equal to the required strengths calculated for factored loads and forces in such combinations as stipulated in this code.

7.2—Strength and serviceability

7.2.1 Repaired members shall be designed to have adequate stiffness to limit deflections, vibrations, cracking, or any deformations that adversely affect strength or serviceability of a structure.

7.2.2 Repair design and construction procedures shall consider the loading, internal forces, and deformations in both the existing and repaired structure during the repair process.

7.3—Behavior of repaired systems

7.3.1 Repairs incorporating new members shall be designed to integrate the new members with the existing structure, creating a structural system capable of sharing and transferring loads.

7.3.1.1 The design of the repair system shall consider the structural interaction between the existing structure and new members. The effect of the new members on the existing structure shall be evaluated according to the design basis code.

7.2C—Strength and serviceability

7.2.1C Adequate stiffness needs to be determined on a project-specific basis and is a function of the structure type, the desired performance of the structure, and loading conditions and use.

7.2.2C During the repair process, it may not be possible or practical to relieve existing stresses or deformations. Consideration should be given to the in-place internal forces and deformation present in the existing structure during the repair and the subsequent internal forces from the design loads that the repaired section will be required to resist. Internal forces and deformations caused by existing loads may be locked in by the repair.

Analysis to evaluate the effects of structural modifications, such as installation of slab openings, should verify that strength is adequate and that all serviceability conditions are met. As an example, creating a large opening in structural slabs may necessitate cutting reinforcement, which can significantly influence the global behavior of the structure. Supplementary strengthening may be required to address force redistribution that can exceed the existing strength. Slab punching shear strength should be evaluated for openings at the intersection of column strips to verify that the slab is still adequate. This is especially critical near corner and edge columns where the slab shear is typically highest.

7.3C—Behavior of repaired systems

7.3.1C Repair of an existing structure may be achieved by improving the global behavior of the structure by adding new structural members that act integrally with the existing structural system. Repair can also be achieved by improving the behavior of the existing members that incorporate repair materials or systems.

Load sharing and load transfer should exist between the existing structure and the new members so that the load path and force distribution assumed by the licensed design professional can occur. The effects of adding new members on the global stiffness and force distribution should be considered.

7.3.1.1C The design of the repair system should consider connections of new members to the existing structure. Connections of new members should be designed to transfer design forces between new members and the existing structure.

New members may need to be separated from adjacent existing members to prevent or minimize interaction that

CODE

7.3.2 Repairs to existing members shall account for force transfer at the interface between the existing member and the repair material or repair system. It is permissible to use ACI 318-11 provisions in the design of force transfer between new and existing concrete.

7.3.2.1 Structural repairs shall maintain composite behavior under service loads. Repairs shall be designed for the material and existing substrate to perform as a composite section at service loads.

COMMENTARY

may result in damage to adjacent portions of the structure. Transfer of forces between new and existing members should not compromise the performance of the existing structural system.

7.3.2C In repairing existing members, induced forces on the repaired member are shared between the existing member and the repair material or system. The repair should be designed to allow for transfer of forces between the components.

The requirements for composite behavior between the repair and the existing member may vary depending on the type of repair (structural or nonstructural) and the performance criteria at service, and ultimate limit states. While certain designs require composite behavior up to an ultimate limit state, others may be limited to service conditions. Composite behavior can be achieved by chemical bonding, mechanical means, or a combination thereof. The design should specify the repair materials and techniques that will develop the level of composite behavior to achieve the intended performance of the repaired member. Specific reference is made to ACI 318-11 Sections 11.6 and 17.5 for design requirements for force transfer between new and existing concrete. Techniques other than shear-friction may be acceptable.

Design provisions for fiber-reinforced polymer (FRP) are provided in ACI 440R. Design provisions for composite structural steel sections are provided in the Steel Construction Manual (AISC 325-11).

Repair design may not require all components of the repair to remain composite with the existing member at factored loads. An example of this may be the top-side concrete repair of a cantilevered balcony adjacent to the façade of a building. As long as the top reinforcing bars in the cantilever extending from the wall are developed beyond the repair area, the bond of the repair area to the substrate is not critical. Strength calculations should be based on the achievable level of composite behavior.

Spalling of cover concrete or other repair materials may be tolerated at conditions such as a seismic event. In these situations, the member may achieve design strength without the repair material in place. Section 7.3.2.1 applies in these events.

7.3.2.1C Repairs may be needed to improve durability or aesthetics. Therefore, they may not require composite behavior under service or factored loads. In some cases, such as overhead repair or façade repair, repairs may be required to maintain composite behavior under both service and factored loads to prevent falling hazard. If composite behavior cannot be achieved, the repaired system should have sufficient redundancy or connections to prevent falling hazards.

The licensed design professional will typically design and detail repairs to be fully composite under factored strength conditions, and that design will typically satisfy service load strength requirements. The provisions of ACI 318M-11 (Sections 11.6 and 17.5) can be used to design fully composite

CODE

COMMENTARY

7.4—Bond

7.4.1 The required bond strength shall be at least 1.5 times greater than the calculated design bond force at the repair material to existing concrete interface.

7.4.1.1 The measured bond strength shall not be less than the lower of the required bond strength or the tensile strength of the existing concrete. If the tensile strength of undamaged existing concrete is unknown, it shall be permitted to take the tensile strength as $0.33\lambda\sqrt{f'_c}$.

7.4.1.2 Testing to measure bond strength of repair materials to existing concrete and the tensile strength of the existing concrete shall be in accordance with ASTM C1583.

7.4.1.3 It shall be permitted to provide supplementary reinforcement to achieve required bond strength.

7.4.2 The licensed design professional shall verify the concrete substrate has adequate strength to sustain and transfer design forces of externally attached reinforcement.

repairs. Alternately, principles of engineering mechanics can be used to assess force demand at service load conditions.

7.4C—Bond

7.4.1C A low bond strength may not be sufficient to satisfy durability requirements. The International Concrete Repair Institute (ICRI) No. 210.3 discusses bond strength in repairs.

7.4.1.2C Bond strength testing may not be required in all repairs. ACI 503R and ICRI No. 210.3 provide guidance for performing the test and the test frequency.

7.4.1.3C For bonded repairs, the tensile and shear strength of the repair system interface should be greater than that of the substrate. Lower bond strength may be acceptable for situations with high substrate strength and the bond strength meets design requirements. When failure occurs within the substrate, failure should occur within the substrate at a depth sufficient to preclude bruising or microfractures causing low bond strength.

A properly prepared substrate is achieved by removing the layer of existing deteriorated, damaged, or contaminated concrete. The exposed sound concrete is then roughened and cleaned to allow for adequate bond of a repair material or protective coating to the substrate.

Adequate bond means continued adhesion of the applied materials to reinforcement or other surfaces against which it is placed. The bond should be sufficient to resist stress due to tension, shear, and compressive forces due to differential shrinkage and creep between repair materials and the existing concrete. Bond between the repair material and reinforcement should develop the design tensile and compressive strength of the reinforcement. The adhesion or cohesion provided by the bond should not deteriorate unacceptably over the anticipated service life of the repair.

Heavy corrosion products on the reinforcement, on the concrete surface, or both, inhibit development of bond between existing concrete and new repair materials. Heavy corrosion products can be considered Rust Grade 5 or less in accordance with ASTM D610. Corrosion or other bond-inhibiting products should be removed to achieve proper bond.

Shear-friction provisions of ACI 318M may be used to design supplementary reinforcement.

7.4.2C If external reinforcement is added to a structure, the minimum strength of the existing concrete necessary for the transfer of load from the external reinforcement to the structure needs to be established. For example, ACI 440.2R requires a minimum concrete compressive strength of 17 MPa to bond FRP.

CODE

7.4.3 Adhesives used to bond existing concrete, repair materials, and repair reinforcement shall transfer required forces between elements.

7.4.4 The selection of adhesives used in repair shall consider load type and duration and the effect of exposure conditions on adhesive properties.

7.5—Materials

7.5.1 The licensed design professional shall consider the properties of repair materials and systems in designing repairs and in specifying repair materials and repair procedures. These include, but are not limited to: physical properties of the repair materials, type of application, adhesion, shrinkage, thermal movement, durability, corrosion resistance, installation methods, curing requirements, and environmental condition.

COMMENTARY

7.4.3C Guide for the verification of bond can be found in ACI 440R, ACI 503R, ICRI 210.3, and ASTM C1583.

7.4.4C ACI 503.5R, ACI 503R, and ACI SP-165 (Swami and Gaul 1996) provide guidance for the selection of adhesives.

7.5C—Materials

7.5.1C Physical properties include mechanical, chemical, and electrical properties. Documentation should be obtained for properties of each repair material. The stated properties should be verified that they satisfy the project requirements. ACI and ICRI provide guidelines for the selection of repair materials (ACI 301M, ACI 318M-11, ACI 503R, ACI 503.5R, ACI 503.6R, ACI 546.3R, ICRI No. 320.2R, ICRI No. 320.3, ICRI No. 330.1, and ICRI No. 340.1).

