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An ACI Standard

Assessment, Repair, and Rehabilitation of Existing Concrete Structures—Code and Commentary

Reported by ACI Committee 562

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Assessment, Repair, and Rehabilitation of Existing Concrete Structures—Code and Commentary

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An ACI Standard

Reported by ACI Committee 562

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ACI CODE-562-21, "Assessment, Repair and Rehabilitation of Existing Concrete Structures—Code Requirements and Commentary," was developed to provide design professionals a code for the assessment of the damage and deterioration, and the design of appropriate repair and rehabilitation strategies. The Code provides minimum requirements for assessment, repair, and rehabilitation of existing structural concrete buildings, members, systems and, where applicable, nonbuilding structures. ACI 562-19 was specifically developed to work with the International Existing Building Code (IEBC) or to be adopted as a stand-alone code.

Keywords: assessment; bond; corrosion; damage; durability; evaluation; existing structure; fiber-reinforced polymer (FRP); interface bond; licensed design professional; maintenance; rehabilitation; reliability; repair; strengthening.

CONTENTS

ACI 562-21 supersedes ACI 562-19, was adopted November 10, 2021, and was published November 2021.

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PREFACE

This code provides minimum requirements for assessment, repair, and rehabilitation of existing concrete structures, members, and systems. This code was developed by an ANSI-approved consensus process. This code can supplement the International Existing Building Code (IEBC), supplement the code governing existing structures of an authority having jurisdiction, or act as a stand-alone code in a locality that has not adopted an existing building code.

The Code is specifically written for use by a licensed design professional. This code provides minimum requirements for assessment, design and construction, or implementation of repairs and rehabilitation, including quality assurance requirements, for structural concrete in service. This code has no legal status unless it is adopted by the authority having jurisdiction. Where the code has not been adopted, it serves as a standard to provide minimum requirements for assessment, design, and construction for the repair and rehabilitation of existing structural concrete. ACI 318 provides minimum requirements for the materials, design, and detailing of structural concrete buildings and, where applicable, nonbuilding structures, and for new construction within existing structures were noted herein.

Key changes from ACI 562-19 to ACI 562-21 include:

(a) The revised code, 562-21, to be used with any existing structures code (not just IEBC).

(b) Chapters 1 and 4 have been combined. Chapter 4 was reduced to meet the goal in (a).

(c) The content of Appendix A was revised and moved into the body of ACI 562, Chapter 4.

(d) The deletion of Appendix A.

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CODE

CHAPTER 1—GENERAL REQUIREMENTS

1.1—Scope

This Code shall apply to assessment, repair, and rehabilitation of existing concrete structures as:

1. A code supplementing an existing building code, or
2. A stand-alone code for existing concrete structures when an existing building code is not adopted.

1.2—General

1.2.1 ACI 562, "Code Requirements for Assessment, Repair, and Rehabilitation of Existing Concrete Structures," is hereafter referred to as "this Code."

1.2.2 The *licensed design professional* is responsible for the assessment or repair and rehabilitation design, or both.

1.2.3 The requirements of this Code use strength design provisions for demands and capacities, unless otherwise noted.

1.3—Purpose

1.3.1 The purpose of this Code is to safeguard the public by providing minimum requirements for assessment, repair, and rehabilitation of existing concrete structures.

1.4—Applicability of this Code

1.4.1 This Code provides minimum requirements for assessment, repair, and rehabilitation of structural concrete components in existing structures, including buildings and non-building structures.

COMMENTARY

R1—GENERAL REQUIREMENTS

R1.1 This Code provides assessment, design, construction, and durability requirements for repair and rehabilitation of existing concrete structures. Throughout this Code, the term "structure" means an existing building, non-building structure, member, system, or element, if the construction is concrete or mixed construction with concrete and other materials.

This Code can be used in combination with an existing building code adopted by an authority having jurisdiction. For buildings, this is anticipated to be the state or local adoption of the International Existing Building Code (IEBC) developed by the International Code Council (ICC). Other codes may be applicable to non-building structures, or to structures that are not addressed in the IEBC. The provisions of this Code are intended to be used with the IEBC and similar codes.

If an existing building code is not adopted, Chapter 4 provides requirements for use as a stand-alone code.

R1.2—General

R1.2.3 If the existing building code or this Code permits the original building code to be used and that code uses allowable stress design, the licensed design professional should consider using the strength design provisions of this Code as a check in the evaluation of existing structures originally designed with allowable stress methods. Allowable stress design methods can result in designs that have inconsistent levels of structural reliability compared with modern strength design provisions (MacGregor 1974, Ellingwood et al. 1980).

R1.4—Applicability of this code

R1.4.1 This Code focuses on buildings and non-building structures as addressed by building codes or an authority having jurisdiction.

For buildings or structures similar to buildings, members that are addressed by this Code include but are not limited to foundations, soil-supported slabs, concrete portions of composite members, and precast and prestressed concrete.

In typical U.S. practice, owners are required to maintain existing structures to prevent unsafe conditions from occur-

ring or repair an existing structure if unsafe conditions are present. The minimum level of repair for an existing structure will typically address these unsafe or potentially unsafe conditions.

The licensed design professional can perform assessment, design, and quality assurance activities that exceed the minimum requirements of this Code. Requirements beyond the minimum stated in this Code, such as those for long-term durability, disproportionate collapse resistance, redundancy, or integrity can be considered. Exceeding the code minimum requirements is not a violation of this Code.

The Owner and the licensed design professional should agree on the intent of the repair and rehabilitation program and desired outcome at the onset of the project. The desired outcome may include consideration of the design service life of the repairs, structural reliability, serviceability considerations, and other factors. Due to the uncertain remaining service life of existing structures and the technical requirements of repair construction, quality assurance and construction observation in excess of that required by the general building code is commonly needed.

1.4.2 This Code does not apply to the repair of non-structural concrete or to aesthetic improvements, except if failure of such repairs would result in a dangerous condition.

R1.4.2 If nonstructural concrete requires repair, that repair is not required to comply with the requirements of this Code. The licensed design professional designing repairs to nonstructural concrete should consider the consequence of repair failure to determine if provisions of this Code are applicable.

1.4.3 The term "existing building code" refers to the code adopted by the authority having jurisdiction that regulates existing buildings or structures.

R1.4.3 The code commonly governing existing buildings in the United States is the IEBC, which provides limits on the extent of damage to be repaired using the original building code.

1.4.4 The term "original building code" refers to the general building code adopted by the authority having jurisdiction at the time the existing structure was permitted for construction.

R1.4.4 This description of "original building code" is consistent with the IEBC. In assessing existing structures, the licensed design professional may need to consider changes in the codes adopted by the authority having jurisdiction for the structure from the time of the original design through the time of the completion of construction. For buildings with major alterations or additions, the original building code should refer to the code in effect when the subject portion of the building was permitted, and different portions of a building may have different original building codes.

1.4.5 The term "current building code" refers to the general building code adopted by the authority having jurisdiction that regulates new building design and construction.

R1.4.5 The current building code establishes the design and construction regulations for new construction. Strength design regulations of the current building code typically include:

(a) Required strengths calculated using combinations of factored loads (strength-design demands)

(b) Design strengths (capacities) based on testing of materials, members, and systems

(c) Analytical methods used to calculate member and system capacity

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CODE | COMMENTARY

(d) Strength reduction factors that have been established to be consistent with reliability indices used with the strength-design demands

The load factors and strength reduction factors in the current building code were obtained through design code calibration procedures to achieve the targeted reliability indices that produce historically acceptable structural safety for new structures. The targeted reliability indices are generally based on past structural behavior, engineering experiences, cost and consequences of failure, and other factors. The resulting demand-to-capacity ratios for new structures provide the limits that are not to be exceeded in designing new construction, but these demand-to-capacity ratio limits need not to be the same as those for existing structures as noted in 4.5.3.

The general building code in the United States is typically based on the International Building Code (IBC) published by the ICC. Prior to 2015, Chapter 34 of the IBC included provisions for existing structures. For the design and construction of new concrete structures, the IBC and most other older general building codes often reference ACI 318, Building Code Requirements for Structural Concrete and Commentary, with exceptions and additions.

1.5—Administration

1.5.1 If provisions in this Code conflict with the regulations governing existing structures of the authority having jurisdiction, the regulations of the authority having jurisdiction shall govern.

1.5.2 If provisions in this Code conflict with requirements of standards referenced within this Code, this Code shall govern.

1.5.3 Alternative materials, design, and methods of construction shall be permitted in accordance with the existing building code or by the authority having jurisdiction.

R1.5—Administration

R1.5.3 New methods of design, new materials, and new uses of materials for repair and rehabilitation usually undergo a period of development before being permitted by a code.

Provision 1.5.3 mirrors IEBC 104.11 that permits building official approval of alternative methods, design, or materials on a project-by-project basis, provided that the alternative is demonstrated to the satisfaction of the building official to provide equivalent quality, strength, effectiveness, fire resistance, durability, and safety.

For systems considered under 1.5.3, specific tests, factored load combinations, strength reduction factors, deflection limits, and other pertinent requirements should be set by the authority having jurisdiction and should be consistent with the intent of this Code. Provision 1.5.3 does not apply to scale model testing used to supplement calculations or to strength evaluation of existing structures.

If the existing building code adopted by the authority having jurisdiction includes provisions for approval of alternative materials in new construction, the same process may be used for materials in repair applications, provided the materials conform to the provisions of this Code. Typically, the approval process requires the evaluation to be completed

CODE

COMMENTARY

by an approved agency, and the material properties and use requirements be summarized in a written evaluation report. This process is intended to allow for use of new materials and new classes of materials that do not have approved design standards or material specifications.

1.6—Design and construction documents

1.6.1 The basis of design shall be documented by the licensed design professional. Documentation shall identify the work area and scope of repair and rehabilitation.

R1.6—Design and construction documents

R1.6.1 The basis of design provides a summary of the assessment of the existing structure and a summary of the construction documents from original construction or prior rehabilitation used in developing the basis of design. The basis of design can be documented in a written report or included in construction documents. Information on some structures may be unavailable or unnecessary if strengthening is not required and should be so documented in the basis of design. The licensed design professional should review requirements of the authority having jurisdiction to determine the information to include in the basis of design documentation and filing requirements for the basis of design. Items that may be documented in the basis of design include:

(a) Detailed description of the structure, including age of construction, structural systems, identified original building code, and past and current uses

(b) Documentation of potentially dangerous structural conditions in the work area of the structure determined in the assessment

(c) Documentation of substantial structural damage in the work area

(d) Members and systems of the work area requiring increase in capacity beyond the demands of the original building code

(e) Conditions and details of the proposed rehabilitation work

(f) Assessment criteria and findings

(g) Design-basis code criteria and basis of rehabilitation design

(h) Shoring requirements such as loads to be resisted and spacing of shoring members

(i) Quality assurance and quality control (QA/QC) requirements

(j) Types and frequency of future inspection

(k) Types and frequency of future maintenance

(l) Recommendations to address serviceability conditions

A maintenance protocol should be provided in the basis of design, or in as-built or close-out documents. A maintenance protocol that addresses project-specific conditions provides the most effective method to ensure durability and should be established as part of the repair and rehabilitation design. The protocol should include required inspections and intervals between inspections, after completion of the repair or rehabilitation. 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). Recommendations should be provided to the owner

CODE | COMMENTARY

on inspection and maintenance to be undertaken during the remaining design service life of the repair material or the repaired part of the structure.

1.6.2 The construction documents for repair and rehabilitation shall convey the location, nature, and extent of the work, and the necessary information to perform the work in conformance with the requirements of this Code and the authority having jurisdiction. Construction documents shall require that materials used for the work conform to this Code and governing regulatory requirements at the time the work is executed.

R1.6.2 As applicable, the construction documents may consist of drawings and specifications if needed, and should indicate:

(a) Name and date of issue of the building code and supplements to which the assessment, repairs, or rehabilitation conforms

(b) Design assumptions and repair requirements including specified properties of existing and remedial materials used for the project and the strength requirements at stated ages or stages of construction

(c) Details, locations, and notes indicating repair types, materials, preparation requirements, and other pertinent information to implement the repairs, strengthening, or rehabilitation of the structure

(d) Magnitude and location of prestressing forces

(e) Anchorage details for prestressing reinforcement

(f) Reinforcement embedment, development, and lap splice lengths

(g) Type and location of mechanical or welded splices of reinforcement

(h) Shoring or bracing criteria required before, during, and at completion of the assessment, repair, or rehabilitation

(i) Quality assurance program including specific inspections, testing requirements, and acceptance criteria

ACI 563 is a reference specification that is available as a resource to users of this Code.

1.6.3 Calculations pertinent to design shall be submitted with the construction documents if required by the authority having jurisdiction.

R1.6.3 Analyses and designs should include calculations, evaluation, and design assumptions. If computer-based analyses and designs, such as finite element methods, are used, they should include input and computer-generated output.

1.6.4 Load testing and scale model testing shall be permitted to supplement calculations.

R1.6.4 Results from load tests, scale model tests, and other types of physical testing can be used to supplement analytical procedures in the evaluation of existing structures. Strength evaluation procedures are described in ACI 437R; scale model tests are described in White (1970).

1.7—Assessment, design, and construction requirements if used with an existing building code

1.7.1 If this Code is used in conjunction with an existing building code, assessment, repair, and rehabilitation shall be in accordance with the existing building code and the provisions of 1.7.

R1.7—Assessment, design, and construction requirements if used with an existing building code

R1.7.1 The provisions of this Code are intended to be used in combination with the existing building code. The existing building code provides criteria for evaluation, when required, and criteria for repair or rehabilitation, when required. These existing building code criteria are supplemented by 1.7.2 through 1.7.9.

The code governing existing buildings in the United States is commonly the IEBC. The IEBC provides limits regarding the extent of damage that can be repaired using the original building code.

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1.7.2 In all cases, assessment, repair, and rehabilitation shall be permitted to conform to the requirements of the *current building code*.

R1.7.2 The current building code is always permitted as the design-basis code because it provides an acceptable level of safety. Use of the current building code may, however, result in an overly conservative assessment of an older structure. Therefore, the choice to use this option should be given careful consideration. The licensed design professional should review the benefits and drawbacks of using the current building code with the Owner at the start of the project.

1.7.3 Loads shall be as specified in the existing building code. Load combinations and strength reduction factors shall be in accordance with Chapter 5.

1.7.4 Evaluation and assessment of the existing structure shall be performed as required by the existing building code.

R1.7.4 The requirements of the existing building code should be followed to determine the required extent of evaluation and assessment of the existing structure. If the existing building code permits repair to the pre-damage condition without requiring assessment or evaluation, such repair is permitted by this Code. If the existing building code does not provide specific assessment requirements, 1.8 and Chapter 6 can be used for assessment of existing structures.

1.7.5 Design of repairs and rehabilitation shall be in accordance with Chapters 7 and 8.

1.7.5.1 Repairs that do not change the strength, stiffness, or ductility of the seismic-force-resisting system are permitted for structures in all Seismic Design Categories.

R1.7.5.1 The licensed design professional should evaluate any required repairs to the seismic-force-resisting system of a structure and determine how the repairs will affect the performance of the structure under design loadings. Changes in the strength, stiffness, or ductility of the structure may results in unintended distribution of load during seismic loading. Repairs to the seismic-force-resisting system can be designed using this Code if the repairs do not affect the strength, stiffness, or ductility of the structure. Seismic performance evaluation of existing structures is described in FEMA P-58 and FEMA P-695.

1.7.5.2 For structures assigned to Seismic Design Category A in accordance with ASCE/SEI 7, repair and rehabilitation of the lateral-force-resisting system shall be permitted in accordance with this Code.

R1.7.5.2 Repairs to the lateral-force-resisting system of a structure that result in a change in the strength, stiffness, or ductility of those elements are only permitted in structures in Seismic Design Category A in accordance with ASCE/SEI 7.

1.7.5.3 This Code shall not be used for repair or rehabilitation of elements of seismic force-resisting systems that result in a change in the strength, stiffness, or ductility of those elements from their pre-damage condition for structures in Seismic Design Categories B through F in accordance with ASCE/SEI 7.

R1.7.5.3 This Code is not permitted for seismic force-resisting systems because changes in strength, stiffness, or ductility introduced during rehabilitation can adversely affect the overall seismic resistance of the structure. For this reason, repair and rehabilitation of the seismic force-resisting system is beyond the scope of this Code. Other standards, such as "Seismic Evaluation and Retrofit of Existing Buildings," (ASCE 41) address repair and strengthening of seismic force-resisting systems.

1.7.6 This Code is not to be used for strengthening of elements of seismic-force-resisting systems.

R1.7.6 Evaluation and strengthening of the seismic-force-resisting system should be performed in accordance with the requirements of the existing building code and standards

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CODE

COMMENTARY

such as ASCE 41, which was specifically developed for seismic evaluation of existing structures.

1.7.6.1 Voluntary strengthening shall be permitted, subject to the limitations of the existing building code and Chapter 5.

1.7.7 Design of new elements shall be in accordance with the requirements of the *existing building code*.

1.7.7.1 Detailing of new elements and connection of new elements to the existing structure shall be in accordance with the current building code.

1.7.8 All work shall conform to the requirements of Chapters 9 and 10.

1.8—Assessment, design, and construction requirements if not used with an existing building code

1.8.1 If this Code is used as a stand-alone code, preliminary assessment, assessment, repair, and rehabilitation shall be in accordance with 1.8.2 through 1.8.12.

R1.8—Assessment, design, and construction requirements if not used with an existing building code

R1.8.1 Section 1.8 provides similar information to 1.7 but is applicable if this Code is used as a stand-alone code. Chapter 4 provides criteria for evaluation, if required, and design criteria for repair or rehabilitation, if required. The criteria in Chapter 4 are supplemented by the requirements of 1.8.2 through 1.8.12.

1.8.2 In all cases, it shall be permitted to conform to the requirements of the current building code.

R1.8.2 As stated in R1.7.2, it is always permitted to use the current building code as the *design-basis code*. However, this may result in an overly conservative assessment of an older structure. The licensed design professional should review benefits and drawbacks of using the current building code with the Owner at the start of the project.

1.8.3 The *design-basis code* shall be determined in accordance with Chapter 4 based upon the results of the preliminary assessment and assessment.

1.8.4 *Preliminary assessment and assessment*

1.8.4.1 Preliminary assessment of an existing structure shall be conducted within the *work area*. The preliminary assessment shall include investigation and review of the structure, construction documents, reports, local jurisdictional codes, and other available documents. Existing in-place conditions shall be visually or otherwise investigated to establish existing geometry and structural conditions.

R1.8.4 *Preliminary assessment and assessment*

R1.8.4.1 The goal of the preliminary assessment is to examine available information about the structure within the work area, and to make an initial determination of its adequacy to withstand in-place environmental conditions and design loads. The results of the preliminary assessment should be used to make decisions regarding the current in-place condition, need for additional information, work items necessary as part of the assessment, possible rehabilitation design and construction work to consider, and if there is a need for temporary shoring for safety of the existing structure. The preliminary assessment results should be updated as additional data regarding the examined structure become available.

