

ACI 369.1-17

An ACI Standard

Standard Requirements for Seismic Evaluation and Retrofit of Existing Concrete Buildings (ACI 369.1-17) and Commentary

Reported by ACI Committee 369



American Concrete Institute
Documents



Document Sharing Hub

**Standard Requirements for Seismic Evaluation and Retrofit of Existing Concrete Buildings
(ACI 369.1-17) and Commentary**

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

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

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

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

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

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

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

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

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

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

www.concrete.org

Standard Requirements for Seismic Evaluation and Retrofit of Existing Concrete Buildings (ACI 369.1-17) and Commentary

An ACI Standard

Reported by Committee 369

Wassim M. Ghannoum, Chair

Siamak Sattar, Secretary

Anna C. Birely
Sergio F. Brena
Casey Champion
Jeffrey J. Dragovich
Kenneth J. Elwood
Una M. Gilmartin
Arne Halterman
Wael Mohammed Hassan

Mohammad Iqbal
Jose M. Izquierdo-Encarnacion
Afshar Jalalian
Thomas Kang
Dominic J. Kelly
Insung Kim
Laura N. Lowes
Kenneth A. Luttrell

Adolfo B. Matamoros
Steven L. McCabe
Murat Melek
Jack P. Moehle
Arif M. Ozkan
Robert G. Pekelnicky
Jose A. Pincheira
Mario E. Rodriguez

Murat Saatcioglu
Siamak Sattar
Halil Sezen
Roberto Stark
Andreas Stavridis
John W. Wallace
Tom C. Xia

Consulting Members

Sergio M. Alcocer
David Bonowitz

Charles J. Hookham
Shyh-Jiann Hwang

Regan Milam
Andrew D. Mitchell

Raj Valluvan

PREFACE

*This standard provides retrofit and rehabilitation criteria for reinforced concrete buildings based on results from the most recent research on the seismic performance of existing concrete buildings. The intent of this standard is to provide a continuously updated resource document for modifications to Chapter 10 of **ASCE 41-17**, similar to how the National Earthquake Hazards Reduction Program (NEHRP) Recommended Seismic Provisions produced by the Federal Emergency Management Agency (FEMA) (**FEMA 450**) have served as source documents for the International Building Code (IBC) and its predecessor building codes. Specifically, this version of ACI 369.1 serves as the basis for Chapter 10, "Concrete," of ASCE 41-17.*

This standard should be used in conjunction with Chapters 1 through 7 of ASCE 41-17. Chapter 1 of ASCE 41-17 provides general requirements for evaluation and retrofit, including the selection of performance objectives and retrofit strategies. Chapter 2 of ASCE 41-17 defines performance objectives and seismic hazards.

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

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

*Chapter 3 of ASCE 41-17 provides the requirements for evaluation and retrofit, including treating as-built information and selecting the appropriate screening procedures. Chapter 4 of ASCE 41-17 summarizes Tier 1 screening procedures, while Chapters 5 and 6 summarize Tier 2 deficiency-based procedures and Tier 3 systematic procedures for evaluation and retrofit, respectively. Chapter 7 of ASCE 41-17 details analysis procedures referenced in ACI 369.1, including linear and nonlinear analysis procedures, acceptance criteria, and alternative methods for determining modeling parameters and acceptance criteria. Chapter 8 of ASCE 41-17 provides geotechnical engineering provisions for building foundations and assessment of seismic-geologic site hazards. References to these chapters can be found throughout the standard. The design professional is referred to FEMA 547 for detailed information on seismic rehabilitation measures for concrete buildings. Repair techniques for earthquake-damaged concrete components are not included in ACI 369.1. The design professional is referred to **FEMA 306**, **FEMA 307**, and **FEMA 308** for information on evaluation and repair of damaged concrete wall components.*

This standard does not provide modeling procedures, acceptance criteria, and rehabilitation measures for concrete-encased steel composite components. Future versions will provide provision updates for concrete moment frames and will add provisions for concrete components and systems omitted in the present version of the standard.

ACI 369.1-17 was adopted September 22, 2017 and published October 2017. Copyright © 2017, American Concrete Institute.