The design of a repair should consider the compatibility of the repair materials and the materials of the existing structure. Compatibility of repair materials and systems include dimensional compatibility, bond compatibility and durability, mechanical compatibility, and electrochemical and permeability compatibility. Generally, the intent is to use a repair material or repair system that has physical, mechanical, and other properties that are as close as possible to those of the parent material to provide long-term performance.

Individual repair materials may have different properties yet will perform satisfactorily when combined in a repair system. An example of this is where materials with differing thermal coefficients of expansion may be used, provided that the overall performance of the system is not affected by thermal changes.

Volume stability of repair materials should be accounted for in the design of the repair. Volume stability, also known as dimensional compatibility, refers to initial and time-dependent changes in the volume of the repair material after placement. Volume stability is often measured as changes in a linear dimension of the repair and should be accounted for in the design of a repair system. Autogenous shrinkage, chemical shrinkage, degree of restraint, environmental conditions, drying shrinkage, creep, thermal changes, moisture absorption, and other factors all affect volume stability. Experience has shown that volume change of the repair materials has often been the cause of poor performance of repairs. Properties of repair materials should be selected for volume stability and dimensional compatibility with the existing structure to reduce the probability of cracking caused by volume changes. Dimensional compatibility refers to movement or shrinkage differences in a single direction or multiple directions.

Volume stability is discussed in ACI 209R, ACI 209.1R, ACI 546.3R, and ICRI No. 320.2R.

Repair materials such as portland-cement concrete, portland-cement mortar, polymer-cement concrete, polymer concrete, fiber-reinforced concrete, resin-based materials, and similar products are commonly used. The repair mate-

CODE**COMMENTARY**

7.5.2 Materials already in use in a structure shall be permitted to remain in use where such materials conform to the design basis code.

7.5.3 Materials conforming to ACI 318M-11 or permitted by this code shall be used for repairs and alterations.

7.5.4 Alternate materials shall be permitted according to the licensed design professional's approval and in accordance with 1.4.

7.6—Design and detailing considerations

7.6.1 Repair design shall be based upon the member conditions identified in Chapter 6.

7.6.2 *Concrete*—The in-place properties of the existing concrete, as identified in Chapter 6, shall be used in the repair design.

7.6.3 Reinforcement

7.6.3.1 Reinforcement that is damaged or corroded shall be permitted to remain. The effective cross-sectional area shall be used in the repair design in accordance with the design basis code. The effect of corrosion damage on development of steel reinforcement shall be assessed by the licensed design professional. At locations where deformations are no longer present, reinforcing bars shall be considered as smooth bars.

rials do not necessarily contain portland cement. Repair materials should be selected to achieve the necessary service, strength, and durability requirements.

The selection of reinforcement material should consider the durability, performance at elevated temperatures, and ductility. Electrical and chemical reactivity between the reinforcement, the repair material, and the existing reinforcement should also be considered.

Refer to ACI 440.1R for internal FRP reinforcement, ACI 440.2R for externally bonded FRP reinforcement, and ICRI No. 330.1 and ACI SP-66 (ACI Committee 315 2004) for steel reinforcement.

Required properties of the repair reinforcement should be specified in the contract documents. Specified properties are dependent on the requirements of the repair and may include yield strength, tensile strength, chemical properties, elongation, material type, and moduli.

7.6C—Design and detailing considerations

7.6.2C The extent and cause of deterioration and the concrete strength and quality should be determined, including compressive strength, chlorides, carbonation, sulfate attack, alkali-silica reaction, physical damage, corrosion-induced spalling, and cracking.

Chloride penetration can cause corrosion that can lead to cracking and spalling. The depth of a spall reduces the effective area of concrete section. Degradation of the concrete will affect the compressive strength of the concrete.

7.6.3.1C Strength evaluation and repair design should consider the in-place condition of the existing reinforcement, including the effective cross-sectional area of the reinforcing bars after removal of corroded material. The effective area is calculated using the remaining effective diameter of the reinforcing bar accounting for the loss of metal due to corrosion considering the location of the loss of area within the member. Further considerations may also include the location of the corroded areas, loss of confinement, the loss of bond, and the effect of corrosion on member strength. If the structure is fire damaged, steel reinforcement may

CODE

7.6.3.2 Design shall consider the location and detailing of the existing reinforcement in accordance with the evaluation of Chapter 6.

7.6.3.3 Both existing and new reinforcement shall be adequately developed. Development length shall be permitted to be calculated based upon development in both the existing concrete and new materials.

7.6.4 *Prestressed structures*

7.6.4.1 The effects of existing prestressing shall be included in the repair design.

7.6.4.2 The effects of modifications to existing structure geometry, existing damage conditions, prestressing force release, and construction sequence shall be included in the repair design.

COMMENTARY

be annealed, and the yield strength reduced. Refer to ACI 216.1M for additional guidance. Specific durability requirements related to corroded reinforcement are addressed in Section 8.4.

7.6.3.2C The location and detailing includes the horizontal and vertical positions, orientation, geometry of the reinforcement, development of reinforcement, and the presence of hooks and crossties. Field examination to locate existing reinforcement may be required.

7.6.3.3C Reinforcement development may be inadequate due to corrosion, mechanical damage, insufficient or loss of concrete cover, delaminated concrete, concrete strength, or other conditions. The design of the repair should evaluate the required development length to develop the design force of existing reinforcement. Detailing of the repair should include the proper development of new reinforcement to achieve the design force. ACI 318-11 provides guidance for detailing of steel reinforcement. ACI 440.1R and ACI 440.2R provide detailing guidance for internal FRP reinforcement and externally bonded FRP reinforcement, respectively.

7.6.4.1C Requirements for repair of structures with bonded and unbonded prestressing are different. Post-tensioned structures (with bonded and unbonded tendons) are often cast-in-place monolithic structures, whereas pretensioned structures (with bonded strands) are often single-span precast structures. Each system is unique and should be individually considered. The repair of prestressed structures requires an assessment of the existing tendons. Repair of unbonded tendons may require tendon detensioning. Guidance for evaluation methods and repair techniques of unbonded post-tensioned structures is provided in ACI 423.4R, ACI 222.2R, ICRI No. 210.2, ICRI No. 320.4, and PTI DC80.2 (2010).

7.6.4.2C Analysis of prestressed structures is required to evaluate the effect of damaged or severed prestressing reinforcement on the structural strength and performance. The effect of a severed bonded tendon is typically localized because the severed tendon will be effective after a development length is achieved and the full strength of the tendon will be reestablished. For structures with bonded tendons, shoring, if necessary, may only be required locally at the repair area.

Review of grouting quality assurance and supervision documents may be required to ensure adequately grouted tendons after the application of prestress. The presences of voids, moisture in ducts, and the presence of chlorides or carbonation need to be identified in the existing grout. Field evaluation of existing grout may be required even when existing documentation of the original construction is available.

Unbonded tendons are designed to be permanently debonded from the member and often extend over multiple spans. As a result, damage or discontinuity of tendons at one

CODE

COMMENTARY

7.6.4.3 Stresses in remaining section after concrete removal shall not exceed the limits established in the design basis code.

7.6.5 *Anchoring to concrete*—Post-installed anchors and dowels shall be designed to transfer design forces to the substrate considering anchor failure modes and the condi-

location will reduce the strength for the entire length of the tendon.

Analysis to evaluate the effects of structural modifications such as installation of slab openings should verify that strength is adequate and that all serviceability conditions (for example, deflections) are met. Large openings can significantly influence the global behavior of the structure and may result in strengthening needs.

Corrosion on prestressing strands may have an effect on strand integrity and strength. Prestressing strand requires examination for conditions such as corrosion pitting and hydrogen embrittlement (refer to ICRI No. 210.2 and ACI 222.2R). Analysis methods of unbonded post-tensioned structures are provided in ACI 222.2R, ACI 423.4R, ICRI No. 210.2, ICRI No. 320.4, PTI (2006), and PTI DC80.2 (2010).

Repair and structural modification may require detensioning of prestressing tendons. Unbonded tendons should be detensioned in a controlled manner to ensure performance and safety. Unless not needed based on analysis, all interrupted unbonded tendons should be reanchored and restressed to restore full structural strength. This can be achieved by splicing or by installing new tendon anchorages at the edge of new opening, and after concrete reaches the necessary strength, the tendons are restressed.

When unbonded tendons are severed, the prestressing force will be lost for the full length of the tendon. Releasing or cutting tendons may affect multiple spans and may require shoring beyond the area where cutting or releasing of tendons occurs. Adjacent spans require temporary shoring depending on the number of tendons severed at one time and the applied loads. Analysis based on actual loading at the time of the modification may show shoring to be unnecessary.