The licensed design professional may determine that 4.6 applies in a preliminary assessment based on engineering judgment and without analysis if the following are confirmed:

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CODE

1.8.4.2 The preliminary assessment shall determine if visible potentially dangerous structural conditions are present.

1.8.4.3 In performing a preliminary assessment, it shall be permitted to use the original or current building code for assessment criteria.

1.8.4.4 If required, the in-place strength of the existing structure shall be determined considering in-place geometry and material properties including effects of material deterioration and other deficiencies. If material properties are not available, a preliminary assessment is permitted using material properties in accordance with 6.3.2.

COMMENTARY

(a) Historical performance of the structure and visual observation of the structural condition of members and systems indicate acceptable behavior precluding assessment by 4.3.

(b) Review of construction documents and observation of current structural conditions indicate damage or deterioration of the structure below the level requiring assessment by 4.4 and 4.5.

(c) Modifications for additions, alterations, and changes in occupancy are not planned.

If the structure is determined to be structurally acceptable, repairs are permitted that address durability and serviceability of 4.6 without analyzing members and systems and without checking the demand-to-capacity ratio limits of 4.3 through 4.5. Structural performance should be considered acceptable if past and present performance has been satisfactory and observations do not indicate structural distress beyond levels expected.

The extent of damage or deterioration should be limited, and the licensed design professional should not have a concern about the capacity of the structure if repairs are completed using the provisions of 4.6 without verifying the demand-to-capacity limits of 4.4 and 4.5.

R1.8.4.2 Potentially dangerous structural conditions may require the Owner to install shoring, limit access, or take other measures to mitigate these conditions.

R1.8.4.3 The assumed preliminary assessment criteria should be substantiated or modified in accordance with the assessment details of Chapter 6.

R1.8.4.4 If required as a part of the preliminary assessment, strength calculations should be based on in-place conditions and should include an assessment of the loss of strength due to deterioration. 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, FEMA P-58, FEMA P-154, FEMA 306, FEMA 307, ASCE/SEI 11, ASCE/SEI 41, ATC-20, ATC-45, ATC-78 and The Concrete Society *Technical Report 68* (2008). If material test results are initially unavailable, historical properties based on typical values used at the time of construction can be used in a preliminary evaluation. If available, material properties from construction documents can also be used in a preliminary evaluation.

The assessment of existing structures should initially focus on critical gravity-load-resisting members such as columns, walls, and members that are expected to have limited ductility, followed by an assessment of the lateral-load-resisting system.

Assessing fire damage and other forms of deterioration that affect 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.

CODE

COMMENTARY

Examples of deterioration mechanisms that result in possible changes in material properties or structural performance include corrosion of steel reinforcement, thermal damage, chemical reactions such as alkali-aggregate, and freezing-and-thawing damage.

Conditions to be documented include cracking, spalls, member deflection, cross-section dimensions that are different than specified on the original construction drawings, and construction deviating from tolerances permitted under the original design criteria.

1.8.4.5 A structural assessment in accordance with Chapter 6 shall be performed if a member or structure exhibits damage, displacements, deterioration, structural deficiencies, or performance that is unexpected or inconsistent with available design and construction documents or code requirements in effect at the time of construction.

R1.8.4.5 The preliminary assessment is generally the first portion of the work necessary to determine the rehabilitation category. Based upon preliminary assessment results, a structural assessment may be required to determine the extent of damage or if potentially dangerous structural conditions are present. However, in some cases, the licensed design professional may deem that a structural assessment is not required based on judgment in accordance with 1.8.4.1 through 1.8.4.4.

1.8.5 Loads shall be as specified in the design-basis code. Load combinations and strength reduction factors shall be in accordance with Chapter 5.

1.8.6 Design of repairs and rehabilitation shall be in accordance with Chapters 7 and 8.

1.8.6.1 Repairs that do not change the strength, stiffness, or ductility of the seismic-force-resisting system are permitted for structures in all Seismic Design Categories.

R1.8.6.1 Refer to R1.7.5.1.

1.8.6.2 For structures assigned to Seismic Design Category A in accordance with ASCE/SEI 7, repair and rehabilitation of the lateral-force-resisting system shall be permitted in accordance with this Code.

R1.8.6.2 Refer to R1.7.5.2.

1.8.6.3 Use of this Code is not permitted for repair or rehabilitation of elements of seismic force-resisting systems that result in a change in the strength, stiffness, or ductility of those elements from their pre-damage condition for structures in Seismic Design Categories B through F.

R1.8.6.3 Refer to R1.7.5.3.

1.8.7 Retrofit of elements of seismic-force-resisting systems shall be performed in accordance with ASCE/SEI 41.

1.8.8 Design of new elements shall be in accordance with the *design-basis code.*

1.8.8.1 Detailing of new elements and connection of new elements to the existing structure shall be in accordance with the current building code.

1.8.9 Voluntary strengthening shall be permitted, subject to the limitations of Chapters 4 and 5.

1.8.10 All work shall conform to the requirements of Chapters 9 and 10.

CODE

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

This chapter defines notation and terminology used in this Code.

c = depth of neutral axis, in.

D = effect of service dead load

d_t = distance from extreme compression fiber to centroid of extreme tension reinforcement, in.

$\bar{f_c}$ = average core strength modified to account for the diameter and moisture condition of the core, psi

f_c' = specified concrete compressive strength, psi

f_{ceq} = equivalent specified concrete strength used for evaluation, psi

f_y = specified yield strength of steel reinforcement, psi

$\bar{f_y}$ = average yield strength value for steel reinforcement, psi

f_{yeq} = equivalent yield strength of steel reinforcement used for evaluation, psi

k_c = coefficient of variation modification factor for concrete testing sample size

k_s = coefficient of variation modification factor for steel testing sample size

L = effect of service live load

ℓ_t = span of member under load test and taken as the smaller of: (a) distance between centers of supports; and (b) clear distance between supports plus thickness h of member; for a cantilever, it shall be taken as twice the distance from face of support to cantilever end, in.

n = number of specimens tested

R_a = service load capacity of structural member, system, or connection including effects of damage, deterioration of concrete and reinforcement, and faulty construction determined using allowable stresses according to the original building code

R_{cn} = current in-place nominal capacity of structural member, system, or connection including the effects of damage, deterioration of concrete and reinforcement, and faulty construction

R_{ex} = nominal resistance of the structure during an extraordinary (that is, low-probability) event calculated using the probable material properties

R_n = nominal capacity of structural member, system, or connection excluding the effects of damage, deterioration of concrete and reinforcement, and faulty construction

S = effect of service snow load

T_g = glass transition temperature, °F

U = required strength or demand using nominal loads and factored load combinations for strength design provisions

U_c = required strength or demand using nominal loads of the current building code and factored load combinations of ASCE/SEI 7 for strength design

COMMENTARY

R2—NOTATION AND DEFINITIONS

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CODE

U_o = required strength or demand using nominal loads and factored load combinations of the original building code for strength design

U_{o^*} = required strength or demand using nominal loads of the original building code and factored load combinations of ASCE/SEI 7 for strength design

U_s = demand using service loads of the original building code and allowable stress load combinations of the original building code

V = coefficient of variation (a dimensionless quantity equal to the sample standard deviation divided by the mean) determined from testing of concrete or steel samples from structures

v_{ni} = nominal interface shear stress capacity

v_u = resultant interface stress demand from the transfer of tension and shear

ε_t = net tensile strain in the extreme tension reinforcement at nominal strength

ε_y = yield strain of steel reinforcement

ϕ = strength reduction factor

ϕ_{ex} = strength reduction factor used to check strength of the structure without external reinforcement after an extraordinary event

ϕ_o = strength reduction factor from the original building code used in the design of an existing structure

2.2—Definitions

assessment—the process of investigating by systematically collecting information that affects the performance of an existing structure; evaluating the collected information to make informed decisions regarding the need for repair or rehabilitation; detailing of findings as conclusions and reporting recommendations for the examined structural concrete work area (member, system, or structure).

COMMENTARY

R2.2—Definitions

For consistent application of this Code, it is necessary that terms be defined if they have particular meanings in the Code. The definitions given are for use in application of this Code only and do not always correspond to ordinary usage. A glossary of common terms relating to cement manufacturing, concrete design and construction, and research in concrete is contained in "ACI Concrete Terminology" (CT) available on the ACI website.

This definition with specific details for existing concrete is adapted from ACI 364 and ASCE/SEI 11.

An assessment is the process of acquiring knowledge of the existing structure for the purpose of judging its future performance. The results of the investigation and evaluation are used to make decisions on the appropriate course of action regarding the future use of the structure and the suitability of the structure to continue in service.

Assessments should be limited to the work area and may include:

(a) Investigation of the in-place condition of the existing structure by:

i. Collection and review of field data for the structure, such as geometry, material strengths, conditions, symptoms of distress, extent of damage, measurement of displacements, environmental factors, and reinforcement sizes and placement

ii. Collection of background data, such as construction plans, construction records, original code, current code, and existing building code, and historical events

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CODE

COMMENTARY

(b) Evaluation of an existing structure, member, or system of the work area (refer to Commentary for “evaluation”)

(c) Findings and conclusions of the investigation and evaluation:

i. Define the existing structure, member, or system rehabilitation category using the assessment criteria of this Code

ii. Identify the work area, scope of work, and likely cause of damage, distress, and deterioration

iii. Identify extents of possible faulty construction

iv. Evaluate test results to determine cause of failure and estimate likely future performance

(d) Determine repair and rehabilitation concepts, strategies, alternatives, and recommendations

i. Develop cost-impact or economic study as necessary to appraise remedial work and maintenance

ii. Describe repair and rehabilitation work recommendations

(e) Report conclusions and recommendations:

i. Work area limits and limitations of information collected and evaluated

ii. Assessment criteria and results of the evaluation such as calculations, tests, and analyses

iii. Details of findings (conclusions) and recommendations

iv. Safety issue requirements (for example, recommendation for any temporary shoring)

assessment criteria—codes, standards, loads, demands, capacities, strength reduction factors, materials, material properties, connections, and details used in the evaluation.

authority having jurisdiction—person or entity that has legal control over the applicable building code and permitting procedures for a structure.

An example of an authority having jurisdiction is the local building official.

bond—(1) mechanism for transfer of forces to reinforcing bars embedded in concrete through a combination of adhesion, friction, and interlock of the bar deformations; (2) adhesion between layers of a repair area or between a repair material and a substrate produced by adhesive properties of the repair material or other supplemental materials.

bond-critical application—strengthening or repair system that relies on load transfer from the substrate to the system material through shear and tension transfer at the interface; strengthening or repair system for which bond rather than mechanical attachment is used as the primary load transfer mechanism.

capacity—the strength, stiffness, ductility, or energy dissipation of a material, member, or system as determined by analysis or testing.

compatible—the ability of two or more materials in contact to function together with no significant detrimental effect on the intended service life.

condition assessment—a process of reviewing information gathered about the current condition of a structure or its components, its service environment and general circumstances, whereby its adequacy for future service may be

CODE

established against specified performance requirements for a defined set of loadings or environmental exposures, or both.

connector steel—steel elements, such as reinforcing bars, shapes, or plates, embedded in concrete or connected to embedded elements to transfer load, restrain movement, or provide stability.

contact-critical application—strengthening or repair system that relies on load transfer from the substrate to the system material through bearing perpendicular to the interface.

construction documents—written and graphic documents and specifications prepared or assembled that describe the location, design, materials, and physical characteristics of the elements of a project necessary for obtaining a building permit and for construction of the project.

damage—a decrease in the capacity of an existing member or structure resulting from unexpected events, such as excessive loads and displacements, or from deterioration.

dangerous—designation applied to any building, structure, or portion thereof that meets any of the following conditions:

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 actual loads already in effect; or under snow, wind, rain, flood, earthquake, or other environmental loads when such loads are imminent.

demand—the force, deformation, energy input, and chemical or physical attack imposed on a material, member, or system that has to be resisted.

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

design-basis criteria—codes, standards, loads, displacement limits, material properties, connections, details, and protections used in the design of mandated or voluntary work.

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

deterioration—(1) physical manifestation of failure of a material (for example, cracking, delamination, flaking, pitting, scaling, and spalling) caused by environmental or internal autogenous influences; (2) decomposition of material during either testing or exposure to service conditions.

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

COMMENTARY

An example of a contact-critical application is the addition of a confinement jacket around a column.

This definition has been adopted from the IEBC.

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CODE

equivalent concrete cover—a system to supplement insufficient concrete cover to provide durability or fire protection equivalent to the specified minimum cover.

evaluation—the process of determining and judging the adequacy of a structure, member, or system for its current intended use or performance objective.

COMMENTARY

This definition is adapted from ASCE/SEI 11. In a structural evaluation, the licensed design professional should determine the structural adequacy, serviceability, and durability of an existing structure based upon compiled data and other knowledge. An evaluation may require professional judgment to gauge structural adequacy. Structural analyses and member capacity calculations may be required to determine possible ranges of existing structural capacities and variations in demands. The goal of the evaluation process is to appraise the in-place condition to determine adequacy for current or proposed future use. Structural evaluation requires determining capacity and demand, which may vary widely depending on the acquired information, test results, computational models, and analyses. Determining the demand-to-capacity ratios; and judging structural reliability limits may be open to interpretation based on project requirements, structural experience, knowledge, and past performance.

Evaluation activities may include:

(a) Tests to confirm reinforcement location and orientation, strength of materials or structural capacity of existing members or systems, or the presence of contaminants

(b) Analysis of test results to establish statistical equivalent material properties, limits of faulty construction, and structural capacity

(c) Screening of observations and tests for mechanisms and causes of damage, distress, and deterioration

(d) Establishing the assessment criteria

(e) Calculating demand loadings, serviceability limits, lateral displacements, and durability requirements

(f) Analysis of the structure to determine the capacity of the structure to withstand current or future load demands and comply with serviceability limits

- i. Determination of demand-to-capacity ratios to appraise structural adequacy and judge the need for repair and rehabilitation
- ii. Determination of maintenance requirements necessary for the service life of the structure

existing structure—(1) when used in conjunction with an existing building code, a building erected prior to the date of adoption of the appropriate code, or one for which a legal building permit has been issued; (2) when not used in conjunction with an existing building code, building for which a legal certificate of occupancy has been issued; for structure not covered by a certificate of occupancy, a structure that is complete and permitted for use or otherwise legally defined as an existing structure or building

Definition 1 is consistent with the IEBC. Definition 2 is different from the IEBC definition of an existing building.

factored load—product of the nominal load and load factor.

faulty construction—deficient construction resulting from errors or omissions in design or improper construction causing displacement of supporting portions of the structure or resulting in deficient materials, incorrect geometry or location of members, improper reinforcement, or con

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CODE

detailing that reduce capacity of or have an adverse effect on long-term durability or serviceability.

in-place condition—current condition of an existing structure, system, member, or connection including component sizes and geometry, material properties, faulty construction, deterioration, and damage.

interface reinforcement—(1) existing or supplemental reinforcement that is properly anchored on both sides of an interface; (2) post-installed reinforcement such as adhesive anchors or mechanical anchors, or other mechanical connections that provides a method of force transfer across an interface.

interface shear stress—shear stress resulting from transfer of forces at bonded interfaces between repair material and existing substrate used to achieve composite behavior.

licensed design professional—(1) an engineer or architect who is licensed to practice structural design as defined by the statutory requirements of the professional licensing laws of a state or jurisdiction; (2) the engineer or architect, licensed as described, who is responsible for the structural design of a particular project (also historically engineer of record).

nominal load—magnitude of load specified by the applicable code before application of any load factor.

nonstructural concrete—any element or portion thereof made of plain or reinforced concrete that is not required to transfer gravity load (other than self-weight), lateral load, or both, along a load path of a structural system to the ground.

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

potentially dangerous—(1) structural state of existing concrete within a work area for an individual structural member, structural system, or structure that meets the definition of dangerous or unsafe, is unstable, has potential of collapse of overhead components or pieces (falling hazards), has been determined to have demand-to-capacity ratio exceeding the limit of 4.3.2, or has potentially hazardous resistance for seismic events; (2) a limit state of unacceptably low margin of safety against collapse without supplemental resistance.

rehabilitation—repairing or modifying an existing structure to a desired useful condition.

repair—the reconstruction or renewal of concrete parts of an existing structure for the purpose of its maintenance or to correct deterioration, damage, or faulty construction of members or systems of a structure.

repair reinforcement—reinforcement used to provide additional strength, ductility, confinement, or any combination of the three, to the repaired member.

repair, structural—restoring a damaged or deteriorated structure to its original capacity or increasing the capacity of a structure.

COMMENTARY

Potentially dangerous conditions of an existing concrete member, system, or structure include conditions that may be defined as dangerous or unsafe in the IEBC. These conditions may be overtly dangerous or unsafe depending on the circumstances or load probability between the time of determination and repair.

Concrete rehabilitation includes: repair to restore original capacity; strengthening to increase the capacity to the current building code requirements; seismic retrofits in accordance with ASCE/SEI 41; and modifications addressing additions, alterations, and change of occupancy.

The definition is adapted from the IEBC and is specific for repair of materials, components, or elements of existing concrete structures. Faulty materials, components, or elements of a structure are considered to be faulty construction resulting from errors or omissions in design or construction.

The definition addresses increasing the capacity to include enhancements such as ductility of existing concrete members. Repairs to nonstructural members, whose failure

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CODE

repair system—the combination of existing and new components, which may include existing reinforcement, repair materials, supplementary reinforcement, and supplemental structural members.

serviceability—structural performance under service conditions.

shoring—temporary support of excavations, formwork, members during repairs, or potentially dangerous structures; the process of erecting shores.

specialty engineer—a licensed design professional retained to design a delegated portion of the project.

stability, global—stability of the overall existing structure with respect to vertical support, uplift, overturning, lateral instability, or sliding.

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

strengthening—increasing the capacity of an existing structure or a portion thereof.

structural analysis—process of using engineering mechanics to determine internal demands on, and capacities of, a structure, member, or system.

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.

substantial structural damage— (1) level of damage as defined in the *existing building code*; (2) if this Code is used as a stand-alone code, substantial structural damage shall be as defined in Chapter 4.

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

undercutting—concrete removal around the reinforcement circumference to allow for existing reinforcement to be encapsulated in repair material.

work area—that portion or portions of a structure consisting of all areas indicated in the construction documents or identified by the owner and licensed design professional for assessment; it excludes portions of the structure where incidental work entailed by the intended work must be performed and portions of the structure where work not initially intended by the owner is required by this Code.

COMMENTARY

would cause or result in potentially dangerous structural conditions, are considered structural repairs.

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CODE

CHAPTER 3—REFERENCED STANDARDS

American Concrete Institute

ACI 216.1-14—Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies

ACI 318-19—Building Code Requirements for Structural Concrete and Commentary

ACI 369.1-17—Standard Requirements for Seismic Evaluation and Retrofit of Existing Concrete Buildings

ACI 437.2-13—Code Requirements for Load Testing of Existing Concrete Structures and Commentary

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

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

American Institute of Steel Construction

ANSI/AISC 360-16—Specification for Structural Steel Buildings

American Society of Civil Engineers

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

ASCE/SEI 37—Design Loads on Structures during Construction

ASCE/SEI 41—Seismic Evaluation and Retrofit of Existing Buildings

American Welding Society

AWS 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/A185M-18—Standard Specification for Steel Welded Wire Reinforcement, Plain, for Concrete (withdrawn 2013)

ASTM A370-14—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)

COMMENTARY

R3—REFERENCED STANDARDS

Both current and withdrawn standards are referenced. Standards that are referenced in the design-basis code are applicable for the assessment 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.