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

CONTENTS

PREFACE, p. 1

INTRODUCTION, p. 3

CHAPTER 1—GENERAL, p. 5

1.1—Scope, p. 5

CHAPTER 2—MATERIAL PROPERTIES AND CONDITION ASSESSMENT, p. 6

2.1—General, p. 6

2.2—Properties of in-place materials and components, p. 6

2.3—Condition assessment, p. 15

2.4—Knowledge factor, p. 17

CHAPTER 3—GENERAL ASSUMPTIONS AND REQUIREMENTS, p. 18

3.1—Modeling and design, p. 18

3.2—Strength and deformability, p. 21

3.3—Flexure and axial loads, p. 22

3.4—Shear and torsion, p. 24

3.5—Development and splices of reinforcement, p. 25

3.6—Connections to existing concrete, p. 29

3.7—Retrofit measures, p. 31

CHAPTER 4—CONCRETE MOMENT FRAMES, p. 32

4.1—Types of concrete moment frames, p. 32

4.2—Reinforced concrete beam-column moment frames, p. 33

4.3—Post-tensioned concrete beam-column moment frames, p. 43

4.4—Slab-column moment frames, p. 46

CHAPTER 5—PRECAST CONCRETE FRAMES, p. 53

5.1—Types of precast concrete frames, p. 53

5.2—Precast concrete frames expected to resist seismic forces, p. 53

5.3—Precast concrete frames not expected to resist seismic forces directly, p. 54

CHAPTER 6—CONCRETE FRAMES WITH INFILLS, p. 56

6.1—Types of concrete frames with infills, p. 56

6.2—Concrete frames with masonry infills, p. 56

6.3—Concrete frames with concrete infills, p. 60

CHAPTER 7—CONCRETE STRUCTURAL WALLS, p. 63

7.1—Types of concrete structural walls and associated components, p. 63

7.2—Reinforced concrete structural walls, wall segments, and coupling beams, p. 65

CHAPTER 8—PRECAST CONCRETE STRUCTURAL WALLS, p. 77

8.1—Types of precast structural walls, p. 77

8.2—Precast concrete structural walls and wall segments, p. 78

CHAPTER 9—CONCRETE BRACED FRAMES, p. 83

9.1—Types of concrete-braced frames, p. 83

9.2—General, p. 83

9.3—Stiffness of concrete braced frames, p. 83

9.4—Strength of concrete-braced frames, p. 84

9.5—Acceptance criteria for concrete-braced frames, p. 84

9.6—Retrofit measures for concrete-braced frames, p. 85

CHAPTER 10—CAST-IN-PLACE CONCRETE DIAPHRAGMS, p. 86

10.1—Components of cast-in-place concrete diaphragms, p. 86

10.2—Analysis, modeling, and acceptance criteria for cast-in-place concrete diaphragms, p. 86

10.3—Retrofit measures for cast-in-place concrete diaphragms, p. 87

CHAPTER 11—PRECAST CONCRETE DIAPHRAGMS, p. 88

11.1—Components of precast concrete diaphragms, p. 88

11.2—Analysis, modeling, and acceptance criteria for precast concrete diaphragms, p. 88

11.3—Retrofit measures for precast concrete diaphragms, p. 88

CHAPTER 12—CONCRETE FOUNDATIONS, p. 89

12.1—Types of concrete foundations, p. 89

12.2—Analysis of existing concrete foundations, p. 90

12.3—Evaluation of existing condition, p. 90

12.4—Retrofit measures for concrete foundations, p. 91

CHAPTER 13—NOTATION AND DEFINITIONS, p. 93

13.1—Notation, p. 93

13.2—Definitions, p. 98

REFERENCES, p. 104

Authored references, p. 106

INTRODUCTION

Earthquake reconnaissance has clearly demonstrated that existing concrete buildings designed before the introduction of seismic design codes in the 1980s are more vulnerable to severe damage or collapse when subjected to strong ground motion than concrete buildings built after that period. Seismic rehabilitation of existing buildings where new components are added or existing components are modified or retrofitted with new materials, or both, can be used to mitigate the risk to damage in future earthquakes. Seismic rehabilitation is encouraged not only to reduce the risk of damage and injury in future earthquakes, but also to extend the life of existing buildings and reduce using new materials in the promotion of sustainability objectives.

It is not possible to codify all problems encountered in the process of performing the seismic evaluation and retrofit of reinforced concrete buildings, nor is the intent of the standard to do so. The standard provides a basic framework for modeling and evaluation of structures that reflects the latest information available from researchers and practicing engineers, so that seismic evaluation and retrofit can be performed with a consistent set of criteria. Many provisions in the standard rely on the use of sound engineering judgement for their implementation. The commentary of the standard provides references that describe in detail the implementation of methodologies adopted in the standard.