Guidance for analysis of unbonded post-tensioned structures is provided in ICRI No. 320.4, PTI (2006), and PTI DC80.2 (2010).

7.6.4.3C Removing surface concrete from a prestressed member may cause excessive compressive and tensile stress in the remaining concrete section and may alter secondary forces and moments due to prestressing in indeterminate structures. This condition is more critical for prestressed joists and girders that have a relatively small section and large prestressing force. Slabs are less critical due to the relatively small initial precompression. Repairs of prestressed members may result in changing the class of the members from Class U to Class T or from Class T to Class C as defined in ACI 318M-11. This change is acceptable as long as durability and strength are addressed as part of the repair design. The impact of removing concrete from a post-tensioned structure is addressed in Scollard and Bartlett (2004). ICRI No. 320.4 provides guidance for removing concrete around anchorages and splices to prevent catastrophic loss of anchorage.

7.6.5C The design of post-installed anchors requires a careful consideration of the loads to be resisted. As an example, a long-term tensile load may require a different

CODE

tion of the substrate into which the anchor is installed. The design of post-installed anchors shall be in accordance with ACI 318M-11.

7.6.6 *Repair geometry*—Configuration of repairs shall consider the potential for stress concentrations and cracking in both the existing structure and the repair area.

7.7—Repair using supplemental post-tensioning

7.7.1 Supplemental post-tensioning shall be permitted for repair and rehabilitation of structures.

7.7.2 Design of repair shall include the effects of the supplemental post-tensioning on the behavior of the structure.

COMMENTARY

anchoring mechanism than a short-term wind or seismic load. Anchors should have adequate strength to transfer design forces across all interfaces and into the existing member. All possible anchor failure modes should be considered to determine its design strength. Anchors should be selected considering the expected concrete substrate cracking condition. For example, post-installed anchors used in the tension zone of concrete members and in structures located in regions of moderate or high seismic risk should be able to transfer the design seismic forces assuming a cracked concrete condition.

Design of post-installed anchors is provided in ACI 318M-11, Appendix D. Building code requirements have prompted the need to verify performance of post-installed anchors in both cracked and uncracked concrete. ACI 355.2 and 355.4 provide the standard required for qualifying post-installed anchors in cracked and uncracked concrete. Specifications for post-installed anchors should include installation, testing, and inspection procedures.

For mechanical anchors, manufacturer's installation instructions should specify procedures for drilling, hole cleaning, installation, torque magnitude, and procedures to engage the anchor.

For adhesive anchors and dowels, hole cleaning and moisture conditions are critically important to almost all adhesive anchor installations. Manufacturer's installation instructions should specify procedures for drilling, hole cleaning, installation, and the care to be taken until the adhesive has cured.

Testing and inspection of post-installed anchors should conform to Chapter 10. Many building codes require that adhesive anchors be installed under special inspection procedures to ensure that the installation is correctly performed in accordance with the design and manufacturer's procedure.

7.6.6C Some repair shapes with sharp reentrant corners can cause stress concentrations that may result in cracking. Long, slender (high aspect ratio) repair areas also tend to result in cracking. The shape of the repair should be considered to reduce stress concentrations and possible cracking. ICRI No. 310.1R provides guidance for repair shapes. Methods to reduce cracking in concrete repairs include configuring to maximize right-angle geometry, avoiding reentrant corners, uniform depth of edge and uniform depth of substrate, surface preparation, and undercutting of reinforcement with no feather edge conditions.

7.7C—Repair using supplemental post-tensioning

7.7.1C The supplemental post-tensioning can be applied to the structure externally, internally, or both.

7.7.2C Supplemental post-tensioning can introduce additional moment, shear, and axial forces within the existing structure that should be considered in the design and detailing of the repair. The internal forces induced by the supplemental post-tensioning can be significant. For statically indeterminate structures, restraint to post-tensioning deformations can

CODE**COMMENTARY**

7.7.2.1 Stresses due to supplemental post-tensioning shall be combined with existing stresses and the total shall not exceed the limits in the design basis code.

7.7.2.2 Design of supplemental post-tensioning shall provide for the transfer of post-tensioning forces between the post-tensioning system and the structure. Design of concrete supplemental post-tensioning anchorage zones shall be in accordance with ACI 318M-11. Design of steel brackets and supplementary structural steel shall be in accordance with ANSI/AISC 360-10.

7.7.3 Provisions shall be made for effects of post-tensioning, temperature, and shrinkage on adjoining construction, including immediate and long-term deformations, deflections, changes in length, and rotations due to prestressing.

7.7.4 Post-tensioning losses shall be included in the design of supplemental post-tensioning systems.

7.7.5 Repair construction documents shall define the repair sequence, including tendon placement, anchorage, and stressing of the post-tensioned system.

7.8—Repair using fiber-reinforced polymer (FRP) composites

7.8.1 Fiber-reinforced polymer in conformance with ACI 440.6M shall be permitted to repair existing concrete structures.

result in significant internal forces. Refer to ICRI 330.1 for selecting strengthening systems for concrete structures.

7.7.2.1C Adding supplemental post-tensioning to a prestressed member may cause excessive compressive and tensile stress and may alter secondary forces and moments. External post-tensioning may result in changing the class of the members from Class U to Class T or from Class T to Class C as defined in ACI 318M-11. This change is acceptable as long as durability and strength are addressed as part of the repair design.

7.7.2.2C Anchorage of the new post-tensioned reinforcement should be designed and detailed for the transfer of post-tensioning forces to the existing structure. The licensed design professional should consider bearing, spalling, and bursting forces created at anchor zones. Strut-and-tie modeling, as given in ACI 318M-11, may be used to design post-tensioning anchorage zones.

7.7.3C The additional post-tensioning may be restrained by adjacent stiff members such as walls, and reduce the effect of the prestressing on the intended member or have unintended effects on the adjacent construction. In some instances, calculations may reveal that modifications are not necessary.

7.7.4C Losses include seating (anchorage); elastic shortening; creep of original concrete; shrinkage of original concrete; creep of repair material; shrinkage of repair material; tendon relaxation; and friction between the post-tensioning reinforcement and ducts, bearings, or deviators. Assessment of losses of supplemental post-tensioning force should consider the existing conditions of the repaired elements, as the members may have already experienced time-dependent creep and shrinkage.

7.7.5C Repair design using supplemental post-tensioning systems should include contract documents for installation sequence including shoring, removal of concrete, placement of new material and reinforcement, additional anchorage requirements, horizontal shear transfer requirements, curing, and stressing. Installation of supplementary post-tensioning involves application of significant forces, which may require project safety and protection procedures.

7.8C—Repair using fiber-reinforced polymer (FRP) composites

7.8.1C Fiber-reinforced polymer fabrics, bars, or shapes can be used as externally bonded reinforcement, internal reinforcement, and as internal or external prestressed reinforcement. FRP shapes may be used as additional stand-alone structural members. Design and detailing of externally bonded FRP systems should be consistent with ACI 440.2R. Particular attention should be given to strength increase

CODE

7.8.2 Structural members repaired or modified with externally-applied FRP composites shall have adequate unrepaired strength, as defined in 5.5.

7.9—Performance under fire and elevated temperatures

7.9.1 Design of the repair system shall consider elevated temperature performance and shall comply with the fire resistance ratings of the structural members and other fire safety requirements in accordance with the general existing building code.

7.9.2 It shall be permitted to design a repair without supplemental fire protection if the unrepaired member has adequate strength in accordance with 5.5.

7.9.3 The properties of the specified repair materials at elevated temperatures shall be considered.

7.9.4 Repairs using adhesives shall consider their performance at elevated temperatures.

COMMENTARY

limits, service limits, and determination of FRP material design properties.

Design and detailing of internal FRP reinforcement should be consistent with ACI 440.1R. Particular attention should be given to service limits and determination of FRP material design properties.

When internal prestressed FRP reinforcement is used, the design and detailing should be consistent with ACI 440.4R.

7.8.2C Unless protection is provided, to prevent sudden failure of the member in case the FRP system is damaged or becomes ineffective (such as in an extraordinary event like fire), the structural member should have adequate strength without the FRP reinforcement to support factored loads, as defined for extraordinary events in Chapter 5. The design and use of externally bonded FRP may be limited by the service requirements of the repaired member.

7.9C—Performance under fire and elevated temperatures

7.9.1C Regardless of the repair system used, performance of the repaired element under fire and elevated temperatures should be evaluated and the system should be detailed and material selected to provide adequate protection. The repaired elements should comply with applicable building code requirements and relevant fire regulations valid at the project location. Structures renovated for different use or strengthened to support higher loads may require a more stringent fire rating than the original structure. Other requirements such as flame spread and smoke density should also be considered in accordance with the general existing building code and ASTM E84.