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CODE

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

ASTM A497/A497M—Standard Specification for Steel Welded Wire Reinforcement, Deformed, for Concrete (withdrawn 2013)

ASTM A615/A615M-14—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)

ASTM A706/A706M-14—Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement

ASTM A955/A955M-15—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-13—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 C1580-15—Standard Test Method for Water-Soluble Sulfate in Soil

ASTM C1583/C1583M-13—Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off Method)

ASTM D516-16—Standard Test Method for Sulfate Ion in Water

ASTM D4065-12—Standard Practice for Plastics: Dynamic Mechanical Properties: Determination and Report of Procedures

ASTM D4130-15—Standard Test Method for Sulfate Ion in Brackish Water, Seawater, and Brines

ASTM E329-14a—Standard Specification for Agencies Engaged in Construction Inspection, Testing, or Special Inspection

BSI Group

BS EN 1504-10:2017—Products and systems for the protection and repair of concrete structures. Definition, requirements, quality control and evaluation of conformity. Site application of products and systems and quality control of the works

International Code Council

IBC—International Building Code

IEBC—International Existing Building Code

COMMENTARY

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CODE

CHAPTER 4—CRITERIA AS A STAND-ALONE CODE FOR ASSESSMENT, REPAIR, AND REHABILITATION OF EXISTING STRUCTURES

4.1—General

4.1.1 This Chapter applies for the assessment and for repair and rehabilitation of existing structures and their components if this Code is used as a stand-alone code.

4.1.2 The design-basis code for the project shall be based on requirements set forth in this Chapter.

4.1.3 Assessment results shall be used to classify the work and to establish the design-basis criteria.

4.1.4 Design-basis criteria shall be used to establish the applicable building code for repair and rehabilitation design.

4.2—Criteria for the assessment, repair, and rehabilitation design of existing concrete structures

4.2.1 The design-basis criteria of the project shall be determined based upon the results of the preliminary assessment as described in 1.8 or the detailed assessment as described in Chapter 6, if performed, using the requirements in this Chapter. The design-basis criteria shall be used consistently for all assessment, repair, and rehabilitation design in the work area or areas of the structure.

4.2.1.1 It shall be permitted to use the current building code as the design-basis code for all damage states, deterioration, and faulty construction.

4.2.2 Assessment criteria, design-basis criteria, and the requirements for applying these criteria are provided in 4.3 through 4.9.

4.2.3 The current building code shall be used to detail new members and connections between new concrete members and existing construction. If the original building code is used as the design-basis criteria for repairs, new or supplemental elements built integrally with the existing concrete

COMMENTARY

R4—CRITERIA AS A STAND-ALONE CODE FOR ASSESSMENT, REPAIR, AND REHABILITATION OF EXISTING STRUCTURES

R4.1—General

R4.1.1 The assessment, repair, and rehabilitation design criteria and associated demand-to-capacity ratios used in this Chapter were developed to be consistent with the performance-based procedures presented in 1.3.1.3 of ASCE/SEI 7-16. The licensed design professional for the project should evaluate the applicability of this Chapter to non-building structures (for example, environmental engineering structures such as those addressed in ACI 350 - Code Requirements for Environmental Engineering Concrete Structures and Commentary) as these structures may be designed with procedures that are not compatible with the performance-based procedures of ASCE/SEI 7.

R4.2—Criteria for the assessment, repair, and rehabilitation design of existing concrete structures

R4.2.1 Structures constructed under previously adopted codes or before the adoption of a building code may not satisfy all current building code requirements. This Chapter contains specific requirements that determine if existing structures should be repaired or rehabilitated to satisfy the requirements of the original or the current building code, whichever serves as the design-basis code. Local ordinances may also require that a structure be rehabilitated to satisfy the current code.

R4.2.1.1 Most current building codes, such as those based on the IBC, provide acceptable safety based on consistent statistical probabilities. If the current building code is used, the resulting demand-to-capacity ratios provide limits that should not be exceeded if assessing and designing remedial construction.

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CODE

structure shall be designed using either the original building code or the current building code.

4.2.4 Detailing of the existing reinforcement within the work area need not comply with the current building code if the original building code is the design-basis criteria, and both of the following conditions are satisfied:

(a) The damage or deterioration to existing reinforcement is addressed; and

(b) The repaired work area has capacity equal to or greater than demand in accordance with 5.2.2 if using the original building code requirements or satisfies the requirements of 4.5.3 if using allowable stress design.

COMMENTARY

R4.2.4 The licensed design professional should review conditions related to anchorage of the existing reinforcing steel. Cracking near the ends of the existing reinforcing steel should be reviewed to determine if the cracking is indicative of potential reinforcement anchorage failure. Research has shown that development length equations from previous versions of ACI 318 may be unconservative for top reinforcing steel bars (Feldman and Cairns 2017). Changes in ACI 318 have increased the development length of reinforcing steel.

If the basis of design is the current building code, the licensed design professional should consider the following:

(a) Assessing demand-to-capacity ratios for the existing reinforcing steel using current development length provisions.

(b) Confinement details of the reinforcement if assessing seismic resistance.

The licensed design professional should determine if structural behavior indicates adequate performance. ACI 224.1R, ACI 437R, and ACI 437.1R provide guidance in judging acceptable performance.

4.3—Potentially dangerous structural conditions

4.3.1 If there is a reason for the licensed design professional to question the capacity of the structure or if potentially dangerous structural conditions are observed as a part of the preliminary assessment, an assessment shall be performed in the work area.

R4.3—Potentially dangerous structural conditions

R4.3.1 Structural assessments are required if damage, deterioration, structural deficiencies, or behavior are observed during the preliminary assessment that are unexpected or inconsistent with available construction documents. The structural assessment should be performed in accordance with 1.8 or Chapter 6, or both. Results of the assessment should also be reviewed to identify the presence of potentially dangerous structural conditions, which could include any instability, the potential for collapse of overhead components or pieces (falling debris hazards), or an unacceptable risk of collapse under service load conditions.

4.3.2 For gravity, fluid, soil, and wind loads, potentially dangerous structural conditions exist in members or structures if the demand-to-capacity ratio is greater than 1.5, as given in Eq. (4.3.2).

$$U_c/\phi R_{cn} > 1.5 \tag{4.3.2}$$

In Eq. (4.3.2), U_c is the strength-design demand determined by using the nominal gravity, fluid, soil, and wind loads identified in the current building code and the factored load combinations of ASCE/SEI 7, excluding seismic, flood, and tsunami forces; and ϕR_{cn} is the current in-place nominal capacity adjusted by the strength reduction factor (ϕ) in 5.3 or 5.4.

R4.3.2 Demand-to-capacity ratios are used to quantify the adequacy of the member or structure. The threshold demand-to-capacity ratios determine when different levels of intervention may be required. This Chapter provides how the applicable demands and capacities are determined. Demands may be determined based upon factored loads associated with *current building codes* (U_c as defined in 4.3.2) or factored loads used for the original design (U_o as defined in 4.5.2) of the structure. The calculated capacity of the structure will vary depending upon the condition of the structure and extent of evaluation used to confirm as-built properties of the structure.

In assessing potentially dangerous structural conditions, the strength-design demand of Eq. (4.3.2) combines current building code nominal gravity loads (dead, live, and snow) with lateral loads from fluid, soil, and wind (excluding seismic, flood, and tsunami forces) using factored load combinations of ASCE/SEI 7-16. The demand-to-capacity

ratio for unsafe conditions was developed consistent with the performance-based procedures contained in 1.3.1.3 of ASCE/SEI 7-16. A demand-to-capacity ratio greater than 1.5, calculated using Eq. (4.3.2), represents a condition with limited to no margin of safety against failure for ASCE/SEI 7 loads (Stevens and Kesner 2016a; Stevens et al. 2019).

In the assessment of potentially dangerous structural conditions, the licensed design professional should determine if it may be appropriate to include structural redundancies, alternative load paths, primary and secondary supporting elements, redistribution of loads, collapse mechanisms, reduced live loads, measured displacements (sagging, listing, leaning, and tilting), second-order effects, and other loads specific to the structure, such as drifting snow, lateral earth and fluid pressures, self-straining loads, ice, and floods. References for potentially dangerous structural conditions include: commentary to Chapter 1 of ASCE/SEI 7-10, Galambos et al. (1982), Ellingwood et al. (1982), and Ellingwood and Ang (1972). These references provide target reliability indices, basic probability theory, and concepts for an evaluation.

4.3.2.1 If the demand-to-capacity ratio given by Eq. (4.3.2) exceeds 1.5 for a member or members in the work area, the design-basis criteria shall be the current building code.

4.3.2.2 Assessment and repair shall be limited to the members, elements, and connections identified to be *potentially dangerous* and their load paths to the supporting ground.

4.3.2.3 For structures without potentially dangerous structural conditions and if the demand-to-capacity ratio is 1.5 or less, the design-basis criteria shall be determined in accordance with 4.4 through 4.9.

4.3.3 Assessment criteria for seismic resistance, unless addressed by the authority having jurisdiction, are limited to conditions associated with Structural Performance Level, Collapse Prevention of structures in Seismic Design Category D, E, and F of ASCE/SEI 7 using Earthquake Hazard Level, BSE-1E and shall be determined using ASCE/SEI 41 and this Code. The design-basis criteria to address seismic conditions in concrete structures shall be this Code and ASCE/SEI 41.

R4.3.3 Compliance with ASCE/SEI 41 for Structural Performance Level, Collapse Prevention using an applicable Earthquake Hazard Level BSE-1E should be reviewed and approved by the authority having jurisdiction for the assessment of potentially hazardous seismic conditions of concrete structures. Assessment of seismic conditions for concrete structures is not required, but may be considered, for structures in regions of low or moderate seismicity. If no requirements for seismic structural conditions are provided by the authority having jurisdiction, the licensed design professional should refer to ATC-78, the IEBC, and ASCE/SEI 41 appendices for guidance.

4.4—Substantial structural damage

4.4.1 If there is a reason for the licensed design professional to question the capacity of the structure based upon the results of the preliminary assessment, an assessment shall be performed in the work area to determine if substantial structural damage is present. Substantial structural damage in the work area shall be assessed using cur

R4.4—Substantial structural damage

CODE

code demands, nominal capacities according to Chapter 6, and strength reduction factors according to 5.3 or 5.4.

4.4.2 Substantial structural damage to vertical members of the lateral-force-resisting system shall have occurred if, in any story, the shear walls or columns of the lateral-force-resisting system are damaged such that the nominal capacity of the current lateral-load-resisting system of the structure ($\sum R_{cn}$) in any horizontal direction is reduced more than 33 percent from its pre-damage condition ($\sum R_n$). This relationship is given by Eq. (4.4.2).

$$\sum R_n / \sum R_{cn} > 1.5 \tag{4.4.2}$$

4.4.3 Substantial structural damage to vertical elements of the gravity-load-resisting system shall have occurred if:

a) For any wall, column, or group of vertical elements of the gravity-load-resisting system whose tributary area is more than 30 percent of the total area of the structure's floor(s) and roof(s), the damage is such that the total in-place vertical nominal capacity ($\sum R_{cn}$) is reduced more than 20 percent from its predamage condition ($\sum R_n$) as given by Eq. (4.4.3a), and concurrently

b) The current building code factored gravity load combination (dead, live, and snow) demand to in-place vertical design capacity ratio of the damaged elements is more than 1.33, as given by Eq. (4.4.3b).

$$\sum R_n / \sum R_{cn} > 1.25 \tag{4.4.3a}$$

$$\sum U_c / \sum \phi R_{cn} > 1.33 \tag{4.4.3b}$$

4.4.4 The design-basis criteria for structures with substantial structural damage shall be the current building code demands, supplemented by requirements of this Code for the existing structure and ASCE/SEI 41 for seismic design provisions, for the following:

(a) Lateral-force-resisting system in both directions for the case of substantial structural damage in either direction from lateral forces (4.4.2); or

(b) Vertical elements of the gravity-load-resisting system for the case of substantial structural damage from gravity loads (4.4.3).

4.4.4.1 Structures with substantial structural damage caused by an earthquake shall be assessed or rehabilitated for load combinations that include earthquake effects. The seismic design provisions of ASCE/SEI 41 shall be: (a) Earthquake Hazard Level BSE-1E with the Basic Performance Objective of "Life Safety" for Risk Category I, II, or III (ASCE/SEI 7) and of "Immediate Occupancy" for Risk Category IV; and (b) Earthquake Hazard Level BSE-2E with the Basic Performance Objective of "Collapse Prevention" for Risk Category I, II (ASCE/SEI 7) and of "Life Safety" for Risk Category III and IV (ASCE/SEI 7).

COMMENTARY

R4.4.2 The assessment criteria for substantial structural damage are specific to existing concrete structures, which were adapted from the IEBC.

R4.4.3 In Eq. (4.4.3b), the demand load is calculated using the current building code loads and the required load combinations and should include all current building code gravity loads judged to be applicable to the structure, such as drifting snow.

R4.4.4 Supplemental requirements of this Code for the design-basis criteria include strength reduction factors in accordance with 5.3 or 5.4, capacities according to Chapter 6, repairs in accordance with Chapter 7, durability in accordance with Chapter 8, repair construction in accordance with Chapter 9, and quality assurance in accordance with Chapter 10.

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CODE

4.4.4.2 Structures with substantial structural damage caused by wind shall be repaired for load combinations that include the current building code gravity and wind loads.

4.4.5 The design-basis criteria for structures with damage less than substantial structural damage shall be determined in accordance with 4.5 through 4.9.

4.5—Conditions of deterioration, faulty construction, or damage less than substantial structural damage with strengthening

4.5.1 If a member, system, or structure in the work area has deterioration, contains faulty construction, or damage determined to be less than substantial structural damage, it shall be assessed by checking one of the criteria in 4.5.2, 4.5.3, or 4.5.4. Provisions 4.5.2 through 4.5.4 shall not be applied in combination with each other.

4.5.2 The demand-to-capacity ratio of the member, system, or structure of the work area shall be evaluated using the nominal loads, load combinations, and capacities established by the original building code. Repairs are required if the demand-to-capacity ratio exceeds 1.0, as given by Eq. (4.5.2).

$$(U_o)/(\phi_o R_{cn}) > 1.0 \quad (4.5.2)$$

In Eq. (4.5.2), U_o is the strength-design demand determined by using the nominal loads and factored load combinations of the original building code. $\phi_o R_{cn}$ is the current in-place nominal capacity adjusted by the strength reduction factor (ϕ_o) of the original building code.

4.5.2.1 If $U_o/\phi_o R_{cn}$ is greater than 1.0, strengthening repairs are required to bring the structural capacity to the level required by the original building code.

4.5.2.2 If $U_o/\phi_o R_{cn}$ is 1.0 or less, repairs to strengthen the structure are not required.

4.5.3 If approved by authority having jurisdiction, alternative assessment criteria based on engineering principles shall be permitted for the member, system, or structure of the work area.

COMMENTARY

R4.5—Conditions of deterioration, faulty construction, or damage less than substantial structural damage with strengthening

R4.5.1 For structures without dangerous conditions or substantial structural damage, repairs can be made to satisfy the strength requirements of the original building code. Provision 4.5.2 addresses the situation where the original building code was based upon strength design principles. Provision 4.5.3 allows for alternative assessment criteria that may be advantageous if significant changes in demand have occurred. Provision 4.5.4 addresses the situation where allowable stress design methods were used in the original design.

R4.5.2 Concrete structures with damage less than substantial structural damage, deterioration, or containing faulty construction have historically been restored to the strength required by the original building code.

A demand-to-capacity ratio of limit of 1.0 allows strengthening repair that restores the structural reliability of the existing structure to the level prior to damage and deterioration.

Historical performance is often an acceptable indicator of adequate safety if the structure has been subjected to known loads even if the strength-design demand or capacity of the structure based on the requirements of the original building code is significantly different from the current building code.

If the capacity of the structure is not in question, assessment checks are not required.

R4.5.2.1 Repair of the existing concrete structure is permitted to restore a member, system, or structure within the work area to the capacity requirements of the original building code based on the material properties of the original construction.

R4.5.3 Assessment criteria other than the current building code or ASCE/SEI 41 may be used. The references of R4.3.2 should be considered in the selection of applicable assessment criteria. Stevens and Kesner (2016b) provides additional information on the alternate assessment criteria in this section.

Beyond using the current building code, the assessment criteria should address if the demand or capacity of the original structure or member is significantly inconsistent with current standards resulting in unacceptable structural safety. Factors such as an increase in load intensity; added loads; changes in load factors, strength reduction factors, or

CODE

COMMENTARY

load combinations; modification of analytical procedures; or differences between the capacity determined by the original and the current building code (such as a change from allowable stress design to strength design) should lead the licensed design professional to question the applicability of using the original building code for the assessment of an existing structure. Engineering principles to determine acceptable structural safety can be based on either a probabilistic evaluation of loads and capacities to show adequate structural reliability indices or an evaluation procedure based on demand-to-capacity ratios that are derived from the basic engineering principles as presented in current design standards.

Alternative assessment criteria for a structure that has damage less than substantial structural damage, deterioration, or faulty construction that are based on the demand-to-capacity ratio threshold of 1.05 using the original building code are as follows:

(a) If the current building code demand (U_c) does not exceed the original building code demand (U_o^*) by 5 percent ($U_c \leq 1.05U_o^*$), check the demand-to-capacity ratio using the original building code demand (U_o^*) to determine if it exceeds 1.05, as given by Eq. (R4.5.3a).

$$U_o^*/\phi R_{cn} > 1.05 \qquad \text{(R4.5.3a)}$$

If the demand-to-capacity ratio exceeds 1.05, the system or member should be strengthened using the original building code demand (U_o^*). If the demand-to-capacity ratio does not exceed 1.05, strengthening is not required.

(b) If the current building code demand (U_c) exceeds the original building code demand (U_o^*) by more than 5 percent ($U_c > 1.05U_o^*$), check the demand-capacity ratio using the current building code demand (U_c) to determine if it exceeds 1.1, as given by Eq. (R4.5.4b).

$$U_c/\phi R_{cn} > 1.1 \qquad \text{(R4.5.3b)}$$

If the demand-to-capacity ratio exceeds 1.1, the system or member shall be strengthened using the current code demand. If the demand to capacity ratio does not exceed 1.1, strengthening is not required.

The strength reduction factors (ϕ) of 5.3 or 5.4 are applied in both Eq. (R4.5.3a) and (R4.5.3b).

The current building code strength-design demand (U_c) combines current building code nominal gravity loads (dead, live, soil, and snow) using the factored load combinations of ASCE/SEI 7. The original building code strength-design demand (U_o^*) combines the original building code nominal gravity loads (dead, live, and snow) using the factored load combinations of ASCE/SEI 7.