Keywords: acceptance criteria; building; deformation-controlled; dynamic analysis; earthquake; force-controlled; modeling parameters; nonlinear analysis; retrofit; seismic evaluation.



THIS PAGE INTENTIONALLY LEFT BLANK.



STANDARD

COMMENTARY

CHAPTER 1—GENERAL

1.1—Scope

This standard sets forth requirements for the seismic evaluation and retrofit of concrete components of the seismic force-resisting system of an existing building. These building standard requirements apply to existing concrete components, retrofitted concrete components, and new concrete components. Provisions of this standard do not apply to concrete-encased steel composite components.

Chapter 2 specifies data collection procedures for obtaining material properties and performing condition assessments. **Chapter 3** provides general analysis and design requirements for concrete components. **Chapters 4** through **9** provide modeling procedures; component strengths; acceptance criteria and retrofit measures for cast-in-place and precast concrete moment frames; concrete frames with masonry infills; cast-in-place and precast concrete structural walls; and concrete braced frames. **Chapters 10** through **12** provide modeling procedures, strengths, acceptance criteria, and retrofit measures for concrete diaphragms and concrete foundation systems.

C1.1—Scope

These standard requirements were developed based on the best knowledge of the seismic performance of existing concrete buildings at the time of publication. These requirements are not intended to restrict the licensed design professional from using new information that becomes available before the issuance of the next edition of this standard. Such new information can include tests conducted to address specific building conditions.

This standard provides short descriptions of potential seismic retrofit measures for each concrete building system. The licensed design professional, however, is referred to **FEMA 547** for detailed information on seismic retrofit measures for concrete buildings. Repair techniques for earthquake-damaged concrete components are not included in this standard. The licensed design professional is referred to **FEMA 306**, **FEMA 307**, and **FEMA 308** for information on evaluation and repair of damaged concrete wall components.

Concrete-encased steel composite components behave differently from concrete sections reinforced with steel reinforcement. Concrete-encased steel composite components frequently behave as over-reinforced sections. This type of component behavior was not represented in the data sets used to develop the force-deformation modeling relationships and acceptance criteria in this standard, and is not covered in this standard. Concrete encasement is often provided for fire protection rather than for strength or stiffness and typically lacks transverse reinforcement. In some cases, the transverse reinforcement does not meet detailing requirements in **AISC 360**. Lack of adequate confinement can result in lateral expansion of the core concrete, which exacerbates bond slip and undermines the fundamental principle that plane sections remain plane.

Testing and analysis used to determine acceptance criteria for concrete-encased steel composite components should include the effect of bond slip between steel and concrete, confinement ratio, confinement reinforcement detailing, kinematics, and appropriate strain limits.

To preserve historic buildings, exercise care in selecting the appropriate retrofit approaches and techniques for application.

STANDARD

COMMENTARY

CHAPTER 2—MATERIAL PROPERTIES AND
CONDITION ASSESSMENT

2.1—General

Mechanical properties of materials shall be obtained from available drawings, specifications, and other documents for the existing building in accordance with the requirements of **ASCE 41-17**, 3.2. Where these documents fail to provide adequate information to quantify material properties, such information shall be supplemented by materials testing based on requirements of Chapter 2. Material properties of existing concrete components shall be determined in accordance with 2.2. The use of default material properties based on historical information is permitted in accordance with 2.2.5. A condition assessment shall be conducted in accordance with 2.3. The extent of materials testing and condition assessment performed shall be used to determine the knowledge factor as specified in 2.4.

2.2—Properties of in-place materials and
components2.2.1 *Material properties*

C2.1—General

Chapter 2 identifies properties requiring consideration and provides requirements for determining building properties. Also described is the need for a thorough condition assessment and use of knowledge gained in analyzing component and system behavior. Personnel involved in material property quantification and condition assessment should be experienced in the proper implementation of testing practices and the interpretation of results.

When modeling a concrete building, it is important to investigate local practices relative to seismic design. Specific benchmark years can be determined for the implementation of earthquake-resistant design in most locations, but caution should be exercised in assuming optimistic characteristics for any specific building. Particularly with concrete materials, the date of original building construction significantly influences seismic performance. Without deleterious conditions or materials, concrete gains compressive strength from the time it is originally cast and in place. Strengths typically exceed specified design values (28-day or similar). In older construction, concrete strength was often very low (less than 3000 psi) and it was rarely specified in the drawings. Early adoptions of concrete in buildings often used steel reinforcement with relatively low strength and ductility, limited continuity, and reduced bond development. Continuity between specific existing components and elements, such as beams, columns, diaphragms, and shear walls, can be particularly difficult to assess because of concrete cover and other barriers to inspection.