7.9.2C A repair system can be selected without additional fire protection provided that the existing unrepaired member has adequate strength to support the loads, as defined in 5.5, during a fire event. Fire performance requirements and evaluation procedures are outlined in ACI 216.1M, ACI 318M-11, ACI 440.2R, ASCE/SEI/SFPE 29, and AISC Design Guide 19 (Ruddy et al. 2003).

7.9.3C Repair material specifications should comply with the requirements of relevant fire regulations valid at the project location. Where there is a conflict between the properties of specific products or systems and fire regulations, alternative repair principles or methods should be used to avoid such a conflict. In general, polymer mortar and polymer concrete have higher coefficients of thermal expansion and higher resistance to water vapor transmission and lower resistance to fire and elevated temperatures compared to cementitious alternatives.

7.9.4C ACI 440.2R reports that the physical and mechanical properties of the resin components of FRP systems are influenced by temperature and can degrade at temperatures close to and above their glass-transition temperature T_g . An

CODE

COMMENTARY

acceptable service temperature for FRP is established by ACI 440.2R as $T_g - 15^\circ\text{F}$. This value accounts for typical variation in test data for dry environment exposures. Adhesive-bonded FRP reinforcement should not be used if the maximum service temperature is greater than $T_g - 15^\circ\text{F}$. A service temperature exceeding this limit is considered an elevated temperature and should be addressed using adhesive system with a higher T_g value or using alternate repair systems.

When performance at elevated temperatures is critical, the use of mechanical or cementitious grouted anchors may provide a higher level of reliability than adhesive anchors and adhesively bonded systems.

An appropriate fire rating can be achieved using materials with an established fire rating. In the absence of an established fire rating, detailed fire analysis may be used to establish a fire rating.

Most repair adhesives cannot resist elevated temperatures (temperatures greater than their glass transition temperature) such as the temperatures generated during a fire event. Because of the degradation of most adhesive-based repair systems (such as adhesive-bonded anchors and adhesive-bonded reinforcement) at elevated temperature, the bond strength of these systems should be assumed to be completely lost when temperatures exceed the glass transition temperature. The repair material should be selected such that the critical temperature is greater than the expected service temperature. The critical temperature should be taken as the lowest $T_g - 15^\circ\text{F}$ of the components of the repair system.

Adhesive-based repair systems can be considered effective during a fire event if a fire protection system is used that maintains the temperature of the adhesive-based system below its critical temperature.

7.9.5 Supplemental fire protection to improve the fire rating of repaired systems shall be permitted.

7.9.5C Standard fire protection systems can be used to increase the fire rating of repaired systems. National codes and professional organizations list generic ratings for concrete structural members, giving the minimum thickness of concrete cover needed to protect the main steel reinforcement from fire effects (IBC 2012; NFPA 5000 2009; PCA 1985,1994). In addition to increasing the cover thickness, fire performance of reinforced and prestressed concrete members may be enhanced by fire protection systems as proven by fire testing or analytical methods (ACI 216.1M). Concrete cover for nonmetallic reinforcement may need to be substantially greater than cover for steel reinforcement to achieve the same fire resistance rating.

7.9.6 Fire rating of repaired systems, based on ACI 216.1, shall be permitted.

7.9.6C The fire rating of a repaired system or assembly can be determined in accordance with ACI 216.1M, which requires the application of the expected service load to the test specimen. The applied load should reflect the use of the tested member in terms of magnitude and layout.

The criteria for evaluating a structure for fire safety are different than that for strength design and typically incorporate lower material strengths and load factors, and may not require the use of strength reduction factors. The licensed

CODE**COMMENTARY**

design professional should verify that the fire-reduced strength of the member is greater than the force demand due to expected service loads during the fire event. The fire-reduced strength should be based on reduced material strengths for the maximum expected temperature in a fire event, which can be determined in accordance with ASTM E119 and ACI 216.1M.

CODE

COMMENTARY

CHAPTER 8—DURABILITY

8.1—General

8.1.1 Durability of all materials incorporated into a repair shall be considered for individual repairs, the overall durability of the repaired structure, and the interaction of the repair system with the existing structure.

8.1.2 Repair materials and methods shall be selected that are intended to be compatible with the structure, durable within the service environment, and consider the anticipated maintenance.

8.1C—General

8.1.1C The durability of materials incorporated into a repair depends on their ability to withstand the environment where they are installed. The durability of repairs is dependent on the compatibility between repair materials, the structure, and the surrounding environment. To achieve compatibility, the repair and the structure need to interact on several levels, including chemical, electrochemical, and physical behavior, without detriment.

8.1.2C The design service life of a structure and repaired members is a goal established by the licensed design professional to achieve an economical repair that satisfies both safety and serviceability requirements. Only through satisfactory repair construction practices, including application of the specified repair materials, is it possible to achieve that goal. The design service life of the structure and repaired members, including maintenance requirements, should be estimated in consultation with the owner and consideration of the properties of the materials. Such design service life should be reflected in the repair design and maintenance requirements, as well as be incorporated into the construction documents. A repaired section is considered to be the combination of the installed repair material(s) and the substrate material(s). Service life is discussed in ACI 365.1R.

Some examples of end-of-service life include:

- (a) Structural safety is unacceptable due to material degradation or the actual strength is less than the required strength.
- (b) Maintenance requirements exceed resource limits.
- (c) Aesthetics become unacceptable.
- (d) Structural functionality is no longer sufficient.
- (e) Deformation capacity of the structure has been degraded due to a seismic event.

The cause of degradation should be addressed as a first step in predicting each type of service life. Types of degradation include:

- (a) Mechanical (abrasion, fatigue, impact, overload, settlement, explosion, vibration, excessive displacement, loads, or ground motion from a seismic event).
- (b) Chemical (alkali-aggregate reaction, sulfate attack, acid dissolution, soft water leaching, and biological action).
- (c) Physical (freezing and thawing, scaling, differing coefficients of thermal expansion, salt crystallization, radiation exposure [ultraviolet light], fire, and differential permeability between materials).
- (d) Reinforcement corrosion (carbonation, corrosive contaminants, dissimilar metals, stray currents, and stress corrosion cracking).

Preparation methods, materials, placement, and installed systems should be defined in the construction documents to reflect the design intent and requirements to achieve compatibility.

Repaired sections should be resistant to expected service conditions that can result in degradation, including the

CODE**COMMENTARY**

causes of degradation listed previously within the design service life.

Repaired sections should be resistant against:

(a) Chlorides and other corrosive contaminants that are already present in the concrete or the penetration of corrosive contaminants into the concrete (such as chlorides) that lead to corrosion of reinforcement or other embedments (8.4).

(b) Freezing-and-thawing damage if subject to saturation and a freezing-and-thawing environment. ASTM C666 may be used to define a durability factor. A durability factor of greater than 80 percent has generally been found to be acceptable in many locations for resistance to freezing and thawing for cementitious materials (refer to ACI 546.3R).

(c) Scaling if exposed to deicing salts.

(d) Exposure to ultraviolet or other radiation degradation within the repair environment unless other means are provided to address such degradation.

(e) Fatigue resulting from loading cycles and load reversal. For example, fatigue resistance may be needed in repair areas subject to many cycles of repeated loading.

(f) Impact, erosion, and vibration effects if exposed to conditions causing deterioration by these mechanisms.

(g) Abrasion resistance of repaired sections subject to heavy traffic, impingement of abrasive particles, or similar conditions.

(h) Chemical exposure may include sulfate attack, acids, alkalis, solvents, leaching of cementitious materials due to soft water, salt crystallization, and other factors that are known to attack or deteriorate the repair material or concrete substrate. Water penetration into concrete is associated with many types of chemical attack and other deterioration mechanisms.

(i) Thermal exposure and cycles.

(j) Alkali-aggregate reactions.

(k) Differential permeability between the repair and existing concrete if the repair material or the substrate concrete is vulnerable to deterioration due to trapped moisture, such as freezing-and-thawing damage of saturated concrete, corrosion of embedded steel reinforcement, alkali-aggregate reactions, or sulfate attack (refer to ACI 546.3R).

(l) The carbonation susceptibility, depth, and rate of both the existing concrete and the repair material in repairs containing reinforcement or other embedments requiring alkaline passivation of the metal for protection from corrosion (8.4).

8.2—Cover

8.2.1 The concrete cover requirements shall be in accordance with the design basis code. For alternative materials and methods, an equivalent cover that provides sufficient corrosion protection and fire protection shall be in accordance with 1.4.2. Sufficient anchorage and development for the reinforcement shall be provided regardless of methods used to provide corrosion protection.

8.2C—Cover

8.2.1C The code language is intended to allow equivalent cover such as anti-carbonation coatings, or intumescent coatings to be used if they can be demonstrated to be effective according to the approval detailed in 1.4.2. The licensed design professional should review anchorage and development requirements when alternative methods of corrosion protection are used.