The use of structure-specific data is acceptable, if substantiated by the licensed design professional. For these assessment criteria, the demand-to-capacity ratio relations given by Eq. (R4.5.3a) or (R4.5.3b) may be used in the assessment depending on whether or not the current building code demand exceeds the original building code demand by more than 5 percent.

CODE

4.5.4 If the concrete design criteria of the original building code used allowable stress design, the demand-to-capacity ratio of the member, system, or structure of the work area shall be evaluated based on allowable stress design load combinations for demand (U_s) and resistance calculated using allowable stresses (R_a). Strengthening repairs are required if the demand-to-capacity ratio is greater than 1.0, as given by Eq. (4.5.3)

$$U_s/R_a > 1.0 \quad (4.5.4)$$

4.5.4.1 Repair of the existing concrete structure shall be permitted to strengthen a member or system of the work area to the capacity required by the original building code based on the material properties of the original construction. New concrete members and connections to existing construction that are part of the repair shall in accordance with 4.2.3.

4.5.4.2 If the demand-to-capacity ratio given by Eq. (4.5.4) is 1.0 or less, strengthening repairs are not required.

4.5.5 The design-basis code criteria for existing structures other than those to be strengthened according to 4.3 through 4.5 shall be in accordance with 4.6 through 4.9.

4.6—Conditions of deterioration, faulty construction, or damage less than substantial structural damage without strengthening

4.6.1 If less-than-substantial structural damage is present, structures damaged, deteriorated, or containing faulty construction that do not require strengthening in accordance with 4.5 shall use the provisions of Chapters 7 through 10 as the design-basis code criteria.

4.7—Additions

4.7.1 If additions are made that change the gravity load demands on the member, system, or structure of the work area, the demand shall be determined for

(a) The configuration prior to the addition using the original building code loads and load combinations, and

(b) The configuration with the addition using the current building code loads and load combinations.

4.7.2 For the existing elements of the work area required to support gravity loads of the addition, if the gravity load demands of the current building code with the addition are more than 5 percent greater than the demands of the original building code without the addition, the *design-basis code* for the addition shall be the current building code, and this Code shall be used for the existing member, system, or structure of the work area.

COMMENTARY

R4.5.4 Before the adoption of ACI 318-63, the design of reinforced concrete structures was based upon allowable stress design principles. Original building code demands should include nominal gravity loads (dead, live, and snow) using the load combinations of the original building code. Consideration should be given to including measured displacements (sagging, listing, leaning, and tilting), second-order effects, and other loads specific to the work area, such as drifting snow, lateral earth pressures, self-straining loads, ice, and floods. Using allowable stress design is inconsistent with the reliability principles of strength design. To adequately address current safety levels, consideration should be given to verification of safety using the strength design provisions of 4.5.2. It may be appropriate to consider detailing requirements for ductility if considering seismic resistance.

R4.6—Conditions of deterioration, faulty construction, or damage less than substantial structural damage without strengthening

R4.6.1 Serviceability requirements including deflection limits and reinforcement for cracking are given in the current building code are not requirements of this Code but should be considered in the assessment and repair of existing structures.

R4.7—Additions

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CODE

4.7.3 For the member, system, or structure of the work area supporting gravity loads whose calculated capacity using this Code is to be decreased as part of an addition, the elements of the work area shall be shown to have or be rehabilitated to have a demand-to-capacity ratio equal to or less than 1.0 for the current building code demand.

4.7.4 If the addition is not independent of the existing building for lateral-force resistance, the design-basis code for the existing lateral-force-resisting system with the addition shall be the current building code.

4.7.4.1 The licensed design professional shall be permitted to use the original building code load demands and capacities if the demand-to-capacity ratio for each lateral-force-resisting member, with the addition and the current building code demand, does not exceed the demand-to-capacity ratio without the addition using the original building code demand increased by 10 percent.

COMMENTARY

R4.7.4.1 This exception permits the licensed design professional to use the original building code for the design-basis criteria of existing lateral-force-resisting members if the members of the existing lateral-force-resisting system comply with Eq. (R4.7.4.1).

$$U_c/R_n \text{ (with addition)} \leq 1.1U_o/R_n \text{ (without addition)} \quad \text{(R4.7.4.1)}$$

4.8—Alterations

4.8.1 If alterations are made that change the gravity load demands on the member, system, or structure in the work area of the existing structure, the demand shall be determined for

1) The configuration prior to the alterations using the original building code loads and load combinations, and

2) The configuration with the alterations using the current building code loads and load combinations.

4.8.2 For the existing elements in the work area required to support gravity loads of the alteration, if the gravity load demands of the current building code with the alterations are more than 5 percent greater than demands of the original building code without the alterations, the design-basis code criteria shall be the current building code, and this Code shall be used for the existing member, system, or structure within the work area.

4.8.3 When the capacity of the member, system, or structure in the work area of the existing structure supporting gravity loads is to be reduced as part of the alteration, the reduced capacity shall not be less than the current building code demand.

4.8.4 If the alteration increases design lateral loads, results in a structural irregularity according to ASCE/SEI 7, or decreases the capacity of lateral-force-resisting system, the design-basis code criteria for elements of the work area shall be the current building code.

R4.8—Alterations

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CODE

4.8.4.1 It shall be permitted to use as an alternate, the load demands and capacities from the original building code for any lateral-force-resisting member, system, or structure in the work area if the demand-to-capacity ratio with alterations using the current building code is not more than the demand-to-capacity ratio without alterations using the original building code increased by 10 percent.

4.9—Change of occupancy or use

4.9.1 If the change of occupancy or use of the existing structure changes demand on the member, system, or structure of the work area, the demand shall be determined using:

(a) the original building code loads and load combinations; and

(b) the current building code loads and load combinations.

4.9.2 For the existing elements of the work area if demand based on the current building code is greater than the demand on those elements based on the original building code, the affected elements shall be strengthened using the current building code demands and the capacities based on this Code, and the lateral-force-resisting system shall be evaluated and strengthened if necessary to meet the Basic Performance Objective for Existing Buildings, as defined in ASCE/SEI 41. New concrete members and connections to existing construction shall be in accordance with 4.2.3.

COMMENTARY

R4.8.4.1 This exception permits the licensed design professional to use the original building code for the assessment and design-basis code criteria of existing lateral-force-resisting members if the members of the existing lateral-force-resisting system comply with Eq. (R4.8.4.1).

$$U_c/R_n \text{ (with alterations)} \leq 1.1 U_o/R_n \text{ (without alterations)} \quad \text{(R4.8.4.1)}$$

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CODE

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

5.1—General

5.1.1 If this code is part of the design-basis code, the load factors, load combinations, and strength reduction factors in this chapter shall be used for the assessment of the existing structure and the design of rehabilitation.

5.1.2 It shall not be permitted to use load factors and load combinations from the original building code with strength reduction factors from this chapter. It shall not be permitted to use load factors and load combinations from this chapter with strength reduction factors from the original building code.

5.1.3 For work areas subjected only to construction activity and not subjected to design occupancies, it shall be permitted to determine factored loads on only those areas in accordance with ASCE/SEI 37.

5.1.4 When assessing an existing structure, consideration shall be given to effects caused by loads or imposed deformations that the structure is subjected to, if required by the authority having jurisdiction, even if such effects may not have been specified in the original building code.

5.2—Load factors and load combinations

5.2.1 Design of rehabilitation shall account for existing loads and imposed deformations of the structure; the effects of load redistribution due to damage, deterioration, or load removal; and the sequencing of load application, including construction and shoring loads, during the rehabilitation process.

5.2.2 Structural assessment shall consider whether the design strengths of members and connections in the work area are sufficient to resist the factored load combinations required by this Code. In the rehabilitation design, the structural members and connections shall have design strengths at least equal to the required strengths calculated for the factored load combinations as required by this Code.

5.2.3 Required strength U shall be at least equal to the effects of factored load combinations as specified in the design-basis code.

5.2.4 Required strength U shall include internal load effects due to reactions induced by prestressing with a load factor of 1.0.

COMMENTARY

R5—LOADS, FACTORED LOAD COMBINATIONS, AND STRENGTH REDUCTION FACTORS

R5.1—General

R5.1.1 Load factors, load combinations, and strength reduction factors are intended to achieve consistent acceptable levels of safety among all the structural elements in a system. They are obtained through rational design code calibration procedures that consider the accuracy of the strength prediction models and on the expected loads during the design service life of the structure.

R5.1.2 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 safety.

R5.1.3 This provision permits the less stringent loads and load factors in ASCE/SEI 37 to be applied for areas designated for construction activity and not subjected to design occupancies.

R5.1.4 Examples of such loads include vibration or impact loads. Examples of such imposed deformations include unequal settlement of supports, sagging, and listing, leaning and tilting, and those due to prestressing, shrinkage, temperature changes, and creep.

R5.2—Load factors and load combinations

R5.2.2 The basic requirement for strength design or assessment is expressed as:

design strength (for example, capacity) ≥ required strength (for example, demand)

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

The design strength is the nominal strength multiplied by the strength reduction factor ϕ.

R5.2.3 The required strength U is expressed in terms of factored loads, which are the product of specified nominal loads multiplied by load factors.

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CODE

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

5.3—Strength reduction factors for rehabilitation 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 shown in Table 5.3.2.

Table 5.3.2—Maximum strength reduction factors for rehabilitation design

Strength	Classification	Transverse reinforcement	ϕ
Flexure, axial, or both	Tension-controlled*		0.90
	Compression-controlled†	Spirals‡	0.75
		Other	0.65
Shear, torsion, or both			0.75
Interface shear			0.75
Bearing on concrete§			0.65
Post-tensioned anchorage zones			0.85
Struts, ties, nodal zones and bearing areas in strut-and-tie models			0.75

*Applies when the steel tensile strain at member failure exceeds $2.5\varepsilon_y$, where ε_y is the yield strain of the tensile reinforcement.

†Applies when the steel tensile strain at member failure does not exceed ε_y. For sections in which the net tensile strain in the extreme tension steel at nominal strength is between the limits for compression-controlled and tension-controlled sections, linear interpolations of ϕ shall be permitted.

‡Spirals shall satisfy 10.7.6.3, 20.2.2, and 25.7.3 of ACI 318-14.

§Does not apply to post-tensioned anchorage zones or elements of strut-and-tie models.

5.3.3 Computation of 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 assessment

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

COMMENTARY

R5.2.5 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. For jacking loads less than the maximum tendon jacking force, or for jacking loads applied to 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.

R5.3—Strength reduction factors for rehabilitation design

R5.3.2 For a steel yield strength of 60 ksi, 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 tension- and compression-controlled sections are 0.375 and 0.6, respectively. The 0.6 limit for compression-controlled sections applies to sections reinforced with Grade 60 steel and to prestressed sections. For other grades of steel reinforcement, the term c/d_t is a function of the yield strain of the steel reinforcement (ε_y). The c/d_t ratio is calculated as $c/d_t = 0.003/(0.003 + \varepsilon_y)$.

R5.4—Strength reduction factors for assessment

R5.4.1 Strength reduction factors given in 5.4.1 are larger than those in 5.3.1. These increased values are justified by the

CODE

Chapter 6, and material properties are determined in accordance with 6.4, it shall be permitted to increase ϕ from those specified in 5.3, but ϕ shall not exceed the values shown in Table 5.4.1.

Table 5.4.1—Maximum strength reduction factors for assessment

Strength	Classification	Transverse reinforcement	ϕ
Flexure, axial, or both	Tension-controlled*		1.00
	Compression-controlled†	Spirals‡	0.90
		Other	0.80
Shear, torsion, or both			0.80
Interface shear			0.80
Bearing on concrete§			0.80
Struts, ties, nodal zones, and bearing areas in strut-and-tie models			0.80

*Applies when the steel tensile strain at member failure exceeds $2.5\varepsilon_y$, where ε_y is the yield strain of the tensile reinforcement.

†Applies when the steel tensile strain at member failure does not exceed ε_y. For sections in which the net tensile strain in the extreme tension steel at nominal strength is between the limits for compression-controlled and tension-controlled sections, linear interpolations of ϕ shall be permitted.

‡Spirals shall satisfy 10.7.6.3, 20.2.2, and 25.7.3 of ACI 318-14.

§Does not apply to post-tensioned anchorage zones or elements of strut-and-tie models.

5.4.2 If an evaluation of members is based on historical material properties as given in Tables 6.3.2a through 6.3.2c, the ϕ-factors not exceeding those in 5.3 shall apply.

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

5.5—Additional load combinations for structures rehabilitated with external reinforcing systems

5.5.1 For rehabilitation achieved with external reinforcing systems that are susceptible to damage by vandalism or impact, the required strength of the structure without rehabilitation shall equal or exceed the effects of the load combinations specified in 5.5.2. The performance of externally reinforced elements subjected to fire shall be evaluated using the load combinations specified in 5.5.3.

COMMENTARY

improved reliability due to the use of accurate field-obtained material properties, and actual in-place dimensions. They have been deemed appropriate for use in ACI 318 and have had a lengthy history of satisfactory performance.

R5.4.3 The resistance factor for assessment of plain concrete is the same as that specified for design in 5.3.4. Material properties for plain concrete determined in accordance with 6.3.5 may increase its nominal resistance, but the strength reduction factor remains unchanged because plain concrete failures are usually brittle.

R5.5—Additional load combinations for structures rehabilitated with external reinforcing systems

R5.5.1 The additional load combinations specified in this section are intended to ensure adequate strength should the external reinforcing system be sufficiently damaged to become ineffective. External reinforcing systems should be evaluated to determine if they are susceptible to damage from accidental vehicular impact or vandalism. Alternately, the rehabilitation measures may include physical design features that protect the external reinforcing system from these types of damage. The requirements of this section are not intended for the assessment of the effect of blast loadings, blast effects or a generalized assessment of extraordinary events on structures.

The requirements of this section are not intended for the design of structures that are exposed to elevated tempera-

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CODE

5.5.2 For external reinforcing systems susceptible to damage, the required strength of the structure without such external reinforcement shall satisfy Eq. (5.5.2a) and (5.5.2b)

$$\phi R_n \geq 1.1D + 0.5L + 0.2S \qquad (5.5.2a)$$

$$\phi R_n \geq 1.1D + 0.75L \qquad (5.5.2b)$$

where D, L and S are the effects due to the dead, live, and snow loads, respectively, calculated for the rehabilitated structure; ϕ is the strength reduction factor in 5.3 or 5.4, as applicable; and R_n is the nominal strength of the structural member computed using the material properties determined from Chapter 6, without the contribution of the external reinforcing system.

5.5.2.1 If the live load has a high likelihood of being a sustained load, the live load factor in Eq. (5.5.2a) and (5.5.2b) shall be increased to 1.0.

5.5.3 Structural members with external reinforcement shall satisfy Eq. (5.5.3)

$$\phi_{ex} R_{ex} \geq (0.9 \text{ or } 1.2)D + 0.5L + 0.2S \qquad (5.5.3)$$

where ϕ_{ex} = 1.0; R_{ex} is the nominal resistance of the structural member, computed using the probable material properties during the fire event and considering the contribution of external reinforcement in accordance with Section 5.5.3.3; and S is the specified snow load. The dead load factor of 0.9 shall be applied when the dead load effect counteracts the total load effect.

5.5.3.1 Additional live loads incurred during a fire shall be considered, with a load factor of 1.0.

5.5.3.2 Internal forces and imposed deformations due to thermal expansion during the fire event shall be considered, with a load factor of 1.0, in determining the demands on the structural system.

COMMENTARY

tures during routine service, such as manufacturing facilities and other industrial environments. Refer to Section 7.9.4.

R5.5.2 These load combinations are intended to minimize the risk of failure of the strengthened structural member in the case where, during normal operating conditions, the external reinforcement is damaged. Such damage may not be detected immediately and so the structure (or structural component) may remain in service until the damage is identified and addressed. The load factors for live and snow loads in Eq. (5.5.2a) correspond to the arbitrary point-in-time loadings specified in ASCE/SEI 7. Equation (5.5.2b) is compatible with ACI 440.2R.

R5.5.2.1 Examples include library stack areas, heavy storage areas, warehouses, and other storage occupancies with a live load exceeding 100 lb/ft^2.

R5.5.3 Equation (5.5.3) is intended to ensure that the repaired element will maintain sufficient strength, accounting for its probable reduced material properties due to elevated temperatures, during a fire event. If fire protection is applied to the repaired element, its effect on the external reinforcement and existing elements should be considered.

The design-basis code and ACI 216.1 should be reviewed to determine the required duration of the fire event.

Equation (5.5.3) was developed from Eq. (2.5.1) of ASCE/SEI 7-10 and is limited to the evaluation of fire effects on structures with external reinforcement. Guidance on computing the structural effects caused by the fire event is provided in 5.5.3.1 and R5.5.3.1.

Strength of the affected portion of the structure during a fire event should be based on reduced steel and concrete strengths. Guidance concerning probable material properties during a fire event may be obtained from ACI 216.1.

R5.5.3.1 Live loads associated with firefighting operations may include wetting of the building contents, which can be idealized as a live load of 20 lb/ft^2.

R5.5.3.2 Thermal expansion of a member during a fire event will generate internal thrust forces if that expansion is restrained. The generated thrust force, while potentially large, is considerably less than that computed using conventional elastic properties and thermal expansion coefficients. This thrust may increase the moment capacity and the corresponding fire endurance of the restrained member.

Procedures for calculating thermal induced thrust forces can be found in NIST (2010) and Buchanan (2001). PCI (2010) provides methods for determining (a) the magnitude and location of the thrust generated by a given fire tempera-

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CODE

COMMENTARY

ture and duration, and (b) the increase in moment capacity caused by a known thrust force.

5.5.3.3 Any contribution of external reinforcement that is not protected using a fireproofing system shall be neglected during a fire event. The contribution of any adhesively bonded external reinforcement to the strength of a member during a fire shall be ignored.

R5.5.3.3 Section 7.9 gives member strength requirements for protected and unprotected external reinforcing systems subjected to elevated temperatures during a fire event.

5.5.3.4 When the live load acting on the member to be strengthened has a high likelihood of being a sustained load, a live load factor of 1.0 shall be used in Eq. (5.5.3).

R5.5.3.4 Refer to R5.5.2.1.

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CHAPTER 6—ASSESSMENT, EVALUATION, AND ANALYSIS

6.1—Structural assessment

6.1.1 A structural assessment shall be performed if required per 1.7.5. The structural assessment shall comprise 1) an investigation to establish the in-place condition of the structure in the work area, including environment, geometry, material strengths, reinforcing-steel sizes and placement, and signs of distress; 2) an evaluation to define the causes of distress and criteria for selection of rehabilitation solution(s); and 3) development of appropriate rehabilitation strategies.

6.2—Investigation and structural evaluation

6.2.1 An investigation and structural evaluation shall be performed when there is a reason to question the capacity of the structure in the work area and insufficient information is available to determine if an existing structure is capable of resisting design demands.

6.2.2 Where repairs are required to an individual member or connection in a structure, determine if similar conditions exist beyond the work area also require evaluation.

6.2.3 An investigation shall document conditions as necessary to perform an evaluation of the structure in the work area.