Properties of welded wire reinforcement for various periods of construction can be obtained from the **Wire Reinforcement Institute (2009)**.

Documentation of the material properties and grades used in component and connection construction is invaluable and can be effectively used to reduce the amount of in-place testing required. The licensed design professional is encouraged to research and acquire all available records from original construction, including photographs, to confirm reinforcement details shown on the plans.

Further guidance on the condition assessment of existing concrete buildings can be found in the following:

- a) **ACI 201.1R**, which provides guidance on conducting a condition survey of existing concrete structures
- b) **ACI 364.1R**, which describes the general procedures used for the evaluation of concrete structures before retrofit
- c) **ACI 437R**, which describes methods for strength evaluation of existing concrete buildings, including analytical and load test methods

C2.2—Properties of in-place materials and
componentsC2.2.1 *Material properties*

STANDARD

2.2.1.1 General—The following component and connection material properties shall be obtained for the as-built structure:

- a) Concrete compressive strength
- b) Yield and ultimate strength of nonprestressed and prestressed steel reinforcement, cast-in-place and post-installed anchors, and metal connection hardware

Where materials testing is required by **ASCE 41-17**, 6.2, the test methods to quantify material properties shall comply with the requirements of 2.2.3. The frequency of sampling, including the minimum number of tests for property determination, shall comply with the requirements of 2.2.4.

2.2.1.2 Nominal or specified properties—Nominal material properties, or properties specified in construction documents, shall be taken as lower-bound material properties. Corresponding expected material properties shall be calculated by multiplying lower-bound values by a factor taken from Table 1 to translate from lower-bound to expected values. Alternative factors shall be permitted where justified by test data.

Table 1—Factors to translate lower-bound material properties to expected strength material properties

Material property	Factor
Concrete compressive strength	1.50
Steel reinforcement tensile and yield strength	1.25
Connector steel yield strength	1.50

2.2.2 Component properties—The following component properties and as-built conditions shall be established:

- a) Cross-sectional dimensions of individual components and overall configuration of the structure
- b) Configuration of component connections, size, embedment depth, type of anchors, thickness of connector material, anchorage and interconnection of embedments, and the presence of bracing or stiffening components

COMMENTARY

C2.2.1.1 General—Other material properties and conditions of interest for concrete components include:

- a) Tensile strength and modulus of elasticity of concrete
- b) Ductility, toughness, and fatigue properties of concrete
- c) Carbon equivalent present in the steel reinforcement
- d) Presence of any degradation such as corrosion or deterioration of bond between concrete and reinforcement

The extent of effort made to determine these properties depends on availability of accurate, updated construction documents and drawings; construction quality and type; accessibility; and material conditions. The analysis method selected—for example, linear static procedure (LSP) or nonlinear static procedure (NSP)—might also influence the testing scope. Concrete tensile strength and modulus of elasticity can be estimated based on the compressive strength and may not warrant the damage associated with any extra coring required.

The sample size and removal practices followed are referenced in **FEMA 274**, C6.3.2.3 and C6.3.2.4. **ACI 228.1R** provides guidance on methods to estimate the in-place strength of concrete in existing structures, whereas **ACI 214.4R** provides guidance on coring in existing structures and interpretation of core compressive strength test results. Generally, mechanical properties for both concrete and steel reinforcement can be established from combined core and specimen sampling at similar locations, followed by laboratory testing. Core drilling should minimize damage to the existing steel reinforcement.

C2.2.2 Component properties—Component properties are required to properly characterize building performance in seismic analysis. The starting point for assessing component properties and condition is retrieval of available construction documents. A preliminary review should identify primary gravity and seismic-force-resisting elements and systems and their critical components and connections. If there are no drawings of the building, a design professional should perform a thorough investigation of the

STANDARD

- c) Modifications to components or overall configuration of the structure
- d) Most recent physical condition of components and connections, and the extent of any deterioration
- e) Deformations beyond those expected because of gravity loads, such as those caused by settlement or past earthquake events
- f) Presence of other conditions that influence building performance, such as nonstructural components that can interact with structural components during earthquake excitation

2.2.3 Test methods to quantify material properties

2.2.3.1 General—Destructive and nondestructive test methods used to obtain in-place mechanical properties of materials identified in 2.2.1 and component properties identified in 2.2.2 are specified in this section. Samples of concrete, reinforcement, and connector steel shall be examined for physical condition, as specified in 2.3.2.