CODE

8.2.2 Corrosion—Where concrete cover for existing reinforcement is insufficient to provide corrosion protection for the design service life of the structure, additional concrete cover or an alternate means of corrosion protection shall be provided to mitigate corrosion of reinforcement within the repair area.

8.3—Cracks

8.3.1 The design of repairs shall consider the effects of cracks on the expected durability, performance, and design service life of the repair.

8.3.2 The cause and repair of cracking shall be assessed and considered in repair design.

COMMENTARY

8.2.2C Alternate means of protecting reinforcement include the application of waterproof membranes, corrosion inhibitors, and various forms of cathodic protection. Existing reinforcement corrosion, chloride contamination, and carbonation should be considered when evaluating the maintenance requirements and design service life of alternative methods for corrosion protection. Ongoing metal corrosion may create distress and deterioration beyond the limit of the repair area. The licensed design professional should review design service life requirements when widespread durability issues are considered. Issues of concrete cover need to be addressed in a timely manner as distress becomes apparent.

8.3C—Cracks

8.3.1C Protection of repaired concrete may be as vital as the repair itself. Consideration should be given to post-repair cracking and the need for protection of the existing concrete and repair material from the ingress of deleterious materials. ACI 224.1R provides guidance for the prevention and control of cracks. Cracks in concrete structures may impact the long-term performance of the structure. The effect of the cracks on the long-term performance is generally related to the exposure conditions and the size, number, and location of cracks. When the cracked section is subject to aggressive conditions, the service life is reduced.

Cracks reduce the protection provided by the effective cover over steel reinforcement and lead to water and deleterious material ingress, which accelerates the deterioration of embedded reinforcement and can cause other concrete deterioration issues such as freezing-and-thawing deterioration, alkali-aggregate deterioration, and chemical attack.

Crack mitigation methods should consider the causes, movement, size, orientation, width, and complexity of the network of cracks. The characteristics of the substrate, location, and evidence of water transmission should be determined to assess the appropriate method of repair. Active water infiltration should be corrected as required for the durability of the structure.

ACI 224.1R provides additional guidance on the repair of cracks. ACI 503.7 provides a specification for repair of cracks through polymer injection. The comparative cracking potential of different cementitious materials can be evaluated using ASTM C1581.

8.3.2C Cracks in concrete structures can be detrimental to the long-term performance of a structure if the cracks are of sufficient size to allow for the ingress of deleterious materials into the structure. Not all cracks need to be repaired. Cracks above reasonable widths for given exposure conditions (refer to ACI 224R, Table 4.1, for reasonable cracks widths) may require repair or remediation.

There are a variety of different materials that have been used for crack repair. For cracks that are essentially acting as a joint or are active, one type of effective repair is to seal the crack with an elastomeric sealant. Materials used for crack

CODE

8.4—Corrosion and deterioration of reinforcement and metallic embedments

8.4.1 The corrosion and deterioration of reinforcement and embedded components shall be considered in the durability design. Repair materials shall not contain intentionally added constituents that are corrosive to reinforcement within the repair area. Aggregates shall conform to the requirements of ACI 318M-11.

8.4.2 When evaluating long-term durability and strength of a member, existing reinforcement corrosion that is encapsulated within new repair materials shall be considered.

8.4.3 The quality of existing concrete and its ability to protect reinforcement from corrosion and deterioration shall be considered.

COMMENTARY

injection include, for example, epoxy, polyurethane, latex in a cement matrix, microfine cement, and polymethacrylate. For repair by crack injection, the process and material should be appropriate to the site conditions. Crack injection should not be used to repair cracks caused by corrosion of steel reinforcement and alkali aggregate reaction unless supplemental means are employed to mitigate the cause of the cracks. ACI 224.1R provides guidance on the causes and investigation of cracking in reinforced concrete under service loads as well as the evaluation and repair of cracks. All cracks have the potential to become active cracks.

8.4C—Corrosion and deterioration of reinforcement and metallic embedments

8.4.1C Untreated reinforcement corrosion limits the life expectancy of repair areas, repair materials, and repaired structures. Repairs that do not address reinforcement corrosion should consider the design service life and monitoring.

8.4.2C ICRI No. 310.1R states that all damaged concrete and corrosion products are to be removed from reinforcement during repairs. Sufficient concrete should be removed to allow for the new repair materials to completely encapsulate the reinforcement. In some situations, due to congestion of steel reinforcement, access limitations, load considerations, or other factors, it is not possible to remove all corrosion products from the steel reinforcement. Situations exist where corroding reinforcement that cannot be adequately cleaned or repaired is encapsulated with new repair materials. The durability effects of encapsulating the existing steel reinforcement on the member strength should be considered in these situations.

8.4.3C Water and chemical penetration into the concrete can cause corrosion of metallic embedments and damage to nonmetallic reinforcement. Where concrete cover over reinforcement is insufficient to provide corrosion protection for the design service life of the structure, additional concrete cover or an alternate means of corrosion protection should be provided to mitigate reinforcement corrosion. The corrosion of embedded metals adjacent to the repair may be accelerated by the ring anode effect due to differing electrical potential between continuous reinforcement in the repair area and that external to the repair area, depending upon the relative humidity and chloride content. For guidance on alternate methods of corrosion prevention, mitigation, and inhibition, refer to ACI 546R and The Concrete Society Technical Report 50 (1997). Both carbonation and chloride contamination may require consideration and are discussed in ACI 546R.

The anodic ring effect that can be induced by certain repairs should be addressed by incorporating appropriate corrosion mitigation strategies such as cathodic protection, corrosion inhibitors, or other techniques. Consider-

CODE

COMMENTARY

8.4.4 Existing steel reinforcement and added reinforcement shall be protected from corrosion and deterioration to satisfy durability requirements.

8.4.5 Galvanic corrosion between electrochemically dissimilar materials shall be considered.

8.4.6 Corrosion protection of bonded and unbonded prestressing materials and prestressing system components shall be addressed during the repair design.

8.4.7 If electrochemical protection systems are used to protect steel reinforcement in repair areas and structures, the interaction of the protection system with the repaired elements, the entire structure, and environment shall be considered.

ations should include reduction of the phenomenon. ACI 546R, ACI 364.3T, and ACI RAP-8 provide discussion on anodic ring effect. ACI 222R, ACI 222.3R, ACI 364.3T, and FAQ sections from *Concrete International* (2002a,b,c) also provide guidance for corrosion mitigation.

Aesthetics may be affected by different means of protection and may also require consideration. Damage due to fire and fire protection requirements are discussed in 7.9.

8.4.4C Reinforcing steel in concrete construction is traditionally protected by concrete cover, which protects the steel reinforcement from deleterious materials and provides fire protection. The minimum cover thickness is typically required by ACI 318M-11. Adequate protection may be provided by increased section thickness and appropriate coatings such as sealers, intumescent coatings, electrochemical methods, or other techniques.

8.4.5C Reinforcement or metallic embedments in the repair area with differing electrochemical potentials, environments, or both, should be isolated from the existing reinforcement, or the existing reinforcement and metal embedments should be protected to minimize galvanic corrosion. For example, rail or post-pocket repairs can use dissimilar metals from conventional steel reinforcement, which could accelerate the deterioration of the installation (refer to ACI 222R).

8.4.6C Prestressed concrete structures are reinforced with stressed, high-strength steel. The prestressing steel can be either bonded or unbonded to the concrete. The presence of a prestressing force in the steel and the need to transfer the prestressing force into the concrete makes corrosion damage in prestressed concrete members more critical than traditionally reinforced structures (refer to ACI 423.4R).

The bonded or unbonded nature of the prestressing steel, the condition of the steel at the repair area, the attachment of the steel to the structure, the as-designed corrosion protection measures, the existing corrosion condition, and the continuity of the prestressing steel need to be considered to address corrosion protection of the structure. Refer to ICRI 320.4, and 222.2R.

Hydrodemolition and other types of material removal methods should be used cautiously when the structure contains unbonded prestressing steel reinforcement. In these situations, water can be introduced into the corrosion protection (sheathing) surrounding the steel (refer to ICRI No. 310.3), affecting the long-term durability of the prestressing steel reinforcement.

8.4.7C Structures using impressed current electrochemical protection or mitigation systems should have continuous reinforcement, separate zones, or provisions should be made to make the steel electrically continuous. Impressed current electrochemical protection systems should be designed and maintained so as to not promote an alkali-aggregate reaction (AAR) and to avoid embrittlement of prestressing steel.

CODE

8.4.8 Repair materials and reinforcement shall be selected to be compatible such that the characteristics of each material do not adversely affect the durability of the other materials or of the existing concrete and reinforcement.

8.5—Surface treatments and coatings

8.5.1 Moisture transmission through the structure and the influence of the surface treatment on the durability of the structure shall be considered.

8.5.2 The selection of surface treatments applied to concrete surfaces shall consider existing cracks within the concrete and the potential for movement on the repair system durability, the surface treatment, and the anticipated service life of the structure.