COMMENTARY

R6—ASSESSMENT, EVALUATION, AND ANALYSIS

R6.1—Structural assessment

R6.1.1 Field investigations in support of the structural assessment may include visual observations, destructive testing, and nondestructive testing (NDT). Areas of known deterioration and distress in the structural members should be identified, inspected, and recorded as to the type, location, and degree of severity. Investigation procedures are referenced in ACI 201.1R, ACI 228.1R, ACI 228.2R, ACI 364.1R, ACI 437R, ASCE/SEI 11, ASCE/SEI 41, and FEMA P-154. The affected structural members are not only members with obvious signs of distress but also contiguous members and connections in the structural system.

The data gathered to determine the existing capacity should include the following:

(1) The effects of material deterioration, such as loss of concrete strength from chemical attack; freezing and thawing

(2) Loss of steel area due to corrosion or other causes

(3) Missing or misplaced reinforcement

(4) Effects of damaging events, such earthquakes or fire

The effect of deterioration on the ductility of the member should be considered in the evaluation. The strength or serviceability of a member or structure may be compromised by spalling, excessive cracking, large deflections, or other forms of damage or deterioration. Seismic evaluation references for undamaged buildings include FEMA P-58, FEMA P-154 and ASCE/SEI 41 and for damaged buildings include FEMA 306 and FEMA 307.

Where the as-built conditions and properties of historical buildings require evaluation and rehabilitation, care should be taken to minimize the impact of repair design and investigation procedures (U.S. Department of the Interior 1995).

R6.2—Investigation and structural evaluation

R6.2.2 If there is no evidence of damage, distress or deterioration of similar members or connections elsewhere in the work area that required repair, there is no need to perform an evaluation of similar members unless potentially dangerous conditions are present. Such conditions may be a concern if there are significant variances from the original design intent such as lower-strength concrete or insufficient reinforcement. In addition, if the similar members are in an environment that could foster deterioration, then evaluation of these members may be necessary to determine if strengthening or durability enhancements may be required.

R6.2.3 Conditions which may need to be documented include (a) through (g):

CODE

6.2.4 When an analysis is required, the analysis shall be performed in accordance with Section 6.5 and the structural evaluation shall consider the following items.

(a) As-measured structural member dimensions including the reinforcement configuration

(b) The presence and effect of alterations to the structural system

(c) Loads, occupancy, or usage different from the original design

(d) In-place material properties in accordance with 6.3

6.3—Material properties

6.3.1 Concrete compressive strength and steel reinforcement yield strength shall be determined for the structure if a structural evaluation is required. Nominal material properties shall be determined by (a), (b) or (c):

(a) Available drawings, specifications, and previous testing documentation

(b) Historical material properties in accordance with 6.3.2

(c) Physical testing in accordance with 6.4

COMMENTARY

(a) The physical condition of the structural members to examine the extent and location of deterioration or distress

(b) The adequacy of continuous 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; 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) Information needed to assess 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 that the material properties obtained from record documents are accurate. Material testing may be required to verify these values.

R6.2.4 The size, number, and location of the reinforcement may be needed to determine member strength. Nondestructive testing methods, including but not limited to ground-penetrating radar (GPR) and ferromagnetic testing, may be used to determine the location and spacing of reinforcement. These methods may require destructive confirmation. Additional guidance is provided in ACI 228.2R.

R6.3—Material properties

R6.3.1 The construction documents may not represent as-built conditions. Therefore, the evaluation of material properties may require verification by material testing to confirm that the material properties obtained from record documents are representative.

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

(a) Ductility based on the mechanical characteristics of the component materials.

(b) Presence of corrosion of embedded steel reinforcement, including carbonation, chloride intrusion, corrosion-induced spalling

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

(d) Deterioration due to cyclic freezing and thawing

(e) Deterioration of stiffness and strength due to bar slip in cracked sections and joints damaged in seismic events

Other tests for material properties, including petrographic examination, are used.

CODE

COMMENTARY

The choice of tests depends on the structure, member type(s), and distress mechanism.

6.3.2 If available drawings, specifications, or other documents do not provide sufficient information to characterize the material properties, it shall be permitted to determine such properties without physical testing from the historical data provided in Tables 6.3.2a through 6.3.2c.

R6.3.2 Material properties required for seismic evaluation and rehabilitation are discussed in ASCE/SEI 41. The required material properties may include necessary physical and chemical properties of the concrete and reinforcement, and should include the required references to ASTM standards and other methods of determining physical and chemical properties.

Table 6.3.2a—Default compressive strength of structural concrete, psi

Time frame	Footings	Beams	Slabs	Columns	Walls
1900-1919	1000	2000	1500	1500	1000
1920-1949	1500	2000	2000	2000	2000
1950-1969	2500	3000	3000	3000	2500
1970-present	3000	3000	3000	3000	3000

Note: Adopted from ASCE/SEI 41.

Table 6.3.2b—Default tensile and yield strength properties for steel reinforcing bars for various periods*

		Structural†	Intermediate†	Hard†				
	Grade	33	40	50	60	65	70	75
	Minimum yield, psi	33,000	40,000	50,000	60,000	65,000	70,000	75,000
Year	Minimum tensile, psi	55,000	70,000	80,000	90,000	75,000	80,000	100,000
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	—

Note: Adopted from ASCE/SEI 41.

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

†The terms "structural," "intermediate," and "hard" became obsolete in 1968. Hard grade does not correspond to metallurgical hardness.

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COMMENTARY

Table 6.3.2c—Default tensile and yield strength properties of steel reinforcement for various ASTM specifications and periods*

				Structural†	Intermediate†	Hard †				
			Grade	33	40	50	60	65	70	75
			Minimum yield, psi	33,000	40,000	50,000	60,000	65,000	70,000	75,000
ASTM designation‡	**Steel type**	**Year range**	**Minimum tensile, psi**	55,000	70,000	80,000	90,000	75,000	80,000	100,000
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
A616-96‖	Rail	1968-present		—	—	—	X	—		—
A617	Axle	1968-present		—	X	—	—	—		—
A706#	Low-alloy	1974-present		—	—	—	X	—	X	—
A955	Stainless	1996-present		—	X	—	X	—		X

Note: Adopted from ASCE/SEI 41.

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

†The terms structural, intermediate, and hard became obsolete in 1968. Hard grade does not correspond to metallurgical hardness.

‡ASTM steel is marked with the letter W.

§Rail bars are marked with the letter R.

‖Bars marked with "s!" (ASTM A616-96) have supplementary requirements for bend tests.

#ASTM A706 has a minimum tensile strength of 80 ksi, but not less than 1.25 times the actual yield strength.

6.3.3 It shall be permitted to determine material properties through testing in accordance with 6.4.

6.3.4 The material properties provided in the original construction documents or material test reports shall be permitted to be used unless known deterioration that can affect performance has occurred.

R6.3.4 If the results of material testing from original construction are available, these results may be used in the analysis. Additional testing could be required to confirm these material test results if deterioration has occurred.

6.3.5 If historic data are not given in either Table 6.3.2b or 6.3.2c, the historic default value for yield strength f_y shall be taken as 33,000 psi.

R6.3.5 Additional guidance regarding the use of the historic lower bound default value is given in R6.4.4.1.

6.4—Test methods to quantify material and member properties

6.4.1 *General*

R6.4—Test methods to quantify material and member properties

R6.4.1 *General*

6.4.1.1 Destructive and nondestructive test methods used to obtain in-place mechanical properties of materials and member properties shall be in accordance with this section. Compressive strength of sound concrete shall be determined

CODE

by taking and testing core samples or by a combination of cores and by the use of site specific nondestructive testing. Steel reinforcement properties shall be determined by removal of reinforcement samples and destructive testing.

6.4.1.2 The locations and numbers of material samples shall be sufficient to define the material properties of the structural element of concern. The number of samples shall be determined during evaluation.

6.4.2 *Core sampling of concrete for testing*

6.4.2.1 It shall be permitted to determine the compressive strength of sound concrete by taking cores from the members being evaluated. Steel reinforcement shall be located before locating the cores to be extracted.

6.4.3 *Concrete*

6.4.3.1 The cores shall be selected, removed, and tested in accordance with ASTM C42 and ASTM C823. The equivalent specified concrete strength f_{ceq} shall be calculated by:

$$f_{ceq} = 0.9\overline{f}_c\left[1-1.28\sqrt{\frac{(k_cV)^2}{n}+0.0015}\right] \quad (6.4.3.1)$$

where $\overline{f}_c$ is the average core strength, as modified to account for the diameter, length to diameter ratio and moisture condition of the core (following ASTM C42 procedures); V is the coefficient of variation of the core strengths (a dimensionless quantity equal to the sample standard deviation divided

COMMENTARY

R6.4.1.2 Review of available records from the original construction may be used to guide testing. Evaluation, 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 any effects of material deterioration, such as loss of concrete strength from chemical attack and loss of steel area due to corrosion. The impact of deterioration on the expected strength and ductility of the section also should 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.

R6.4.2 *Core sampling of concrete for testing*

R6.4.2.1 NDT may be used to locate existing reinforcement and to avoid damage to reinforcement during coring. Guidelines for core sampling and evaluating core strength data are given in ACI 214.4R. The presence of reinforcement or other foreign material, such as conduit or wood, may adversely affect the test strength of the concrete and cores containing such foreign material should not be used for strength determination. See ASTM C42/C42M Section 5.1.3 and Note 8 for additional information.

R6.4.3 *Concrete*

R6.4.3.1 The equivalent specified strength determined using this procedure can be used in strength equations with the strength reduction factors from Chapter 5. This approach is specified in the Canadian Highway Bridge Design Code (CAN/CSA S6-14) and is based on the approach proposed by Bartlett and MacGregor (1995). Equation (6.4.3.1) is a simplification of criteria given in ACI 214.4R that gives similar results because it includes the strength correction factors for length-to-diameter ratio, core diameter, and drilling damage. The strength value obtained using this procedure is an estimate of the 13 percent fractile of the in-place concrete strength at a confidence interval of 90 percent, based on the field data collected by Bartlett and

CODE

by the mean); n is the number of cores taken; and k_c is the coefficient of variation modification factor, as obtained from Table 6.4.3.1.

Table 6.4.3.1—Concrete 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

6.4.3.2 Nondestructive strength testing to evaluate in-place strength of concrete shall be permitted if a valid correlation is established with core sample compressive strength test results and nondestructive test results. Quantifications of concrete compressive strength by NDT alone shall not be permitted as a substitute for core sampling and testing.

6.4.4 *Steel reinforcement*

6.4.4.1 If the original construction documents are not available and if the properties of the reinforcing bars are unknown, historical values provided in 6.3.2b and 6.3.2c shall be permitted in place of testing. If the grade of material is unknown, the lowest grade provided in Table 6.3.2b for a given historic period shall be used.

6.4.5 *Reinforcement sampling and testing*

6.4.5.1 Coupon samples used for the determination of the yield and tensile strength for steel reinforcement shall be obtained in accordance with ASTM A370. A minimum of three sample coupons, taken from different segments of reinforcement shall be obtained from the members being evaluated.

COMMENTARY

MacGregor (1995). When a different strength fractile or confidence interval is required, the methods presented in ACI 214.4R may be applicable.

The core samples, tested per ASTM C42/42M, are expected to be moisture conditioned following the procedure in the ASTM standard. The correction factors in ASTM C42/42M were developed for lightweight and normal weight concrete with a compressive strength between 2,000 and 6,000 psi (14 MPa to 42 MPa). Core samples are assumed to have a maximum length-to-diameter ratio of 2.1. Bartlett and MacGregor (1994a) discuss the effect of higher compressive strengths on the length-to-diameter ratio.

ASTM C42/42M procedures require a minimum core diameter of 3.70 in. (94 mm), smaller diameter cores are likely to have more variability and a lower strength (Bartlett and MacGregor 1994b).

When the testing requirements of ASTM C42/42M are not met, the user should consult ACI 214.4R.

R6.4.3.2 ACI 228.1R provides information on NDT methods for evaluation of concrete compressive strength and development of statistical correlations between NDT and core test results.

R6.4.4 *Steel reinforcement*

R6.4.4.1 The age of the structure may be known but the grade of reinforcement may not be known. In this case, the lowest grade of reinforcement corresponding to the structure's age should be used. If the date of original construction is unknown, the lower bound value of f_y equal to 33,000 psi may be used instead of testing, provided it is conservative. In some instances, assuming higher yield strengths may be more conservative. Where the demand on one member is governed by the capacity of a connected member, it is appropriate to assign higher yield strengths to the connected member. For example, in seismic analysis at beam column joints, the moment strength of the columns should exceed the moment strength of the beams. When assessing this requirement it is more conservative to assume a higher yield strength for the beam reinforcement than for the column reinforcement.

R6.4.5 *Reinforcement sampling and testing*

R6.4.5.1 Often, the steel reinforcement in a structure is of a common grade and strength. Occasionally, more than one grade of steel is used, for example, smaller diameter (No. 3 and 4) stirrups and other complex bent bars were often fabricated with lower strength material than the longitudinal bars.

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COMMENTARY

CRSI (2014): "Vintage Reinforcement in Concrete Structures," contains supplemental information on mechanical properties of the reinforcement used in different construction eras.

Steel reinforcement information 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 The equivalent specified yield strength f_{yeq} of reinforcement used for analysis shall be calculated by

$$f_{yeq} = (\overline{f_y} - 3500)e^{(-1.3k_sV)} \quad (6.4.6)$$

where $\overline{f_y}$ is the average yield strength value from the tests, in psi; V is the 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.

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

R6.4.6 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 yield strength measured during a coupon test exceeds the static yield strength by 3500 psi. This approach is specified in the Canadian Highway Bridge Design Code (CAN/CSA S6-14).

The factors in Table 6.4.6 reflect the uncertainty of the sample standard deviation 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.

6.4.7 If the properties of the connector steel are unknown, strength shall be determined by (a), (b), or (c):

(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 in accordance with 6.3.6.

R6.4.7 The historic default value is obtained from ASCE/SEI 41.

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.

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

$$f_{yeq} = (\overline{f_y} - 4000)e^{(-1.3k_sV)} \quad (6.4.8)$$

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where $\bar{f}_y$ is the average yield strength value from tests, in psi; V is the 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 existing reinforcement is required, carbon equivalent shall be determined in accordance with AWS D1.4/D1.4M.

6.5—Structural analysis of existing structures

6.5.1 The gravity and lateral-force-resisting structural systems shall be analyzed when required using loads and load combinations determined in accordance with this code that produce the maximum effects on the existing members being evaluated.

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 material deterioration, bond loss, and the redistribution of forces in members and in the structural system as a whole.

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-dimen-

COMMENTARY

R6.5—Structural analysis of existing structures

R6.5.1 Structural evaluation and analyses are conducted to verify strength and serviceability. The analytical methods of 6.5 are used with factored loads to determine strength requirements for a combination of flexure, shear, torsion, and axial loads of pertinent structural members. A service-load analysis may be required to evaluate serviceability issues such as deflection and cracking.

R6.5.3 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, second-order and other nonlinear effects should be included in the analysis.

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.

R6.5.4 Member deterioration and damage may result in distribution of internal forces different than the distribution of forces of the original structural design. The strength and integrity of prestressed structures with damaged prestressing reinforcement requires careful consideration to assess the impact of the damage. The state of the structure should be accurately modeled to determine the distribution of forces. Redistribution of forces may be determined using material nonlinear analysis, by load tests described in ACI 437.2, or by linear analysis, which bounds the limits of redistributed forces.

R6.5.5 The evaluation of load effects requires consideration of both the load paths through the structure and how the forces are distributed in members.

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sional 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 determination of in-place material properties shall be in accordance with 6.3.

6.6—Structural serviceability

6.6.1 If serviceability problems are identified during the preliminary evaluation or the structural assessment, the licensed design professional shall perform a serviceability evaluation based on the existing geometry and properties of the structure.

6.6.2 The serviceability evaluation shall evaluate the structure for the intended use considering the existing geometry and properties of the structure, and shall consider such effects as existing floor levelness, support displacements, vibrations, and deflections.

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 the anticipated phases of the evaluation and repair process.

COMMENTARY

R6.5.6 Modifications to structures in the form of repairs, alterations, or additions may affect the force distribution and load path in a structure.

R6.5.7 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. Additional information may be obtained in ACI 364.10T.

R6.6—Structural serviceability

R6.6.1 Structural serviceability problems may include deflections, floor levelness, vibrations, leakage, or cracking. The data gathered to determine serviceability should include the effects of material deterioration, such as loss of concrete strength from sulfate attack or loss of steel area due to corrosion. Reference ACI 224.1R for additional information on cracking, including causation and repair.

R6.6.2 When specific concerns are raised regarding the serviceability of the structure, the effect of floor levelness, vibrations, and deflections on the structural performance should be investigated by the licensed design professional. The floor levelness, vibrations, and deflections should indicate (or be assessed to determine) if the performance of the structure is acceptable. Acceptable performance criteria will need to be established for an individual structure based upon the intended use of the structure.

The specific performance criteria and the intended function of the structure should be considered. Floor deflection criteria for new structures can be found in ACI 318. Vibration criteria are given in Fanella and Mota (2014).

Information on construction tolerances for new concrete construction is presented in ACI 117.1R; however, some of the tolerances only apply to measurement made during construction and therefore may not be appropriate to be used for existing completed construction. Refer to ATC Design Guide 1 (1999), Fanella and Mota (2014), and Wilford and Young (2006) for information on evaluation of vibration problems in concrete structures.

R6.7—Structural analysis for repair design

R6.7.1 The construction process may involve the application, 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 anticipated construction phases. The additional applied loads may be due to prestressing, vibration, material volume changes (such as creep and shrinkage, or tempera-

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6.7.2 Structural analysis shall account for variations in material properties through member sections.

6.7.3 Section analysis shall use principles of mechanics and shall assume either (a), (b), or (c) as deemed appropriate by the licensed design professional:

(a) Full composite action with no slip at interfaces between repair materials and existing materials

(b) Separate action with full slip between repair and existing materials

(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 there is no adverse effect on the seismic-force-resisting system.

6.7.4.3 The 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 437.2 shall be permitted to supplement an analysis or to demonstrate the strength of the original or repaired structure.

COMMENTARY

ture changes), effect of shoring, and unequal deformation of supports.

R6.7.2 The intent of this section is to address differences in stiffness between repair material and existing substrate. In these situations, localized deformation may occur in the material with the lower modulus of elasticity, affecting the force distribution in the repaired structure.

R6.7.3 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 model the anticipated degree of composite action of the 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.

R6.7.4 *Seismic analysis of repaired structure*

R6.7.4.3 Procedures for seismic rehabilitation of concrete buildings, including analysis, are provided in ASCE/SEI 41, which incorporates ACI 369R. These references provide details for forces, rehabilitation methods, analysis and modeling procedures, and seismic rehabilitation design. Additional references for repair of building damage by a seismic event and rehabilitation of concrete buildings include FEMA 308, FEMA 395 through FEMA 400, and FEMA 547.