When determining material properties with the removal and testing of samples for laboratory analysis, sampling shall take place in primary gravity and seismic-force-resisting components in regions with the least stress.

Where 2.2.4.2.1 does not apply and the coefficient of variation is greater than 20 percent, the expected concrete strength shall not exceed the mean less one standard deviation.

2.2.3.2 Sampling—For concrete material testing, the sampling program shall include the removal of standard cores. Core drilling shall be preceded by nondestructive location of the steel reinforcement, and core holes shall be located to avoid damage to or drilling through the steel reinforcement. Core holes shall be filled with concrete or grout of comparable strength having nonshrinkage properties. If nonprestressed steel reinforcement is tested, sampling shall include removal of local bar segments and installation of replacement spliced material to maintain continuity of the reinforcing bar for transfer of bar force unless an analysis confirms that replacement of the original components is not required.

Removal of core samples and performance of laboratory destructive testing shall be permitted to determine existing concrete strength properties. Removal of core samples shall use the procedures included in **ASTM C42/C42M**. Testing shall follow the procedures contained in **ASTM C42/C42M**, **ASTM C39/C39M**, and **ASTM C496/C496M**. Core strength shall be converted to in-place concrete compressive strength by an approved procedure.

Removal of bar or tendon samples and performance of laboratory destructive testing shall be permitted to determine existing steel reinforcement strength properties. The tensile yield and ultimate strengths for reinforcing and prestressing steels shall follow the procedures included in **ASTM A370**. Reinforcement samples that are slightly damaged during removal are permitted to be machined to a round bar as long as the tested area is at least 70 percent of the gross area of the original bar. Prestressing materials shall meet the supple-

COMMENTARY

building to identify these elements, systems, and components as described in 2.3.

C2.2.3.2 Sampling—**ACI 214.4R** and **FEMA 274** provide further guidance on correlating concrete core strength to in-place strength and provide references for various test methods that can be used to estimate material properties. Chemical composition can be determined from retrieved samples to assess the condition of the concrete. Section C6.3.3.2 of FEMA 274-97 provides references for these tests.

When concrete cores are taken, care should be taken when patching the holes. For example, a core through the thickness of a slab should have positive anchorage by roughening the surface and possibly dowels for anchorage. For that case, the holes should be filled with concrete or grout and the engineer should provide direction for filling the hole so that the added concrete or grout bonds to the substrate.

The steel reinforcement system used in the construction of a specific building is usually of uniform grade and similar strength. One grade of reinforcement is occasionally used for small-diameter bars, such as those used for stirrups and hoops, and another grade for large-diameter bars, such as those used for longitudinal reinforcement. In some cases, different concrete design strengths or classes are used. Historical research and industry documents contain insight on material mechanical properties used in different construction eras (2.2.5). This information can be used with laboratory and field test data to gain confidence in in-place strength properties. Undamaged steel reinforcement can be reduced to a smooth bar if the samples meet the requirements of **ASTM A370**, excluding the limitations of Annex 9. This

STANDARD

mental requirements in **ASTM A416/A416M**, **ASTM A421/A421M**, or **ASTM A722/A722M**, depending on material type. Properties of connector steels shall be permitted to be determined by wet and dry chemical composition tests, and direct tensile and compressive strength tests as specified by **ASTM A370**. Where strength, construction quality, or both, of anchors or embedded connectors are required to be determined, in-place testing shall satisfy the provisions of **ASTM E488/E488M**.

2.2.4 Minimum number of tests—Materials testing is not required if material properties are available from original construction documents that include material test records or reports. Material test records or reports shall be representative of all critical components of the building structure.

Based on 6.2 of **ASCE 41-17**, data collection from material tests is classified as either comprehensive or usual. The minimum number of tests for usual data collection is specified in 2.2.4.1. The minimum number of tests necessary to quantify properties by in-place testing for comprehensive data collection is specified in 2.2.4.2. If the existing gravity-load-resisting system or seismic-force-resisting system is replaced during the retrofit process, material testing is only required to quantify properties of existing materials at new connection points.