COMMENTARY

Impressed current electrochemical protection systems should include a monitoring and maintenance plan developed by a licensed design professional specializing in the design of corrosion protection systems (refer to NACE RP00290, NACE SP0390, NACE 01105, NACE 01102, NACE 01101, NACE 01104, and NACE SP0107).

8.4.8C Materials can degrade under certain exposure conditions with a negative impact on steel reinforcement or concrete materials. For example, in certain situations such as exposure to high temperatures, polyvinyl chloride (PVC) and other polymer-based materials can deteriorate, releasing decomposition products found to cause corrosion.

Even if the conventional steel reinforcement becomes more noble when in electrical contact with a dissimilar metal (for example, embedded aluminum conduit in the presence of chlorides), considerable concrete damage can arise (Monfore and Ost 1965).

Fiber-reinforced polymer (FRP) wrapping should not be used as a corrosion repair strategy on members experiencing corrosion of embedded reinforcement unless the concrete is repaired and corrosion mitigated. Appropriate sections within this document and referenced documents concerning FRP repairs should be consulted (refer to ACI 440.2R).

8.5C—Surface treatments and coatings

8.5.1C Surface treatments, coatings, sealers, or membranes are commonly used to limit the ingress of deleterious materials and moisture into the structure to reduce future deterioration of the structure. Surface treatments, coatings, sealers, and membranes may have a shorter service life than the concrete and can be considered as consumable or requiring periodic replacement or repair to maintain effective protection of the concrete.

In some situations, encapsulation of moisture and deleterious materials by a surface treatment has been found to cause or accelerate deterioration. The condition of the concrete should be appropriate to receive a specific surface treatment, coating, or membrane (ICRI No. 310.2).

8.5.2C Crack development and propagation provide an accelerated mechanism for ingress of moisture and deleterious materials and may also cause a surface treatment to become ineffective.

CODE

COMMENTARY

CHAPTER 9—CONSTRUCTION

9.1—Stability and temporary shoring requirements

9.1.1 Contract documents for repair shall define the temporary shoring and bracing requirements for all phases of the repair project. Temporary shoring and bracing shall be designed by a licensed design professional. Adequate temporary shoring and bracing of affected members shall be provided during evaluation and repair construction. Temporary shoring shall be designed to accommodate in-place conditions in the structure and expected superimposed loads and shall consider the effects of deformation compatibility on the shoring system with the supported and supporting structural elements.

Temporary shoring and bracing installation details shall be reviewed by the licensed design professional for the repair to assess the impact of the shoring on the existing structure.

9.1.2 Global structural stability and the stability of individual members shall be maintained prior to and during all stages of the repair process.

9.1.3 Existing member stability, including the effect of modifications or repairs to existing lateral bracing members, shall be evaluated and maintained at all times.

Temporary bracing shall be provided as necessary to maintain the stability of compression members. The lateral forces for temporary bracing design shall be determined by generally accepted engineering principles or as required by the general building code. Temporary shoring and bracing shall be designed to provide sufficient stiffness to prevent excessive displacement of the existing braced members as determined by the licensed design professional.

9.1.4 The design of shoring and bracing members during the repair or removal of existing adjacent framing shall consider the changes in load paths and unbraced lengths, and the redistribution of loads and internal forces that result in changes in existing applied loads on structural elements. Redistribution of loading due to deformation of members shall be considered in the design of temporary shoring and bracing.

9.1C—Stability and temporary shoring requirements

9.1.1C Temporary shoring and bracing members should be designed to consider changes in bracing and shoring conditions during repair construction and as required to support construction operations. Design of temporary shoring members should be based on the in-place loads on the structure. Loads, such as snow, seismic, wind, and construction live loads, should also be considered in shoring design (ASCE/SEI 37). Design requirements for shoring are also contained in ASCE/SEI 37. Shoring design guidelines are contained in AISC Steel Design Guide Series 10 (Fisher and West 2003). Temporary shoring and bracing installation is generally performed by the contractor. Load, spacing, and placement requirements for temporary shoring and bracing at all phases of a repair should be conveyed to the contractor.

All temporary shoring and bracing design and installation details should be reviewed by the licensed design professional for the repair to assess the impact of the shoring on the existing structure, and to verify conformance of the proposed shoring with any project-specific requirements. Refer to 5.1.4 for load requirements associated with shoring and temporary construction.

9.1.2C The assessment of structural stability includes the overall existing structure, existing members affected by repair, and temporary bracing elements that contribute to overall stability. Stability of these elements should be considered during all phases of the repair process. Temporary measures may be needed to provide bracing and shoring of affected members. If necessary, requirements to preload temporary members should be included in the repair drawings.

9.1.3C Supplemental bracing for compression members may be required if the unbraced length of a compression member is modified during the repair process. Compression members include columns, walls, and other members, such as diaphragms, that resist compressive loads. The design of bracing members is described in various publications (AISC 2006; ANSI/AF&PA NDS 2005). The design load for a bracing member should be based on the existing dead and live loads, construction loads, and other loads that may be resisted by the compression member. A lateral force of 2 percent of the axial load in the member being braced is commonly used as a minimum load in the design of bracing members (ANSI/AISC 360-10).

9.1.4C Removal of existing column, beam, wall, and floor slab elements or parts thereof during repair construction and the placement of shoring and bracing may result in the redistribution of loads and internal forces within the building structure. The removal of existing framing members, diaphragms, or slabs can also affect the unbraced length of the existing framing members in the removal area. Effects of the removal of elements should be considered in assessing the existing structure and shoring and bracing design.

CODE

9.1.5 Where existing structural members support the structure and loads generated during the evaluation and repair construction, the design strength of those members shall be evaluated and shall exceed the temporary required strength due to construction loads. This evaluation shall include the effects of deterioration due to section loss and material degradation. If the strength and stiffness of any member is less than required, shoring shall be provided and remain in place until the existing member is repaired.

9.2—Temporary conditions

9.2.1 Load and load factors during the evaluation and repair construction processes shall be in accordance with 5.1.4. The design strength of the structure or member shall exceed the calculated required strength. The construction loads in 5.1.4 shall only be used during ongoing evaluation and repair process.

9.3—Environmental issues

9.3.1 Contract documents prepared by the licensed design professional shall instruct the contractor or other designated party to be responsible for the implementation of all specified environmental remediation measures, to report any new conditions encountered, and for the control of all construction debris, including environmentally hazardous materials.

COMMENTARY

9.1.5C The design of shoring and bracing members and the assessment of existing members should be based on the existing cross sections present at the time of repair implementation.

9.2C—Temporary conditions

9.2.1C During the evaluation and repair process, a temporary reduction in design load is allowed, except when specifically prohibited by jurisdictional authorities or local building codes. The reduction in the design load intensity should be determined based on the in-place condition of the structure and the time required for the completion of stabilization measures, or repairs using sound engineering principles. The licensed design professional is required to verify that the design strength of the existing structure is satisfactory at all times. The licensed design professional should also consider the expected project duration when using temporary construction loads. If a change in the length of the project or a delay occurs, the reduced design loads may no longer be appropriate.

9.3C—Environmental issues

9.3.1C Evaluation and repair of an existing structure can result in the exposure of workers and the public to potentially hazardous materials. These materials may be exposed, dislodged, carried into the air, or discharged as effluent into surface drainage during the evaluation and repair process. The owner should have an environmental assessment performed during the evaluation and repair process in the areas to be repaired before any work to identify hazardous materials with the potential to present health issues to the workers and public, unless the owner can attest that the existing structure is free of hazardous materials.

During the repair process, the contractor normally is responsible for the implementation of repairs and, accordingly, the control of construction debris, dust, and other materials. Any new conditions uncovered during the repair process should be reported to the owner and licensed design professional.

CODE

COMMENTARY

CHAPTER 10—QUALITY ASSURANCE

10.1—Inspection

10.1.1 Concrete repair and rehabilitation construction shall be inspected as required by the general existing building code or as required by the local jurisdiction.

10.1.2 The licensed design professional shall include in the contract documents testing and inspection requirements applicable to the project.

10.1C—Inspection

10.1.1C In the absence of such inspection requirements, the licensed design professional should recommend to the owner that concrete repair and rehabilitation construction should be inspected during the various work stages by a licensed design professional, a qualified inspector, or a qualified individual.

10.1.2C Required testing and inspections may include (a) through (j):

- (a) Delivery, placement, and testing reports documenting the quantity, location of placement, repair materials tests, and other tests as required;
- (b) Construction and removal of forms and reshoring;
- (c) Concrete removal and surface preparation;
- (d) Placing of reinforcement and anchors;
- (e) Mixing, placing, and curing of repair materials;
- (f) Sequence of erection and connection of new members;
- (g) Tensioning of tendons;
- (h) Construction loads on floors, members, or walls;
- (i) General progress of work;
- (j) Installation and testing of post-installed anchors.