R6.8—Strength evaluation by load testing

R6.8.1 Information obtained during a structural assessment may be insufficient to determine the strength or serviceability of deteriorated or repaired structural members. Structural condition assessments, including destructive testing, can provide some of the information required, but the costs for these assessments can be significant. Further, the results of a structural evaluation may still be inconclusive due to unknown effects of existing conditions or interaction with the repair. In such cases, load testing may provide the most effective means of verifying the strength of a structure or member. Load testing can also be a valuable tool for evaluating the effectiveness of structural repairs. For example, load testing, as defined in ACI 437.2, can be performed to determine if the service load deflection and cracking are acceptable.

CODE

6.8.2 Load tests shall be conducted in accordance with the monotonic or cyclic procedures in ACI 437.2.

6.8.3 The design professional is permitted to waive the $\ell_t/180$ deflection criteria in ACI 437.2.

6.8.4 If a member fails a cyclic load test, it shall be permitted to retest the member or structure in accordance with ACI 437.2. It shall be permitted to waive the maximum deflection limit ($\ell_t/180$) in ACI 437.2 that precludes a retest.

6.8.5 Model analysis shall be permitted to supplement calculations.

COMMENTARY

R6.8.2 If the strength of the structure being evaluated is limited by the strength of concrete, or the expected failure of the structure is controlled by shear or development of the reinforcement, the sustained load applied using the monotonic test allows greater time for widening and propagation of cracks, creep, and slip of reinforcement compared with the cyclic procedure.

R6.8.3 The $\ell_t/180$ deflection limit was included to provide an upper limit on the deflection of a member during a load test. The deflection limit may be waived by the design professional when the tested member is not damaged by large deflections or when the residual deflection criteria is satisfied.

R6.8.4 ACI 437.2 precludes a retest if the member exceeds a maximum deflection limit of $\ell_t/180$ (Section 6.4.4.2 in ACI 437.2). For consistency with the monotonic testing protocol, this $\ell_t/180$ limit is waived.

R6.8.5 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. References are provided in Harris and Sabnis (1999) and White (1970).

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CHAPTER 7—DESIGN OF STRUCTURAL REPAIRS

7.1—General

7.1.1 Repaired structural elements and connections within the work area shall have design strengths at all sections at least equal to the required strengths calculated using the applied factored loads and internal forces in such combinations as required by this code.

7.2—Strength and serviceability

7.2.1 Repaired structures shall be designed to meet the strength requirements of the design-basis code.

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

7.2.3 When serviceability issues are identified consistent with Section 6.6, repairs to address serviceability issues shall be considered.

7.3—Behavior of repaired systems

7.3.1 Repairs to sections, components, reinforcement, connections of members, or systems shall be designed to be integrated with the existing structure, creating a structural system capable of resisting the design loads by sharing and transferring loads between repaired and existing elements.

7.3.1.1 Repairs incorporating new members shall be designed to be integrated with the existing structure, creating

COMMENTARY

R7—DESIGN OF STRUCTURAL REPAIRS

R7.1—General

Repair and rehabilitation, as defined in Chapter 2, are processes in which deficiencies and damage in a structure or member are corrected. The methods used to correct deficiencies and damage in structures will be the same for both repair and rehabilitation projects. For the purposes of this chapter, design requirements for repair and rehabilitation can be considered to be equivalent.

Durability requirements for repairs are in Chapter 8.

R7.1.1 Internal forces include those from externally applied loads and those from imposed deformations, from such actions as prestressing, shrinkage of repair materials, temperature changes, creep, unequal settlement of supports, and listing, leaning and tilting displacements.

R7.2—Strength and serviceability

R7.2.2 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 deformations present in the structure during the repair and the subsequent internal forces from the design loads that the repaired section will resist. Internal forces and deformations caused by existing loads may be locked in by the repair.

Analysis to evaluate the effects of structural modifications should verify that the strength is adequate and that 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 of the affected elements. Slab punching shear strength should be evaluated for openings at the intersection of column strips to verify that the slab is adequate. This is especially critical near corner and edge columns where the slab shear stress is typically highest.

R7.2.3 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.

R7.3—Behavior of repaired systems

R7.3.1.1 Repair of a structure may be achieved by improving the global behavior of the structure by adding

CODE

a structural system capable of resisting the design loads by sharing and transferring loads between new members and existing elements.The effect of the new members on the structure shall be evaluated according to the design-basis code.

COMMENTARY

new structural members that act integrally with the existing structural system or improving the behavior of the existing members. The design of the repair should consider connections of new members to the structure. Connections of new members should be designed to transfer design forces between new members and the structure.

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

New members may need to be separated from adjacent members to prevent or minimize interaction that 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 structural system.

7.3.2 Repairs to members shall account for force transfer at the interface between the member and the repair material or repair system. It shall be permitted to use ACI 318 to design the force transfer mechanism between new and existing concrete.

R7.3.2 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 two components.

The requirements for composite behavior between the repair and the member may vary depending on the type of repair (structural or nonstructural), the performance criteria at service, and the required strength at the 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 318-14, Sections 16.4 and 22.9, for force transfer requirements between new and existing concrete. Techniques other than shear-friction may be acceptable.

Design guidelines for bond of fiber-reinforced polymer (FRP) are provided in ACI 440.1R and 440.2R. Design provisions to achieve composite behavior with structural steel sections are provided in the "Specification for Structural Steel Buildings" (ANSI/AISC 360-16, Chapter I).

7.3.3 Structural repairs required for strength or stiffness shall maintain composite behavior under service load. The repaired system shall be designed to mitigate potentially dangerous conditions if bond between the repair and the substrate is lost.

R7.3.3 Nonstructural repairs intended to improve durability or aesthetics may not require composite behavior under service loads. To prevent potentially dangerous conditions in the event of bond failure in a repair, the repair should encapsulate existing steel reinforcement. Alternately, the repair systems should be designed to provide redundant attachment of the repair material to the existing structure.

7.4—Interface bond of cementitious repair materials

7.4.1 Repair design shall include an analysis to determine the interface shear and tension stresses across bonded interfaces between cementitious repair materials and the existing substrate. The interface analysis shall use factored loads so

R7.4—Interface bond of cementitious repair materials

R7.4.1 The forces acting on the interface between cementitious repair materials and existing substrate can include tension, shear, or a combination of tension and shear depending on repair geometry and the applied loads. The

CODE

addition to internal forces resulting from restrained volume change to calculate the resultant interface stress demand (v_u) from the transfer of tension and shear.

7.4.1.1 Interface shear stress shall be designed based on

$$v_u \leq \phi v_{ni} \tag{7.4.1.1}$$

where v_{ni} is nominal interface shear stress capacity and ϕ is the strength reduction factor determined in accordance with 5.3.2.

7.4.1.2 Testing requirements for interface bond shall be in accordance with Table 7.4.1.2.

Table 7.4.1.2—Testing requirements where v_u is partially or totally resisted by the concrete

v_u	Testing requirements
Less than or equal to 30 psi	Bond integrity testing
Greater than 30 psi	Quantitative bond strength testing unless design satisfies 7.4.5

7.4.2 If v_u does not exceed 30 psi, interface reinforcement shall not be required. Bond integrity testing as specified in the construction documents shall be performed.

7.4.3 If v_u is between 30 psi and 60 psi, interface reinforcement is not required. Quantitative bond strength testing shall be performed to verify performance. Direct tension pull-off tests (ASTM C1583/C1583M) or other similar quantitative test methods shall be specified. The frequency of tests and acceptance criteria shall be specified, but the number of tests on a project shall be at least three (3).

COMMENTARY

tensile and shear demand at an interface between a cementitious repair material and the substrate from applied loads and from volume changes that occur as a result of shrinkage or thermal movement can be calculated using principles of structural mechanics, but these calculations can be complex. Guidance on designing the interface for horizontal shear can be found in Chapter 16 of ACI 318-14, Chapter I of ANSI/AISC 360-16, and Bakhsh (2010).

Where the required nominal interface shear stress is lower than 80 psi, and where good surface preparation, placement, repair materials, and curing techniques are employed, satisfactory composite behavior will likely be achieved without interface reinforcement.

R7.4.2 The 30 psi bond stress specified by this code is based on half of a nominal shear stress of 80 psi multiplied by the strength reduction factor in 5.3.2.

A properly prepared substrate is achieved by removing existing deteriorated, damaged, or contaminated concrete. The exposed sound concrete is then roughened and cleaned to allow for adequate bond of a repair material. ICRI Guideline No. 210.3 presents a discussion of achievable tensile bond strengths, suggests a minimum value of 100 psi for less critical applications, and indicates that tensile bond test values less than 175 psi that fail at the bond interface or superficially within the existing concrete substrate may indicate a partially damaged, contaminated, or otherwise inadequate bond surface. BS EN 1504-10 suggests minimum direct tension strengths of 100 psi for nonstructural repair and 175 to 215 psi for structural repairs. Interface reinforcement may be needed if sufficient interface capacity cannot be achieved through bond.

Bond integrity testing can consist of various nondestructive qualitative test methods such as sounding in accordance with ASTM D4580/D4580M, ground-penetrating radar or impact-echo described in ACI 228.2R or ICRI Guideline No. 210.4.

R7.4.3 The 60-psi bond stress is based on a nominal shear stress of 80 psi multiplied by the strength reduction in 5.3.2

On most concrete repair projects, testing to verify the bond of cementitious repair materials to the substrate is recommended as part of a quality assurance program. Quantitative bond strength testing is required when the bond stress exceeds 30 psi and interface reinforcement is not provided. ICRI Guideline No. 210.3 provides guidance on the number

@seismicisolation

of tests that should be performed based upon the repair area and acceptance criteria.

Bond capacity has primarily been evaluated using direct tension pull-off tests, as defined in ASTM C1583/C1583M and as described in ICRI Guideline No. 210.3. In some instances, laboratory slant shear tests in accordance with ASTM C882/C882M of cores made in the lab or cores taken from mockups in the field have been used to assist the licensed design professional to make informed design decisions. Slant shear test results typically exceed direct tension pull-off test results, but the slant shear strength is greatly influenced by the compressive stress the test setup introduces across the interface and may not be directly comparable to field conditions. Typically direct shear strengths are larger than direct tension strengths. Comparisons of these tests and other tests, for the purpose of achieving adequate bond is discussed in Bakhsh (2010). It generally is adequate to assume that the repair to substrate bond will resist an interfacial shear equal to the direct tensile pull-off test result.

If failure during direct pull-off testing occurs at the bond line, it may indicate inadequate surface preparation of the concrete substrate or the substrate surface was damaged by the surface preparation method (bruising of the substrate). Modifications to the surface preparation procedures may improve the tensile bond strength. Discussion of proper methods for surface preparation can be found in ACI 546R and ICRI Guideline No. 310.2R.

7.4.4 If v_u exceeds 60 psi, interface reinforcement shall be provided.

7.4.5 If v_u is completely resisted by interface reinforcement, quantitative bond strength testing is not required.

R7.4.5 This provision provides an alternative to bond strength testing.

7.4.6 Interface reinforcement shall be designed in accordance with ACI 318.

R7.4.6 ACI 318 provides design provisions for horizontal shear transfer in composite concrete flexural members. Minimum reinforcement is required between horizontal shear stress of 60 and 375 psi (500 psi multiplied by strength reduction factor of 0.75). Where the required design horizontal shear stress is greater than 375 psi, Section 16.4.4.1 of ACI 318-14 requires design per Section 22.9 of ACI 318-14. For cases where there is a net factored tension across the interface, reinforcement should be provided and designed in accordance with ACI 318.

7.4.7 Construction documents shall specify testing requirements for interface reinforcement in the repair applications.

R7.4.7 Testing to verify the performance of the interface reinforcement to transfer horizontal shear can be performed in accordance with the recommendations contained in ACI 355.2 and 355.4. Specific requirements for testing of ties should be included in a quality assurance plan.

Direct tension testing of post-installed interface reinforcement is recommended to provide verification of the installation. Guidance for determining the number of tests and acceptance criteria of the direct tension testing is similar to principles used in developing direct tension pull-off testing requirements described in ICRI Guideline No. 210.3.

@seismicisolation

7.5—Materials

7.5.1 Materials in a structure shall be permitted to remain if such materials are performing satisfactorily.

7.5.2 Except as permitted by this code, materials permitted by the current building code for new construction shall be used. Like materials shall be permitted, provided they do not contain hazardous materials or other materials not permitted by the code for new construction.

7.5.3 Alternate materials shall be permitted following approval in accordance with 1.4.

7.5.4 Design of the repair system shall consider the properties and installation of the repair materials and systems. These include, but are not limited to: physical properties of the repair materials, type of application, adhesion, volume stability, thermal movement, durability, corrosion resistance, installation methods, curing requirements, and environmental conditions.

R7.5—Materials

R7.5.2 Hazardous materials include asbestos or other materials specifically prohibited by the current building code.

R7.5.4 Physical properties of repair materials 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 301, ACI 318, ACI 503R, ACI 503.5R, ACI 503.6R, ACI 506R, ACI 546.3R, ACI 549.1R, ICRI Guideline No. 320.2R, ICRI Guideline No. 320.3R, ICRI Guideline No. 330.1, and ICRI Guideline No. 340.1).

The design of a repair should consider the compatibility of the repair materials with the materials of the existing structure. Compatibility of repair materials and systems include volume stability, 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 is often estimated as a change in the linear dimensions of the repair and should be considered 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 repair materials has often been the cause of poor performance of repairs. Properties of repair materials should be selected considering volume stability relative to the volume stability of the existing concrete in order to reduce the probability of cracking caused by relative volume changes.

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

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

CODE | COMMENTARY

materials might not necessarily contain portland cement, but 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 Guideline No. 330.1 and ACI SP-66 for steel reinforcement.

Required properties of the repair reinforcement should be specified in the construction documents. Specified reinforcement properties are dependent on the requirements of the repair and may include physio-chemical (for example, glass transition temperature, and coefficient of thermal expansion) as well as mechanical properties (for example, ultimate strength, tensile modulus, and ultimate elongation).

7.6—Design and detailing considerations

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

7.6.2 *Concrete*—The in-place properties of the concrete, in accordance with 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 of remaining reinforcement shall be permitted to 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 considered. Where original deformations are no longer effective, reinforcing bars shall be considered as smooth bars.

7.6.3.2 Repair design shall consider the location and detailing of the reinforcement in accordance with the assessment requirements of Chapter 6.

R7.6—Design and detailing considerations

R7.6.2 The extent and cause of deterioration and the concrete strength and quality should be assessed, 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 affects the concrete compressive strength.

R7.6.3 *Reinforcement*

R7.6.3.1 Repair design should consider the in-place condition of the reinforcement, including the effective cross-sectional area of the reinforcing bars. The effective area is calculated using the remaining effective diameter of the reinforcing bar accounting for the loss of section due to corrosion. 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 be annealed, and the yield strength reduced. Refer to ACI 216.1 for additional guidance. Durability requirements related to corroded reinforcement are addressed in 8.4 and ACI 364.1R. CRSI (2014) provides information on older reinforcement systems.

R7.6.3.2 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 reinforcement may be required. Guidance on evaluation techniques for reinforcement location is provided in Chapter 6.

@seismicisolation

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 and in accordance with the design-basis code.

R7.6.3.3 Reinforcement development may be inadequate due to corrosion, mechanical damage, insufficient or loss of concrete cover, delaminated concrete, concrete strength, or other conditions. Equations to calculate the development length have also changed over time and current equations may require longer development lengths than earlier equations. The design of the repair should evaluate the required development length. Detailing of the repair should include the proper development of new reinforcement to achieve the design force. ACI 318 provides development equations and requirements for detailing of steel reinforcement. ACI 369.1 (or ASCE/SEI 41) provides an equation when development length of existing reinforcement does not meet ACI 318. ACI 440.1R and ACI 440.2R provide detailing guidance for internal FRP reinforcement and externally bonded FRP reinforcement, respectively. Additional information can be found in *fib* Bulletin No. 10.

7.6.4 *Prestressed structures*

R7.6.4 *Prestressed structures*

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

R7.6.4.1 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 a condition assessment of the existing tendons. Repair of unbonded tendons may require tendon detensioning. Guidance for analysis, evaluation methods and repair techniques of unbonded post-tensioned structures is provided in ACI 423.4R, ACI 222.2R, ICRI Guideline No. 210.2, PTI DC80.2-10, and PTI DC 80.3/ICRI 320.6.

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

R7.6.4.2 Analysis to evaluate the effects of structural modifications should verify that strength is adequate and that all serviceability conditions (for example, deflection limits) are satisfied.

Analysis of prestressed structures is required to evaluate the effect of damaged or severed prestressing reinforcement on structural strength and performance. The effect of a severed bonded tendon is typically localized because the severed tendon is effective after a development length is achieved and the full strength of the tendon is 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 should be performed to evaluate grouted tendons in advance of any repair or rehabilitation of bonded post-tensioning systems. The presence of voids, moisture in ducts, chlorides and the extent of carbonation in the existing grout need to be identified. Methods for evaluation of chloride-ion content are listed in ASTM C1152M, ASTM C1218, and AASHTO T260. Field evaluation of grout may be required even if documentation of the original construction is available.

CODE

COMMENTARY

Unbonded tendons are designed to be permanently debonded from the member and often extend over multiple spans. As a result, damage or discontinuity of a tendon at one location will reduce the strength for the entire length of the tendon.

If unbonded tendons are severed, the prestressing force is assumed to 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 may 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.

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, unbonded tendons should be reanchored and restressed to restore required structural strength. Cut or damaged unbonded tendons can be restored by splicing or by installing new tendons with anchors at intermediate locations, at the end of the structural member or the edge of any new openings.

The stressing force in a repaired tendon depends on the condition and type of the repaired post-tensioned system and in certain cases this force can be less than the original force if determined to be acceptable by structural analysis. Further discussion of this topic can be found in PTI DC 80.3/ICRI 320.6.

Corrosion on prestressing strands for bonded and unbonded post-tensioned systems may have an effect on strand integrity and strength. Prestressing strands require examination for conditions such as corrosion pitting and hydrogen embrittlement (refer to ICRI Guideline No. 210.2 and ACI 222.2R).

If repairs to prestressed slabs or beams result in increased concrete tensile stress (that is, changing the classification of the prestressed flexural member as defined in ACI 318), impacts of the repair scheme on serviceability should be evaluated.

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

R7.6.4.3 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. 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). PTI DC 80.3/ICRI 320.6 provides guidance for removing concrete around anchors and splices to prevent catastrophic anchorage failure.

7.6.5 *Anchoring to concrete*—Post-installed anchors shall be designed in accordance with ACI 318 to transfer design

R7.6.5 The design of post-installed anchors requires careful consideration of the loads to be resisted. Anchors

CODE

forces to the substrate considering possible anchor failure modes and the condition of the substrate into which the anchor is installed.

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.

7.6.7 *Expansion joint materials*—Selection of expansion joint materials shall consider the anticipated movement of the structure and facility maintenance procedures.

COMMENTARY

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 the 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 hazard should be able to transfer the design seismic forces assuming a cracked concrete condition.

Design of post-installed anchors is provided in ACI 318, which includes provisions that require 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 post-installed expansion or undercut anchors, manufacturer's installation instructions 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. Manufacturer's printed 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 be specified in the construction documents. 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. Refer to ACI 318 for specific inspection requirements for post-installed anchors.