2.2.4.1 Usual data collection—The minimum number of tests to determine concrete and steel reinforcement material properties for usual data collection shall be based on the following criteria:

- a) If the specified design strength of the concrete is known, at least one core shall be taken from samples of each different concrete strength used in the construction of the building, with a minimum of three cores taken for the entire building.
- b) If the specified design strength of the concrete is not known, at least one core shall be taken from each type of seismic-force-resisting component, with a minimum of six cores taken for the entire building.
- c) If the specified design strength of the steel reinforcement is known, nominal or specified material properties shall be permitted without additional testing.
- d) If the specified design strength of the steel reinforcement is not known, at least two strength test coupons of steel reinforcement shall be removed from the building for testing.
- e) Cast-in-place or post-installed anchors shall be classified in groups of similar type, size, geometry, and structural use. In groups of anchors used for out-of-plane wall anchorage and in groups of anchors whose failure in tension or shear would cause the structure not to meet the selected

COMMENTARY

type of reinforcement would occur in a situation where only a limited length of bar can be removed for testing.

C2.2.4 Minimum number of tests—To quantify in-place properties accurately, it is essential that a minimum number of tests be conducted on primary components of the seismic-force-resisting system. The minimum number of tests is dictated by the availability of original construction data, structural system type used, desired accuracy, quality and condition of in-place materials, level of seismicity, and target performance level. Accessibility to the structural system can influence the testing program scope. The focus of testing should be on primary seismic-force-resisting components and specific properties for analysis. Test quantities provided in this section are minimal; the licensed design professional should determine whether further testing is needed to evaluate as-built conditions.

Testing is generally not required on components other than those of the seismic-force-resisting system.

The licensed design professional and subcontracted testing agency should carefully examine test results to verify that suitable sampling and testing procedures were followed and appropriate values for the analysis were selected from the data.

STANDARD

COMMENTARY

performance objective, 5 percent of the anchors with a minimum of three anchors of each anchor group shall be tested in-place in tension to establish an available strength, construction quality, or both. The test load shall be specified by the licensed design professional and shall be based on the anticipated demand or strength in accordance with available construction information. If the test load is used as the basis for anchor strength calculation, the available anchor strength shall not be taken greater than two-thirds of the test load. Testing of the anchors to failure is not required and a test load lower than the expected failure load shall be permitted. If the test load is not achieved in one or more anchors tested in a group, anchors in that group shall be tested under a tensile load smaller than that specified for the preceding tests. Otherwise, the strength of the tested anchor group shall be ignored. Testing in accordance with 2.2.4.2.5 shall be permitted to determine the available strength based on a statistical distribution of the test results.

2.2.4.2 Comprehensive data collection

2.2.4.2.1 Coefficient of variation—Unless specified otherwise, a minimum of three tests shall be conducted to determine any property. If the coefficient of variation exceeds 20 percent, additional tests shall be performed until the coefficient of variation is equal to or less than 20 percent. If additional testing does not reduce the coefficient of variation below 20 percent, a knowledge factor reduction per 4.4 shall be used. In determining coefficient of variation, cores shall be grouped by grades of concrete and element type. The number of tests in a single component shall be limited to prevent compromising the integrity of the component.

2.2.4.2.2 Concrete materials—For each concrete element type of the seismic-force-resisting system, as well as secondary systems for which failure could result in a collapse hazard, a minimum of three core samples shall be taken and subjected to compression tests. A minimum of six total tests shall be performed on a building for concrete strength determination, subject to the limitations of this section. If varying concrete classes or grades were used in the building construction, a minimum of three samples and tests shall be performed for each class and grade. The modulus of elasticity and tensile strength shall be permitted to be estimated from the compressive strength testing data. Samples shall be taken from components, distributed throughout the building, that are critical to the structural behavior of the building.

Tests shall be performed on samples from components that are identified as damaged or degraded to quantify their condition. Test results from areas of degradation shall be compared with strength values specified in the construction documents. If test values less than the specified strength in the construction documents are found, further strength testing shall be performed to determine the cause or identify the degree of damage or degradation.

C2.2.4.2.2 Concrete materials—ACI 214.4R provides guidance on coring in existing structures and interpretation of core compressive strength test results. If a structure was constructed in phases or if construction documents for different parts of the structure were issued at separate times, the licensed design professional, to determine sampling size, should consider the concrete in each construction phase or in each set of construction documents a different type. Section 6.4.3 of ACI 562-16 provides a method to calculate an equivalent specified concrete strength, f'_c , based on statistical analysis of compression strength test results from core samples. ASTM E178 provides guidance on consideration of outliers in a set of core samples. Equation (6.4.3) of ACI 562-16 defines the equivalent specified compressive strength of concrete as a function of the number of tests, the coefficient of variation of the samples, and a factor to account for the number of samples. Section 6.4.3 of ACI 562-16 permits the engineer to select the number of samples used to evaluate concrete compressive strength but imposes a penalty to the results to account for the uncertainty associated with the number of samples.