Inspection and test results should be submitted to licensed design professional and owner.

Repair construction should be checked for quality of materials and workmanship, and for compliance with the intent of the repair construction documents. Inspection should be provided by either repair inspectors or the licensed design professional. Repair inspectors should be qualified by demonstrating competence to the satisfaction of the licensed design professional and as required by the building official, to inspect concrete repair construction. If qualified, the licensed design professional may provide inspection services.

Inspection of concrete repair construction requires review of the work in the field, review of construction documents, comparison of the work with contract documents, documentation and report of the work inspected as conforming or nonconforming, and of corrections to the work. Visual inspection and verification of existing conditions may require review of specific conditions before continuing the construction process and thus require pauses in the construction processes so as not to conceal components of the work before completing necessary inspections and verifications. Inspection of post-installed anchor installation and testing should be performed as required by the construction documents and in accordance with 7.6.5.

Part of all repair inspections is determining compliance with the intent of the contract documents, documenting the inspection, and reporting the inspection. If the inspection shows conformance with the contract documents and no corrections are necessary, then the inspected work should be documented as conforming and reported to the licensed design professional and contractor, noting no corrections. If the inspection shows readily correctable issues and the

CODE**COMMENTARY**

10.1.3 The licensed design professional shall require that existing conditions and reinforcement shall not be concealed with materials that obscure visual inspection before completion of inspection unless the licensed design professional determines that it is only necessary to inspect representative locations rather than every repair location.

10.2—Testing of repair materials

10.2.1 Repair material tests and test frequencies shall be specified by the licensed design professional in the contract documents. Results of tests shall be reported as required by the contract documents and the general existing building code. Test records shall be retained by the testing agency as required by the general building code. In the absence of a general building code, the licensed design professional shall

issues are corrected by the contractor, then the inspected work should be documented as conforming and reported to the licensed design professional and contractor with corrections noted and verified as completed. Nonconforming or deficient components, processes, and procedures (repair not passing inspection) should be reported to the licensed design professional for review and actions should be made to correct the process prior to resuming the repair construction and inspection process. Nonconforming repair construction may include:

- (a) Existing construction that differs from the repair documents
- (b) Existing construction deterioration, distress, or levels of distress beyond those anticipated in the design of repairs
- (c) Deficiencies in repair components
- (d) Deficiencies in construction processes and procedures

Material data sheets are to satisfy the requirement of specific repair material used in construction. The manufacturing date and shelf life of the repair material provide information that the material is within the manufacturer's recommended time limits for installation.

Existing conditions describe the nature and extent of damage and size and condition of the members. Those conditions need to be verified for conformance to the design assumptions. The following are some items where inspections are beneficial:

- (a) Location of repairs
- (b) Surface preparation of existing concrete and existing reinforcement
- (c) Placement of reinforcement and anchors
- (d) Specific materials used in the repairs
- (e) Delivery, placement, and testing reports documenting the quantity and location of placement, repair material tests, strength, and other tests of all repair materials
- (f) Construction and removal of forms and shoring
- (g) Mixing, placing, and curing of repair materials
- (h) Sequence of repair construction
- (i) Tensioning of tendons
- (j) Construction loads
- (k) General progresses of the repair work

10.1.3C Reinforcement includes existing reinforcement, embedded items such as anodes, new reinforcement, and anchors. Removal of deteriorated concrete and reinforcement often uncovers defects not anticipated. The licensed design professional should inspect these uncovered conditions before placement of new repair materials.

10.2C—Testing of repair materials

10.2.1C Tests of repair materials should comply with testing and test frequency of new concrete construction, unless otherwise directed by the licensed design professional in the contract documents and approved by the building code official. It is generally not practical to verify all manufacturers' listed properties of proprietary materials, such as shrinkage, thermal expansion coefficient, and modulus of elasticity. In

CODE

require that the test records be retained by the testing agency for a minimum of 3 years beyond completion of construction.

10.3—Construction observations

10.3.1 Construction observation shall be performed as required by the contract documents.

COMMENTARY

such cases, the licensed design professional should rely on manufacturer product data. The licensed design professional should evaluate the data and, if necessary, have manufacturers perform testing to confirm that their material achieves the published values that they provided for the project. Refer to ACI 546.3R and ICRI 320.2R for guidance. Tests of repair materials' bond to existing materials should comply with requirements of the contract documents.

Testing personnel should be qualified by demonstrating competence to the satisfaction of the licensed design professional and building code official for testing types required of concrete repair and rehabilitation work. The testing agency should maintain a record of the tests performed and the results consistent with the requirements for records of ASTM E329.

10.3C—Construction observations

10.3.1C A primary purpose of construction observation of rehabilitation work is to verify that the exposed existing construction is as assumed in the design and that the contract documents will fulfill the design intent. If the existing construction differs from the design assumptions, requiring modification of the design, changes should be documented and the work modified as necessary. The licensed design professional should report design changes in writing to the owner, rehabilitation inspector, contractor, and building code official resulting from existing construction, nonconforming rehabilitation work, and observed construction deficiencies. Revised design or construction work necessary to correct these deficiencies, and the construction corrections, should be observed.

CODE

COMMENTARY

CHAPTER 11—COMMENTARY REFERENCES*American Concrete Institute*

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

ACI 201.2R-08—Guide to Durable Concrete

ACI 209R-92—Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures

ACI 209.1R-05—Report on Factors Affecting Shrinkage and Creep of Hardened Concrete

ACI 214.4R-10—Guide for Obtaining Cores and Interpreting Compressive Strength Results

ACI 216.1M-07—Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies

ACI 222R-01—Protection of Metals in Concrete against Corrosion

ACI 222.2R-01—Corrosion of Prestressing Steels

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

ACI 224R-01—Control of Cracking in Concrete Structures

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

ACI 228.1R-03—In-Place Methods to Estimate Concrete Strength

ACI 228.2R-98—Nondestructive Test Methods for Evaluation of Concrete in Structures

ACI 301M-10—Specifications for Structural Concrete

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

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

ACI 355.4-11—Qualification of Post-Installed Adhesive Anchors in Concrete and Commentary

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

ACI 364.3T-10—Treatment of Exposed Epoxy-Coated Reinforcement in Repair

ACI 365.1R-00—Service-Life Prediction

ACI 369R-11—Guide for Seismic Rehabilitation of Existing Concrete Frame Buildings and Commentary

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

ACI 437-13—Code Requirements for Load Testing of Existing Concrete Structures and Commentary

ACI 437R-03—Strength Evaluation of Existing Concrete Buildings

ACI 437.1R-07—Load Tests of Concrete Structures: Methods, Magnitude, Protocols, and Acceptance Criteria

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

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

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

ACI 440.4R-04—Prestressing Concrete Structures with FRP Tendons

CODE

COMMENTARY

ACI 503R-93—Use of Epoxy Compounds with Concrete
 ACI 503.5R-92—Guide for the Selection of Polymer Adhesives in Concrete

ACI 503.6R-97—Guide for Application of Epoxy and Latex Adhesives for Bonding Freshly Mixed and Hardened Concrete

ACI 503.7-07—Specification for Crack Repair by Epoxy Injection

ACI 546R-04—Concrete Repair Guide

ACI 546.3R-06—Guide for the Selection of Materials for the Repair of Concrete

RAP-8—Installation of Embedded Galvanic Anodes

American Institute of Steel Construction

AISC 325-11—Steel Construction Manual

AISC 2006—2006 Standard for Steel Building Structures

ANSI/AISC 360-10—Specification for Structural Steel Buildings

American Society of Civil Engineers

ASCE/SEI 7-10—Minimum Design Loads for Buildings and Other Structures

ASCE/SEI 11-99—Guideline for Structural Condition Assessment of Existing Buildings

ASCE/SEI 31-03—Seismic Evaluation of Existing Buildings

ASCE/SEI 37-02—Design Loads on Structures during Construction

ASCE/SEI 41-06—Seismic Rehabilitation of Existing Buildings

ASCE/SEI/SFPE 29-05—Standard Calculation Methods for Structural Fire Protection

American Forest and Paper Association

ANSI/AF&PA NDS-2005—National Design Specification (NDS) for Wood Construction

Applied Technology Council

ATC 20-89—Procedures for Post-Earthquake Safety Evaluation of Buildings

ATC 45-04—Field Manual: Safety Evaluation of Buildings after Windstorms and Floods

ASTM International

ASTM C666/C666M-03(2008)—Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing

ASTM C1581/C1581M-09—Standard Test Method for Determining Age at Cracking and Induced Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage

ASTM C1583/C1583M-04e1—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) (withdrawn 2013)

ASTM D610-08—Standard Practice for Evaluating Degree of Rusting on Painted Steel Surfaces