R7.6.6 Repair shapes with sharp reentrant corners can cause stress concentrations that may result in cracking. Long, slender (high aspect ratio) repair areas also may result in transverse cracking. The shape of the repair should be considered to reduce stress concentrations and possible cracking. Methods discussed in ICRI Guideline No. 310.1R provide guidance to reduce cracking in concrete repairs including providing a uniform depth of edges and substrate, repair geometry, surface preparation, concrete removal below reinforcement (undercutting) and elimination of feather edge repairs.

R7.6.7 Repairs to expansion joint materials are common, particularly those subjected to snow removal operations.

Design and selection of the expansion joints should consider the total anticipated movement of the expansion joint. Typically, expansion joint capacities listed in manufacturer's literature are based on total movement from minimum installation width to maximum installation width and assume the joint will be installed when the joint is at the midpoint of this movement range. Joints installed in the summer or winter months will experience movement

CODE

COMMENTARY

primarily in one direction only, and so may require a larger capacity. Additional guidance can be found in the Parking Facility Maintenance Manual published by the National Parking Association (2016).

7.7—Repair using supplemental post-tensioning

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

7.7.2 The effects of the supplemental post-tensioning on the behavior of the structure shall be considered in the repair design.

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 anchor zones shall be in accordance with ACI 318. Design of steel brackets and supplementary structural steel shall be in accordance with ANSI/AISC 360.

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 Construction documents shall specify the repair sequence, including tendon placement, anchors, and stressing of the post-tensioned system.

R7.7—Repair using supplemental post-tensioning

R7.7.1 Supplemental post-tensioning can be applied to the structure externally, internally, or both.

R7.7.2 Supplemental post-tensioning can introduce moment, shear, and axial forces within the 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 result in significant internal forces. Refer to ICRI Guideline No. 330.1 for selecting strengthening systems for concrete structures.

R7.7.2.1 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 classification of prestressed flexural members as defined in ACI 318 Section 24.5.2. This change is acceptable as long as durability and strength are addressed as part of the repair design.

R7.7.2.2 Anchors for new post-tensioned reinforcement should be designed and detailed for the transfer of post-tensioning forces to the existing structure. Bearing, spalling, and bursting forces created at anchor zones should be considered. Strut-and-tie modeling, as given in ACI 318, may be used to design post-tensioning anchor zones.

R7.7.3 The post-tensioning forces 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.

R7.7.4 Losses include wedge seating in the anchor; elastic shortening; creep of original concrete; shrinkage of original concrete following installation of the supplemental prestressing; creep of repair material; shrinkage of repair material; prestressing relaxation; and friction and wobble 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.

R7.7.5 Repair design using supplemental post-tensioning systems should include construction documents for installation sequence including shoring, removal of concrete, placement of new material and reinforcement, additional anchor requirements, horizontal shear transfer requirements, curing, and stressing. Installation of supplementary post-tensioning

CODE | **COMMENTARY**

involves application of significant forces, which may require project safety and protection procedures by the installer. Refer to 8.4.1 for corrosion protection requirements.

7.7.6 Structural members repaired or modified with externally installed unprotected post-tensioning shall have adequate unrepaired strength, in accordance with 5.5.

R7.7.6 Unless protection of the post-tensioning strengthening system is provided to prevent sudden failure of the member in case the external post tensioning reinforcement is damaged or becomes ineffective (such as fire or impact), the structural member should have adequate strength without the post-tensioning reinforcement to support factored loads, as defined in Chapter 5.

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

7.8.1 Fiber-reinforced polymer composites in conformance with ACI 440.6 and ACI 440.8 shall be permitted to repair concrete structures.

R7.8—Repair using fiber-reinforced polymer (FRP) composites

R7.8.1 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 standalone 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 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.

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

FRP systems should only be installed in or on sound concrete. Concrete distress, deterioration, and corrosion of reinforcement should be evaluated and addressed before the application of the FRP system. Surface preparation requirements should be based on the intended application of the FRP system. FRP applications can be categorized as bond-critical or contact-critical. Bond-critical applications, such as flexural or shear strengthening of beams, slabs, columns, or walls, require an adhesive bond between the FRP system and the concrete. Contact-critical applications, such as confinement of columns, only require intimate contact between the FRP system and the concrete. Contact-critical applications do not require an adhesive bond between the FRP system and the concrete substrate, although one is often provided to facilitate installation. ACI 440.2R provides descriptions of FRP applications and surface preparation and repair requirements.

For bond-critical applications, the concrete substrate should possess the necessary strength to develop the design forces of the FRP system through bond. The substrate, including all bond surfaces between repaired areas and the original concrete, should have sufficient direct tensile and shear strength to transfer force between the existing substrate and FRP system. The tensile strength of the substrate should be at least 200 psi as determined by a pull-off type adhesion test per ASTM D7522/D7522M. Contact-critical applications are not required to meet this minimum bond value as

CODE | COMMENTARY

the design forces of the FRP are developed by deformation or dilation of the concrete section.

For bond-critical applications, the concrete surface should be prepared to a minimum concrete surface profile (CSP) 3 as defined by the ICRI Guideline No. 310.2R. In contact-critical applications, surface preparation should promote continuous intimate contact between the concrete surface and the FRP system. Surfaces to be wrapped should, at a minimum, be flat or convex to promote proper loading of the FRP system.

FRP systems should not be applied to damp or wet surfaces unless the epoxies are formulated by the manufacturer for such applications. Moisture content of the concrete substrate should be evaluated before application of the FRP system as it may inhibit bonding between the concrete substrate and epoxy polymer. Surface moisture should not exceed the limits established by the manufacturer. Testing for presence of moisture should be done in accordance with manufacturer's written recommendations or one of the following: ASTM D4263 – "Standard Test Method for Indicating Moisture in Concrete by the Plastic Sheet Method;" AASHTO FRPS-1-UL – "Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements," first edition; ACI 548.1R; ASTM F1869 – "Standard Test Method for Measuring Moisture Vapor Emission Rate (MVER) of Concrete Subfloor Using Anhydrous Calcium Chloride;" ASTM F2170 – "Standard Test Method for Determining Relative Humidity in Concrete Floor Slabs Using In Situ Probes;" or ASTM F2420 – "Standard Test Method for Determining Relative Humidity on the Surface of Concrete Floor Slabs Using Relative Humidity Probe Measurements and Insulated Hood."

The surfaces to receive moisture testing and the testing equipment should be acclimated near the relative humidity levels and temperatures that the design is anticipated to have in service. Variation between testing and in-service conditions may provide inaccurate or misleading testing results.

7.8.2 Structural members repaired or modified with externally-applied FRP composites shall have adequate unrepaired strength, in accordance with 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 design-basis code.

R7.9—Performance under fire and elevated temperatures

R7.9.1 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 materials selected to provide adequate performance. 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.

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7.9.2 It shall be permitted to design a repair without supplemental fire protection if the unrepaired member has adequate strength during a fire event considering the reduced material properties due to fire exposure in accordance with 5.5.3.

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.

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

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

COMMENTARY

R7.9.2 A repair system can be selected without additional fire protection provided that the existing unrepaired member has adequate strength during a fire event to support the loads, as defined in 5.5.3. Fire performance requirements and evaluation procedures for a structure during a fire event are outlined in ACI 216.1, ASCE/SEI/SFPE 29, and AISC Design Guide 19.

R7.9.3 Repair material specifications should comply with the requirements of relevant fire regulations valid at the project location. If 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.

R7.9.4 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 acceptable service temperature for FRP is established by ACI 440.2R as T_g – 27°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 exceeds T_g – 27°F. A service temperature exceeding this limit temperature should be addressed using an adhesive system with a higher T_g value, using heat protection or insulation systems or using alternate repair systems. Similar service temperature considerations apply to adhesive-bonded steel reinforcement.

Adhesive-based repair systems can be considered effective during a fire event if a fire protection system with an established fire rating is used that maintains the temperature of the adhesive-based system below its glass transition temperature. In the absence of an established fire rating, detailed fire analysis may be used to establish a fire rating of the repaired system.

R7.9.5 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; NFPA 5000 2015; 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 216.1). Concrete cover for nonmetallic reinforcement may need to exceed cover for steel reinforcement to achieve the same fire resistance rating.

R7.9.6 The fire rating of a repaired system or assembly can be determined in accordance with ACI 216.1, which requires

CODE

COMMENTARY

the use of reduced material strength due to fire exposure and a strength reduction factor during fire of 1.0.

The criteria for evaluating a structure for fire safety are different than those for strength design and typically incorporate lower material strengths and required strength, and may not require the use of strength reduction factors (refer to Section 5.5.3). The licensed design professional should verify that the fire-reduced strength of the member exceeds 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 216.1.

Section 1.2 of ACI 216.1 allows alternative methods to assess the fire resistance of assemblies. The fire reduced strength as well as the effect of fire protection system on the overall performance and fire rating of an existing and repaired element can also be determined utilizing available design models and finite element numerical procedures. Descriptions of the detailed analytical methods can be found in Buchanan (2001) and *Technical Report 68* (2008) by the Concrete Society.

The fire resistance or rating of a repaired system or assembly can be determined through full scale testing in accordance with ASTM E119, which requires the application of the expected service load to the test specimen during the full-scale fire test.

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CHAPTER 8—DURABILITY

8.1—General

8.1.1 Durability of individual repairs, the repaired structure, and the interaction between the repaired areas and the remaining structure shall be considered.

8.1.2 Cause(s) of current conditions, defects, and potential future deterioration of repairs shall be assessed as part of the repair design.

COMMENTARY

R8—DURABILITY

R8.1—General

R8.1.1 The durability of materials incorporated into a repair depends on the ability of the materials to withstand the environment where they are installed. A repaired section is considered to be the combination of the installed repair material(s) and the substrate material(s). The durability of repairs is dependent on the compatibility between repair materials, the structure, and the surrounding environment. To achieve compatibility, the repair and structure need to interact on several levels without detriment, including chemical, electrochemical, and physical behavior.

For repair durability design, the design service life is the time frame for which durability should be considered. The design service life requirements for a repaired structure are established by the licensed design professional in consultation with the Owner to achieve a repair that satisfies project requirements including strength, safety and serviceability. Such design service life should be reflected in the repair design and maintenance requirements, as well as incorporated into the construction documents. Design service life may be achieved through satisfactory repair construction practices, including the material selection, surface preparation and application of the repair materials. The design service life of the structure and repaired members, including maintenance requirements, may be estimated by considering the durability of the repair materials and their interaction with the structure. Service life and the parameters to be considered, the limitations, and methods available for conducting a service life prediction are presented in ACI 365.1R. Some examples of end of service life where durability parameters are not met include:

(a) Unacceptable reduction in structural performance

(b) Unacceptable frequency of maintenance cycles and associated activities

(c) Exceeding maximum crack width or crack frequency from corrosion, shear, torsion, flexure

(d) Exceeding maximum permissible chloride level at the interface of the steel in the repair area, or in adjacent areas

(e) Depth of carbonation leading to corrosion of reinforcement

(f) Unacceptable reinforcement section loss due to corrosion

(g) Exceeding maximum concrete deterioration level, mass loss or unacceptable surface conditions due to deterioration mechanisms, such as corrosion, freeze-thaw, chemical attack, abrasion, sulfate attack, alkali-silica reaction (ACI 221.1R, ACI 364.11T), or delayed ettringite formation

(h) Loss of watertightness or excessive/unacceptable leakage

R8.1.2 The presence of deterioration and its cause(s) should be determined as a first step in repair durability design. Causes of deterioration include:

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CODE

COMMENTARY

(a) Mechanical (abrasion, cavitation, 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, or biological action)

(c) Physical (freezing and thawing cycles, scaling, humidity gradients, temperature gradients, differing coefficients of thermal expansion, salt crystallization, radiation exposure, ultraviolet light, fire, or differences in permeability between materials)

(d) Reinforcement corrosion (carbonation, corrosive contaminants, dissimilar metals, stray currents, or stress corrosion cracking, location of reinforcement)

(e) Defects

8.1.3 Repair materials and methods shall be selected to be compatible with the structure, and within the service environment. Anticipated maintenance shall be considered in the selection of repair materials and methods.

R8.1.3 Compatibility in concrete repair systems can be defined as the balance of physical, chemical and electrochemical properties, as well as volume changes between the repair, the reinforcement, and the existing substrate. This balance ensures that the composite repair system withstands stresses induced by loads, chemical and electrochemical effects, and restrained volume changes without distress and deterioration over a designed period of time (Vaysburd and Emmons 2006).

Repaired sections should be resistant to expected service conditions that can result in deterioration during the design service life, including the causes of deterioration listed previously, and combinations of these causes.

Repaired sections should be resistant to:

(a) The ingress of chlorides and other corrosive contaminants that are present in the remaining concrete or the ingressof corrosive contaminants into the concrete that lead to corrosion of reinforcement or other embedments (8.4).

(b) The effects of thermal exposure and cycles.

(c) Freezing-and-thawing damage if critically saturated and subject to a freezing-and-thawing environment.

(d) Scaling if exposed to salts.

(e) Deterioration due to exposure to ultraviolet or other radiation deterioration within the repair environment unless other means are provided to address such deterioration.

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

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

(h) Abrasion due to heavy traffic, impingement of abrasive particles, or similar conditions.

(i) Chemical deterioration which may result from 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.

(j) Carbonation-induced corrosion. Carbonation of concrete and repair materials reduces their pH and dimin-

CODE | COMMENTARY

ishes the passivation effect which may lead to corrosion of embedded reinforcement (refer to 8.4).

(k) Deterioration resulting from deleterious aggregate, alkali-aggregate reactions or other aggregate durability concerns.

(l) Deterioration due to trapped moisture as a result of differential permeability between the repair and existing concrete, leading to freezing-and-thawing damage of critically saturated concrete, corrosion of embedded steel reinforcement, alkali-aggregate reaction, or sulfate attack of either the repair concrete or existing concrete.

Appropriate materials selection for concrete repair is discussed in ACI 546.3R.

Environmental classes that may affect durability performance are shown in Table R8.1.3.

8.2—Cover

8.2.1 Concrete cover shall be in accordance with the design-basis code, or an equivalent cover shall be used. An equivalent cover using alternative materials and methods shall be approved in accordance with 1.4.2.

8.2.2 Concrete cover over remaining and new reinforcement shall meet minimum requirements to provide sufficient (i) corrosion protection; (ii) fire protection; and (iii) anchorage and development.

R8.2—Cover

R8.2.2 Concrete cover protects reinforcement in concrete construction from corrosion until the concrete cover becomes contaminated, cracks or is compromised. The protection provided by the concrete cover is important in determining the service life of the structure. The minimum cover is typically required by the design-basis code. The effects of concrete cover on reinforcement corrosion, chloride contamination, and carbonation should be considered when evaluating the maintenance requirements and design service life of alternative methods for corrosion protection.

Adequate protection may be provided by increased section thickness, the appropriate coatings, such as sealers, or both; or electrochemical corrosion protection methods. Alternative means of protecting reinforcement include the application of waterproof membranes (ACI 515.2R), and various forms of cathodic protection. Active corrosion may create distress and deterioration beyond the limits of the repair area. The design service life should consider the existing conditions and potential distress in repairs areas and areas adjacent to the repair.

Concrete cover also provides fire protection. Fire protection requirements can be met by techniques such as increasing cover, spray-on fire protection or intumescent coatings.

8.3—Cracks

8.3.1 The cause(s) of cracks shall be assessed, and mitigation of cracking shall be considered in the repair design. As part of a repair design, cracking mitigation methods shall consider the causes, movement, size, orientation, width, and pattern of cracks. The characteristics of the substrate, location, and evidence of water transmission shall be determined to assess the appropriate method of repair. Active water infiltration shall be corrected as required for the durability of the structure.

R8.3—Cracks

R8.3.1 Cracks can 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. Identification of their cause(s) and evaluation of their impact on a structure or a concrete component is described

CODE

8.3.2 The design of repairs shall consider the effects of cracks on the expected durability, performance, and design service life of the repair and structure as a whole.

COMMENTARY

in ACI 224.1R. Guidance for the assessment of cracking caused by earthquakes is provided in FEMA 306.

R8.3.2 Not all cracks need to be repaired, however, all cracks have the potential to become active cracks. 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, and guidance for critical crack sizes is provided in ACI 224R, Table 4.1.

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.

Table R8.1.3—Exposure categories and classes (adopted from ACI 318-14)

Category	Class	Condition	
Freezing and thawing (F)	F0	Concrete not exposed to freezing-and-thawing cycles	
	F1	Concrete exposed to freezing-and-thawing cycles with limited exposure to water	
	F2	Concrete exposed to freezing-and-thawing cycles with frequent exposure to water	
	F3	Concrete exposed to freezing-and-thawing cycles with frequent exposure to water and exposure to deicing chemicals	
Sulfate (S)		Water-soluble sulfate (SO_4^{2-}) in soil, percent by mass [(1)]	Dissolved sulfate (SO_4^{2-}) in water, ppm[(2)]
	S0	$SO_4^{2-} < 0.10$	$SO_4^{2-} < 150$
	S1	$0.10 \leq SO_4^{2-} < 0.20$	$150 \leq SO_4^{2-} < 1500$ or seawater
	S2	$0.20 \leq SO_4^{2-} \leq 2.00$	$1500 \leq SO_4^{2-} \leq 10{,}000$
	S3	$SO_4^{2-} > 2.00$	$SO_4^{2-} > 10{,}000$
In contact with water (W)	W0	Concrete dry in service, concrete in contact with water and low permeability is not required	
	W1	Concrete in contact with water and low permeability is required	
Corrosion protection of reinforcement (C)	C0	Concrete dry or protected from moisture	
	C1	Concrete exposed to moisture but not to an external source of chlorides	
	C2	Concrete exposed to moisture and an external source of chlorides from deicing chemicals, salt, brackish water, seawater or spray from these sources	

[(1)]Percent sulfate by mass in soil shall be determined by ASTM C1580.

[(2)]Concentration of dissolved sulfates in water, in ppm, shall be determined by ASTM D516 or ASTM D4130.

There are a variety of different materials that have been used for crack repair, and the correct specification for a given application will affect the design service life of the 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 at the concrete surface(s). Crack injection can be another effective repair approach. For repair by crack injection, the process and material should be appropriate to the site conditions. Dormant cracks can be repaired by injection using materials such as epoxy, polyurethane, latex in a cement matrix, microfine cement, and polymethacrylate. 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.

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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. Repairs shall not contain intentionally-added constituents that are corrosive to reinforcement within the repair area.

8.4.2 The impact on the design service life of the repaired structure shall be considered if it is anticipated that corrosion products cannot be removed from the reinforcement during repair.

8.4.3 The quality of existing concrete and its ability to protect reinforcement from corrosion, fire and other forms of damage and deterioration shall be considered.

8.4.4 Galvanic corrosion between electrochemically dissimilar materials shall be considered.

COMMENTARY

R8.4—Corrosion and deterioration of reinforcement and metallic embedments

R8.4.1 Untreated reinforcement corrosion limits the life expectancy of repair areas, repair materials, and repaired structures. ICRI No. 310.1R provides guidelines on removal of damaged concrete and cleaning of reinforcement. Repairs that do not address reinforcement corrosion may negatively impact the design service life and require more intensive monitoring. The structural design considerations for corroding reinforcement on repairs are described in 7.6.3.1.