Equation (6.4.3) of ACI 562-16 was derived with the objective of calculating the 13 percent fractile of the in-place

STANDARD

The minimum number of tests to determine compressive strength of each concrete element type shall conform to one of the following criteria:

a) For concrete elements for which the specified design strength is known and test results are not available, a minimum of three core tests shall be conducted for each floor level, 400 yd³ of concrete, or 10,000 ft² of surface area, whichever requires the most frequent testing.

b) For concrete elements for which the specified design strength is unknown and test results are not available, a minimum of six core tests shall be conducted for each floor level, 400 yd³ of concrete, or 10,000 ft² of surface area, whichever requires the most frequent testing. Where the results indicate that different classes of concrete were used, the degree of testing shall be increased to confirm class use.

c) Alternately, for concrete elements for which the design strength is known or unknown, and test results are not available, it is permitted to determine the lower-bound compressive strength based on core sample testing and applying the provisions in 6.4.3 of ACI 562-16. If the lower-bound compressive strength is determined in this manner, the expected compressive strength shall be determined as the lower-bound compressive strength value obtained from ACI 562-16, Eq. (6.4.3) plus one standard deviation of the strength of the core samples. When following the provisions in 6.4.3 of ACI 562-16, the minimum number of samples per element type shall be four. The sample locations shall be:

a) Distributed to quantify element material properties throughout the height of the building

b) Distributed to quantify element material properties in locations critical to the structural system being investigated

Quantification of concrete strength via ultrasonics or other nondestructive test methods shall not be substituted for core sampling and laboratory testing.

COMMENTARY

concrete compressive strength, which some studies have shown to be approximately equal to the specified compressive strength of concrete, f'_c (Bartlett and MacGregor 1996). The first term in Eq. (6.4.3) of ACI 562-16 represents the effect of sample size on the uncertainty of the mean in-place strength, where the coefficient k_c is obtained from a student's t distribution with $n - 1$ degrees of freedom and a 90 percent confidence level. The second term in Eq. (6.4.3) of ACI 562-16 represents the uncertainty attributable to correction factors relating cylinder strength to specified compressive strength, which were assumed to have a normal distribution, also estimated with a 90 percent confidence level. The study by Bartlett and MacGregor (1996) showed that the specified compressive strength f'_c , corresponds approximately to the 13 percent fractile of the 28-day in-place strength in walls and columns, and approximately the 23 percent fractile of the 28-day in-place compressive strength in beams and slabs. The former was considered to be a more appropriate measure of specified compressive strength f'_c than the latter because the nominal strength of columns is more sensitive to concrete compressive strength than the strength of beams and slabs (ACI 214.4R).

In 2.2.1.2, it is stated that nominal material properties or properties specified in construction documents shall be taken as lower-bound material properties unless otherwise specified. The method to estimate of the specified concrete compressive strength f'_c in 6.4.3 of ACI 562-16 was adopted in this standard to obtain the lower-bound compressive strength consistent with the provisions in 2.2.1.2.

ACI 214.4R provides guidance on coring in existing structures and interpretation of core compressive strength test results. The minimum of four samples was adopted based on the recommendations in ACI 214.4R. The following equation is provided in ACI 214.4R.

$$n_{\text{samples}} = \left(\frac{2COV_{\text{population}}}{e_{\text{population}}} \right)^2 \quad (C1)$$

where n_{samples} represents the minimum number of samples; $COV_{\text{population}}$ represents the estimated coefficient of variation of the population; and $e_{\text{population}}$ represents the predetermined maximum error expressed as a percentage of the population average. For a total of four samples, the previous equation dictates that the maximum error is equal to the estimate of the coefficient of variation of the population. Bartlett and MacGregor (1995) report that for many batches of cast-in-place concrete, and samples obtained from many members, the coefficient of variation was approximately 13 percent. If the maximum error is equal to the coefficient of variation, a maximum error of 13 percent corresponds to approximately 1.13 standard deviations, which is considered adequate for an estimate of lower-bound material properties.