CODE**COMMENTARY**

ASTM D4065-06—Standard Practice for Plastics: Dynamic Mechanical Properties: Determination and Report of Procedures

ASTM E84-12c—Standard Test Method for Surface Burning Characteristics of Building Materials

ASTM E119-12a—Standard Test Methods for Fire Tests of Building Construction and Materials

Canadian Standards Association

AC308-11—Acceptance Criteria for Post-Installed Adhesive Anchors in Concrete Elements

CAN/CSA S6-06—Canadian Highway Bridge Design Code and Commentary

International Code Council

IBC-2012—International Building Code

IEBC-2012—International Existing Building Code

International Concrete Repair Institute

ICRI No. 210.2-02—Guideline for the Evaluation of Unbonded Post-Tensioned Concrete Structures

ICRI No. 210.3-04—Guide for Using In-Situ Tensile Pull-Off Tests to Evaluate Bond of Concrete Surface Materials

ICRI No. 310.1R-08—Guide for Surface Preparation for the Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion

ICRI No. 310.2-97—Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, and Polymer Overlays

ICRI No. 310.3-04—Guide for the Preparation of Concrete Surfaces for Repair Using Hydrodemolition Methods

ICRI No. 320.2R-09—Guide for Selecting and Specifying Materials for Repair of Concrete Surfaces

ICRI No. 320.3-04—Guideline for Inorganic Repair Material Data Sheet Protocol

ICRI No. 320.4-06—Guideline for the Repair of Unbonded Post-Tensioned Concrete Structures

ICRI No. 340.1-06—Guideline for the Selection of Grouts to Control Leakage in Concrete Structures

NACE International

NACE 01101—Electrochemical Chloride Extraction from Steel-Reinforced Concrete—A State-of-the-Art Report

NACE 01102-02—State-of-the-Art Report: Criteria for Cathodic Protection of Prestressed Concrete Structures

NACE 01104 Electrochemical Realkalization of Steel-Reinforced Concrete—A State-of-the-Art Report

NACE 01105-05—Sacrificial Cathodic Protection of Reinforced Concrete Elements—A State-of-the-Art Report

NACE RP0290-00—Standard Recommended Practice—Cathodic Protection of Reinforcing Steel in Atmospherically Exposed Concrete Structures

NACE SP0107-07—Electrochemical Realkalization and Chloride Extraction for Reinforced Concrete

NACE SP0390-09 (formerly RP0390)—Maintenance and Rehabilitation Considerations for Corrosion Control

CODE

COMMENTARY

of Atmospherically Exposed Existing Steel-Reinforced Concrete Structures

ACI Committee 315, 2004, *ACI Detailing Manual*, SP-66, American Concrete Institute, Farmington Hills, MI, 212 pp.

Bartlett, F. M., and MacGregor, J. G., 1995, "Equivalent Specified Concrete Strength from Core Test Data," *Concrete International*, V. 17, No. 3, Mar., pp. 52-58.

Concrete International, 2002a, "FAQ," V. 24, No. 3, Mar., p. 82.

Concrete International, 2002b, "FAQ," V. 24, No. 6, June, p. 90.

Concrete International, 2002c, "FAQ," V. 24, No. 7, July, p. 91.

Concrete Reinforcing Steel Institute (CRSI), 2001, "Evaluation of Reinforcing Bars in Old Reinforced Concrete Structures," *Engineering Data Report* No. 48, Concrete Reinforcing Steel Institute, Schaumburg, IL.

Fisher, J., and West, M., 2003, "Erection Bracing of Low-Rise Structural Steel Buildings," *Steel Design Guide Series 10*, American Institute of Steel Construction, Milwaukee, WI, 83 pp.

Harris, J. R., and Stevens, G. R., 1991, "Detailing Reinforced Concrete Structures for Post-Elastic Seismic Resistance," *Seismic Design of Reinforced Concrete Structures Inelastic Response and Design*, SP-127, American Concrete Institute, Farmington Hills, MI, pp. 539-554.

IBC, 2012, "International Building Code," International Code Council (ICC), Washington, DC, 666 pp.

IEBC, 2012, "International Existing Building Code," International Code Council (ICC), Washington, DC, 294 pp.

Kahn, L. F., 1980, "Strengthening of Existing RC Columns for Earthquake Resistance," *Seventh World Conference on Earthquake Engineering*, Istanbul, Turkey.

Monfore, G. E., and Ost, B., 1965, "Corrosion of Aluminum Conduit in Concrete," *Journal of the PCA Research and Development Laboratories*, V. 7, No. 1, pp. 10-22.

Murray, T. M.; Allen, D. E.; and Ungar, E. E., 1999, "Floor Vibrations Due to Human Activity," *Steel Design Guide Series 11*, American Institute of Steel Construction, Chicago, IL.

NFPA 5000, 2009, "Building Construction and Safety Code," National Fire Prevention Association, Quincy, MA.

PCA, 1994, "Assessing the Condition and Repair Alternatives of Fire-Exposed Concrete and Masonry Members," Model Code SR322, PCA, Skokie, IL, 15 pp.

Portland Cement Association (PCA), 1985, "Analytical Methods of Determining Fire Endurance of Concrete and Masonry Members—Model Code Approved Procedures," Model Code SR267, PCA, Skokie, IL.

PTI, 2006, *Post Tensioning Manual*, sixth edition, Post-Tensioning Institute, Farmington Hills, MI, 354 pp.

PCI, 2010, *PCI Design Handbook*, seventh edition, Precast/Prestressed Concrete Institute, Chicago, IL.

CODE

COMMENTARY

Priestley, M. J. N.; Seible, F.; and Calvi, G. M., 1996, *Seismic Design and Retrofit of Bridges*, John Wiley & Sons, pp. 686.

PTI, 2010, "Guide for Creating Openings and Penetrations in Existing Slabs with Unbonded Post-Tensioning (PTI DC80.2)," Post Tensioning Institute, Farmington Hills, MI, 16 pp.

Ruddy, J. L.; Marlo, J. P.; Loanneides, S. A.; and Alfawakhiri, F., 2003, "Design Guide 19: Fire Resistance of Structural Steel Framing," American Institute of Steel Construction, Chicago, IL.

Sabnis, G. M.; Harris, H. G.; White, R. N.; and Mirza, M. S., 1983, *Structural Modeling and Experimental Techniques*, Prentice-Hall Inc., Englewood Cliffs, NJ.

Scollard, C. R., and Bartlett, F. M., 2004, "Rehabilitation Criteria for Post-Tensioned Voided-slab Bridges," *Canadian Journal of Civil Engineering*, V. 31, No. 6, pp. 977-987.

Swami, R. N., and Gaul, R., eds., 1996, *Repair and Strengthening with Adhesive Bonded Plates*, SP-165, American Concrete Institute, Farmington Hills, MI, Jan., 264 pp.

The Concrete Society, 1997, "Guide to Surface Treatments for Protection and Enhancement of Concrete," *Technical Report 50*, The Concrete Society, Camberley, UK, 80 pp.

The Concrete Society, 2008, "Assessment, Design and Repair of Fire-Damaged Concrete Structures," *Technical Report 68*, The Concrete Society, Camberly, UK.

Tuutti, K., 1980, "Service Life of Structures with Regard to Corrosion of Embedded Steel," *Performance of Concrete in Marine Environment*, SP-65, American Concrete Institute, Farmington Hills, MI, pp. 223-236.

U.S. Department of the Interior, 1995, "The Secretary of the Interior Standards for the Treatment of Historic Properties with Guidelines for Preserving, Rehabilitating, Restoring, and Reconstructing Historic Buildings," National Park Service, Department of the Interior, Washington, DC. (<http://www.nps.gov/hps/tps/standguide/index.htm>) (accessed February 5, 2013).



American Concrete Institute®
Advancing concrete knowledge

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 and the ACI 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
U.S.A.

Phone: 248-848-3700
Fax: 248-848-3701

www.concrete.org

Code Requirements for Evaluation, Repair, and Rehabilitation of Concrete Buildings and Commentary

The AMERICAN CONCRETE INSTITUTE

was founded in 1904 as a nonprofit membership organization dedicated to public service and representing the user interest in the field of concrete. ACI gathers and distributes information on the improvement of design, construction and maintenance of concrete products and structures. The work of ACI is conducted by individual ACI members and through volunteer committees composed of both members and non-members.

The committees, as well as ACI as a whole, operate under a consensus format, which assures all participants the right to have their views considered. Committee activities include the development of building codes and specifications; analysis of research and development results; presentation of construction and repair techniques; and education.

Individuals interested in the activities of ACI are encouraged to become a member. There are no educational or employment requirements. ACI's membership is composed of engineers, architects, scientists, contractors, educators, and representatives from a variety of companies and organizations.

Members are encouraged to participate in committee activities that relate to their specific areas of interest. For more information, contact ACI.

www.concrete.org



American Concrete Institute®
Advancing concrete knowledge