R8.4.2 Ideally corrosion products should be removed from reinforcement in repairs. In some situations, due to congestion of reinforcement, access limitations, load considerations, or other factors, it is not possible to remove corrosion products from the steel reinforcement. Situations exist where corroding reinforcement that cannot be adequately cleaned or repaired will remain in the repaired structure. The effects of uncleaned reinforcement on the long-term durability of the repaired structure should be considered in these situations. Supplemental corrosion mitigation strategies may be needed in these situations.

R8.4.3 Water and chemical penetration into the concrete can cause corrosion of metallic embedments and damage to nonmetallic reinforcement.

The corrosion of embedded metals adjacent to the repair may be accelerated due to differing electrical potential between electrically continuous reinforcement in the repair area and external to the repair area. This form of corrosion is commonly referred to as the "anodic ring" or "halo effect," and is discussed in ACI 364.3T, ACI 546R, and "ACI RAP Bulletin 8" (ACI Committee E706 2005). The rate of anodic ring corrosion depends upon the chloride content, internal relative humidity, and temperature.

The anodic ring effect, which may be induced by certain repairs, should be addressed by incorporating appropriate corrosion mitigation strategies such as cathodic protection or corrosion inhibitors. ACI 222R, ACI 222.3R, ACI 364.3T, ACI 546R, and *Technical Report 50* (The Concrete Society 1997) and FAQ sections from Concrete International (2002a,b,c) provide guidance for corrosion prevention, mitigation and inhibition. Both carbonation and chloride contamination may require consideration and are discussed in ACI 546R.

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.

R8.4.4 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 that use dissimilar

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CODE

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

8.4.6 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.

8.4.7 Repair materials and reinforcement shall be selected and detailed 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.

COMMENTARY

metals from conventional steel reinforcement could accelerate the deterioration of the installation (refer to ACI 222R).

R8.4.5 The presence of 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). Section 7.6.4 addresses the structural requirements for the repair.

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, the prestressing steel sheathing type and its risk for gaps and breaches that provide transmission pathways for contaminants, and the continuity of the prestressing steel need to be considered to address corrosion protection of the structure. Refer to PTI DC80.3-12/ICRI 320.6 and ACI 222.2R.

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

R8.4.6 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 to not promote an alkali-aggregate reaction (AAR) and to avoid embrittlement of prestressing steel.

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 01101, NACE 01102, NACE 01104, NACE 01105, NACE SP0107, NACE SP0290, and NACE SP0390).

R8.4.7 Incompatibilities can arise from the use of inappropriate materials or components, or dissimilar electrochemical characteristics or physical properties, which can negatively impact the concrete and reinforcement. Some examples include:

(a) 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.

(b) Even if the conventional steel reinforcement becomes more noble 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).

(c) 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

CODE

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 the impact of abrasion, concrete cracks and their anticipated expansion and contraction, and anticipated movement of the structure on the repair system durability, the surface treatment, and the anticipated design service life of the structure.

COMMENTARY

repaired and corrosion mitigated. Appropriate sections within this code and referenced documents concerning FRP repairs should be consulted (refer to ACI 440.2R).

R8.5—Surface treatments and coatings

R8.5.1 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 (ACI 515.1R).

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. 310.2R).

R8.5.2 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.

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CODE

CHAPTER 9—CONSTRUCTION

9.1—General

Construction documents shall specify that:

(a) The contractor has the responsibility to construct the project in accordance with the construction documents and with appropriate standards.

(b) The contractor has the responsibility to provide the necessary resources and access for inspection, testing, field observations, and quality control of the work.

(c) Specific temporary shoring and bracing requirements in accordance with Section 9.2.

(d) Specific jacking requirements.

(e) Project-specific inspection, testing, and construction observation requirements of Chapter 10.

9.2—Stability and temporary shoring requirements

9.2.1 Construction documents shall specify:

(a) Portions of the work that require temporary shoring and bracing during the period before the repair implementation for safety purposes and during construction

(b) Design loads and necessary spacing limitations for design of temporary shoring and bracing

(c) Contractor responsibilities to install, provide quality control, and properly maintain the temporary shoring and bracing

9.2.2 Temporary shoring and bracing design shall consider:

(a) Accommodation for in-place conditions and changes in conditions over the period of the repair phases, per 9.2.7

(b) Effects from measured lateral and vertical displacements, tilting or listing, secondary effects, and superimposed loads

(c) Impact of the temporary shoring and bracing on the structure

(d) Effects of deformation compatibility of the shoring system with the supported and supporting structural members and systems, in accordance with 9.2.6

(e) Structural stability of members, systems, and the structure in accordance with 9.2.5 and 9.2.6

(f) Effects of damage or deterioration of existing members and systems in accordance with 9.2.8

9.2.3 Shoring and bracing design shall be performed by a licensed design professional.

COMMENTARY

R9—CONSTRUCTION

R9.1—General

The information to be presented in construction documents is described in 1.6.1. Specific to the construction process, the construction documents should indicate that the contractor is responsible for construction consistent with the project plans and specifications, and convey project specific shoring, bracing and jacking requirements. During the work, the contractor should make the work available for inspection and observations by the licensed design professional, repair inspectors, and other quality assurance personnel.

R9.2—Stability and temporary shoring requirements

R9.2.1 Project-specific design criteria for the temporary shoring and bracing in the construction documents should include requirements for loading, displacement limits, spacing, placement, and quality control during construction. ACI 563 provides specifications for shoring of repairs.

R9.2.2 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 and bracing members should be based on the in-place loads on the structure, deformations of the structure, and anticipated superimposed loads during construction. Secondary effects that may need to be examined in shoring and bracing design include geometric and material nonlinear response, member and foundation displacement, and internal member forces developed due to placement and alignment of shoring and bracing elements.

Anticipated loads, such as snow, seismic, wind, and construction and occupancy live loads, should be considered in the design criteria of the temporary shoring and bracing. Design requirements for shoring are contained in ASCE/SEI 37. Shoring design guidelines are contained in AISC Steel Design Guide Series 10 (Fisher and West 2003) and ACI SP-4.

R9.2.3 Shoring and bracing design is not usually performed by the licensed design professional of record for the repair design. The contractor will usually retain a specialty engineer to prepare the temporary shoring design details and shoring-plans, showing loads, member type, spacing, and placement sequence for temporary shores and braces at the phases of planned repairs.

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CODE

COMMENTARY

9.2.4 The licensed design professional for the repair design shall review temporary shoring and bracing design and details for compliance with the requirements of the project repair design and the temporary shoring and bracing criteria.

R9.2.4 Temporary shoring and bracing design and installation details should be reviewed by the licensed design professional for the repair project to assess the impact of the shoring on the structure, and to verify conformance of the proposed shoring with project-specific requirements. Refer to 5.1.4 for load requirements associated with shoring and temporary construction. Review of the shoring design by the licensed design professional for the repair design does not normally include a comprehensive review of the shoring design prepared by the specialty engineer and should not be considered a validation of the specialty engineer's design.

9.2.5 The shoring and bracing shall maintain the global structural stability of the structure before remedial construction and during the repair phases.

R9.2.5 The assessment of global structural stability includes the overall structure, members and systems affected by repair, and temporary lateral bracing elements that contribute to overall stability. Stability of these elements should be considered during the phases of the repair process. Temporary measures may be needed to provide lateral bracing and shoring of affected members and systems. If necessary, the criteria to temporarily preload members should be included in the construction documents. Review and redesign for variations in the construction proposed by the contractor with changes in temporary shoring and bracing design and detailing should be addressed in the construction documents.

The licensed design professional should be aware that compression-controlled columns with high axial loads in a structure with substantial structural damage may behave in a brittle manner, with little warning prior to localized failure or possible progressive collapse. Therefore, caution should be taken in the design and installation sequence for stabilization measures in these situations.

9.2.6 The shoring and temporary bracing shall maintain the structural stability of members and systems before construction and during the repair phases.

The lateral forces for temporary bracing design shall be determined using generally accepted engineering principles or as required by the design-basis code. Temporary shoring and bracing shall be designed to provide sufficient stiffness to prevent vertical and lateral displacement of the shored or braced members in excess of limits specified by the licensed design professional for the repair in the construction documents.

R9.2.6 Supplemental bracing for compression members may be required if the cross section or unbraced length of a compression member is modified during the repair process. Compression members include columns, walls, beam flanges, and other members, such as chords or diaphragms that resist compressive loads. The design of bracing members is described in various publications (AISC 2006; ANSI/AF&PA NDS 2014). 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).

9.2.7 The design of shoring and bracing members shall accommodate in-place conditions and changes in conditions during construction. The design shall, at a minimum, include consideration of (a) the changes in load paths, (b) construction loads, (c) unbraced lengths, and (d) the redistribution of loads and internal forces that result from removal of existing adjacent framing or changes in applied loads on structural members.

R9.2.7 Removal of column, beam, wall, and floor slab elements or parts thereof during repair construction and the placement of shoring and bracing can result in the redistribution of loads and internal forces within the building structure. The removal of framing members, diaphragms, or slabs can also affect the unbraced length of the framing members in the removal area. Effects of the removal of elements should be considered in assessing the structure and shoring and bracing design.

CODE

9.2.8 Where structural members are required to support the structure and superimposed loads before repair and during construction, the structural capacity of damaged or deteriorated members shall be evaluated. The evaluation shall consider the actual cross section of the member and reinforcing at the time of the repair including losses of capacity due to damage and deterioration. Structural members that require temporary shoring and bracing to be installed and maintained in place during construction until the member is repaired shall be identified on the construction drawings.

COMMENTARY

R9.2.8 Design of shoring and bracing members and the evaluation of members should be based on the member cross sections before and during the time of repair implementation. To account for unknown conditions, the evaluation by the licensed design professional should consider the importance of the member to the overall stability of the structure.

9.3—Temporary conditions

9.3.1 Load and load factors used during the assessment and construction processes shall be in accordance with 5.1.4.

R9.3—Temporary conditions

R9.3.1 During the assessment and repair process, a temporary reduction in design load may be allowed, except if prohibited by authorities having jurisdiction or local building codes. Reduction in the design load intensity should be determined using the in-place condition of the structure and the time required for the completion of stabilization measures or repairs. ASCE/SEI 37 provides information on reductions in loads based upon the duration of a project. If a change in the length of the project or a delay occurs, the reduced design loads may no longer be appropriate.

9.4—Environmental issues

9.4.1 Construction documents shall specify the contractor or other designated party is responsible for implementing environmental remediation measures, reporting new conditions encountered, and controlling construction debris, including environmentally hazardous materials and conditions.

R9.4—Environmental issues

R9.4.1 Assessment and repair of a structure can result in the exposure of workers and the public to potentially hazardous materials and conditions. Hazardous materials may be exposed, dislodged, carried into the air, or discharged as effluent into surface drainage during the assessment and repair process. Hazardous conditions include noise, nuisance dust, misdirected drainage, and falling debris. The Owner should have an environmental assessment performed during the structural assessment 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 structure is free of hazardous materials.

During the repair project, 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.

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CODE

CHAPTER 10—QUALITY ASSURANCE

10.1—General

10.1.1 Quality assurance requirements of this chapter supplement the current and existing-building code provisions and shall be used for repair and rehabilitation construction.

10.2—Inspection

10.2.1 Concrete repair and rehabilitation construction shall be inspected as required by the building code and construction documents.

10.2.2 The construction documents shall include testing and inspection requirements applicable to the project.

COMMENTARY

R10—QUALITY ASSURANCE

R10.1—General

R10.1.1 The construction documents for repair and rehabilitation projects should include a project-specific quality assurance and inspection program. The quality assurance program should include:

(a) Review of the contractor's quality assurance program

(b) Quality control procedures during the repair process

(c) Review of conditions during the project

(d) Testing of materials used and material installation procedures

Usually, the quality control requirements are specified in the construction documents and the Owner retains the quality control personnel. The contractor is responsible for the work quality, including the quality of materials and workmanship.

R10.2—Inspection

R10.2.1 The quality of concrete repairs is largely dependent upon the workmanship during construction. Inspection is necessary to verify repairs and rehabilitation work are completed in accordance with construction documents. Most general building codes require special inspections for construction, which were developed for new construction. Typical repair construction is different from new construction in scope, and new construction testing requirements may not be sufficient for repair construction. Construction documents should specify inspection requirements for concrete repair and rehabilitation construction during the various work stages. The licensed design professional should recommend that the Owner retain a licensed design professional, a qualified inspector, a qualified individual, or some combination thereof for the necessary inspections.

R10.2.2 Required testing and inspections may include (a) through (j):

(a) Delivery, placement, and testing reports documenting the identity, 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 of the concrete and reinforcement

(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) Review and reporting of construction loads on floors, beams, columns, and walls

(i) General progress of work

(j) Installation and testing of post-installed anchors

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

Repair construction should be inspected to verify the quality of materials, quality of workmanship, and for compliance with the intent of the construction documents. Inspection should be provided by either repair inspectors,

CODE

COMMENTARY

the licensed design professional, or a combination of repair inspectors and the licensed design professional. Responsibilities for performing the inspections should be clearly delineated at the start of a project.

Repair inspector qualifications for inspection of concrete repairs should be demonstrated by certification or previous work history and as required by the authority having jurisdiction before being retained. An individual who has been certified as an ICRI Concrete Surface Repair Technician (ICRI CSRT) or as an ACI Construction Inspector (ACI C630) are examples of qualified inspectors. The licensed design professional may provide inspection services.

Inspection of concrete repair construction as specified in the construction documents should include review of the work in the field, review of construction documents, comparison of the work with construction documents, documentation and report of the work inspected as conforming or nonconforming, and whether corrections were made and verified or are still needed. Inspection and testing of post-installed anchor installation should be performed as required by the construction documents and in accordance with Chapters 17 and 26 of ACI 318-14.

Repair inspections should determine compliance with the intent of the contract documents, document the inspection, and report the inspection results. 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 issues are corrected by the contractor, then the inspected work should be documented as conforming and reported to the licensed design professional, Owner, and contractor with corrections noted and verified as completed. Nonconforming or deficient components, processes, and procedures including the parts of the repairs not passing inspection should be reported to the licensed design professional for review. Actions should be made to correct the process before resuming the repair construction and inspection process. Nonconforming repair construction may include:

(a) Existing construction that differs from the repair documents

(b) 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 indicate the manufacturer's stated material properties that should satisfy the required properties of each specific repair. 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 or deterioration, and size and condition of the members. Those conditions need to be verified for confor-

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mance to the design assumptions. The following are some items where inspections are beneficial:

(a) Location of repairs
(b) Surface preparation of existing concrete and 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

10.2.3 The construction documents shall establish inspection requirements of existing conditions and reinforcement that need to occur before concealing with materials that obscure visual inspection.

R10.2.3 Removal of deteriorated concrete and reinforcement often uncovers unanticipated conditions that should be examined. Visual inspection and verification of existing conditions may require review of project 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. If unanticipated conditions are identified by the repair inspector, the licensed design professional should be informed. The licensed design professional should examine these conditions and determine what measures are to be implemented before placement of new repair materials. The construction documents should specify the locations where inspection is necessary before concealment and provide for possible changes in these locations due to unforeseen conditions. In some projects, all locations will not need to be inspected and representative locations will provide suitable inspection.

10.3—Testing of repair materials

10.3.1 Repair material tests and test frequencies shall be specified in the construction documents. Results of tests shall be reported as required by the construction documents and the design-basis code. Test records shall be retained by the testing agency as required by the design-basis code.

R10.3—Testing of repair materials

R10.3.1 Tests of repair materials should comply with testing and test frequency of new concrete construction, unless otherwise specified in the contract documents and approved by the authority having jurisdiction. It is generally not practical to verify all manufacturers' listed properties of proprietary materials, such as shrinkage, compressive and tensile creep, thermal expansion coefficient, and modulus of elasticity. In such cases, the licensed design professional should seek independent testing data from the manufacturer or others to verify specific manufacturer's listed properties that are critical to the application for the specific lots (or batches) of material to be used. 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 No. 320.2R for guidance. Tests of repair materials' bond to existing materials should comply with requirements of the contract documents.

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CODE

COMMENTARY

Concrete repair materials 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.

As a minimum level of record keeping, the testing agency should maintain a record of the tests performed and the results consistent with the requirements for records in ASTM E329.

10.4—Construction observations

10.4.1 Construction observation shall be performed as required by the authority having jurisdiction and construction documents.

R10.4—Construction observations

R10.4.1 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 work detailed in the contract documents will fulfill the design intent. Construction observations are in addition to the inspection requirements described in 10.2. Construction observations should be performed by the licensed design professional that designed the work or other designated representative to provide these services.

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 or designated person responsible for construction observations should report design changes in writing to the Owner, rehabilitation inspector, contractor, and authority having jurisdiction resulting from existing construction, nonconforming rehabilitation work, and observed construction deficiencies. When construction observations are made by a party designated by the licensed design professional, design changes (construction deviations from the repair design) should also be reported to the licensed design professional. Revised design or construction work necessary to correct these deficiencies, and the construction corrections, should also be observed.

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CODE

COMMENTARY

CHAPTER 11—COMMENTARY REFERENCES

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

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CODE

COMMENTARY

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ACI 548.1R-09—Guide to the Use of Polymers in Concrete
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CODE

COMMENTARY

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CODE

COMMENTARY

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CODE

COMMENTARY

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COMMENTARY

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As ACI begins its second century of advancing concrete knowledge, its original chartered purpose remains "to provide a comradeship in finding the best ways to do concrete work of all kinds and in spreading knowledge." In keeping with this purpose, ACI supports the following activities:

· Technical committees that produce consensus reports, guides, specifications, and codes.

· Spring and fall conventions to facilitate the work of its committees.

· Educational seminars that disseminate reliable information on concrete.

· Certification programs for personnel employed within the concrete industry.

· Student programs such as scholarships, internships, and competitions.

· Sponsoring and co-sponsoring international conferences and symposia.

· Formal coordination with several international concrete related societies.

· Periodicals: the ACI Structural Journal, Materials Journal, and Concrete International.

Benefits of membership include a subscription to Concrete International and to an ACI Journal. ACI members receive discounts of up to 40% on all ACI products and services, including documents, seminars and convention registration fees.

As a member of ACI, you join thousands of practitioners and professionals worldwide who share a commitment to maintain the highest industry standards for concrete technology, construction, and practices. In addition, ACI chapters provide opportunities for interaction of professionals and practitioners at a local level to discuss and share concrete knowledge and fellowship.

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The American Concrete Institute (ACI) is a leading authority and resource worldwide for the development and distribution of consensus-based standards and technical resources, educational programs, and certifications for individuals and organizations involved in concrete design, construction, and materials, who share a commitment to pursuing the best use of concrete.

Individuals interested in the activities of ACI are encouraged to explore the ACI website for membership opportunities, committee activities, and a wide variety of concrete resources. As a volunteer member-driven organization, ACI invites partnerships and welcomes all concrete professionals who wish to be part of a respected, connected, social group that provides an opportunity for professional growth, networking and enjoyment.

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