Users of the document are cautioned that, for coefficients of variation between 13 and 20 percent, the minimum number of samples needed to limit the error below one

STANDARD

COMMENTARY

standard deviation according to the recommendations in **ACI 214.4R** is higher than four. For example, for a coefficient of variation of 20 percent, a minimum of seven samples is recommended to limit the error to one standard deviation. If the maximum error is reduced to 10 percent, the minimum number of samples recommended is significantly higher. For a coefficient of variation of 15.87 percent (one standard deviation away from the mean) and a maximum error of 10 percent, the minimum number of samples recommended is 11, and for a coefficient of variation of 20 percent and a maximum error of 10 percent, the minimum number of samples recommended is 16. If the coefficient of variation exceeds 20 percent, the requirements in 2.2.4.2.1 are satisfied.

Ultrasonics and nondestructive test methods should not be substituted for core sampling and laboratory testing as they do not yield accurate strength values directly. These methods should only be used for confirmation and comparison. Guidance for nondestructive test methods is provided in **ACI 228.2R**.

2.2.4.2.3 *Nonprestressed reinforcing and connector steels—*

Tests shall be conducted to determine both yield and ultimate strengths of reinforcing and connector steel. Connector steel is defined as additional structural steel or miscellaneous metal used to secure precast and other concrete shapes to the building structure. A minimum of three tensile tests shall be conducted on nonprestressed steel reinforcement samples from a building for strength determination, subject to the following supplemental conditions:

- a) If original construction documents defining properties exist, then at least three strength coupons shall be removed from random locations from each element or component type and tested.
- b) If original construction documents defining properties are unavailable, but the approximate date of construction is known and a common material grade is confirmed, at least three strength coupons shall be removed from random locations from each element or component type for every three floors of the building.
- c) If the construction date is unknown, at least six strength coupons for every three floors shall be performed.

Refer to 2.2.3.2 for replacement of sampled material.

2.2.4.2.4 *Prestressing steel—*Sampling prestressing steel tendons for laboratory testing shall only be performed on prestressed components that are part of the seismic-force-resisting system. Prestressed components in diaphragms shall be permitted to be excluded.

Tendon or prestress removal shall be avoided if possible. Any sampling of prestressing steel tendons for laboratory testing shall be done with extreme care. It shall be permitted to determine material properties without tendon or prestress removal by careful sampling of either the tendon grip or the extension beyond the anchorage, if sufficient length is available.

STANDARD

All sampled prestressed steel shall be replaced with new, fully connected, and stressed material and anchorage hardware, unless an analysis confirms that replacement of original components is not required.

2.2.4.2.5 Cast-in-place or post-installed anchors—Cast-in-place or post-installed anchors shall be classified in groups in accordance with 2.2.4.1. In groups of anchors used for out-of-plane wall anchorage and in groups of anchors whose failure in tension or shear would cause the structure not to meet the selected performance objective, 10 percent of the anchors with a minimum of six anchors of each anchor group shall be tested in-place in tension to establish an available strength, construction quality, or both. Testing of the anchors to failure is not required. The test load shall be specified by the licensed design professional and shall be based on the anticipated demand or strength in accordance with available construction information. If the test load is used as the basis for anchor strength calculation, the available anchor strength shall not be taken greater than two-thirds the test load. Testing of the anchors to failure is not required and a test load lower than the expected failure load shall be permitted.

2.2.5 Default properties—Default material properties to determine component strengths shall be permitted to be used in conjunction with the linear analysis procedures of ASCE 41-17 Chapter 7.

Default lower-bound concrete compressive strengths are specified in Table 2. Default expected concrete compressive strengths shall be determined by multiplying lower-bound values by an appropriate factor selected from Table 1, unless another factor is justified by test data. The appropriate default compressive strength, lower-bound strength, or expected strength as specified in ASCE 41-17, 7.5.1.3, shall be used to establish other strength and performance characteristics for the concrete as needed in the structural analysis.

Default lower-bound values for steel reinforcement are specified for various ASTM specifications and periods in Tables 3 and 4. Default expected strength values for steel reinforcement shall be determined by multiplying lower-bound values by an appropriate factor selected from Table 1, unless another factor is justified by test data. Where default values are assumed for existing steel reinforcement, welding or mechanical coupling of new reinforcement to the existing steel reinforcement shall not be permitted.

The default lower-bound yield strength for steel connector material shall be taken as 27,000 lb/in.² The default expected yield strength for steel connector material shall be deter-

COMMENTARY

C2.2.4.2.5 Cast-in-place or post-installed anchors—To estimate ultimate strength of the anchors in accordance with 3.6, the frequency of the test should be increased to at least 25 percent of the anchors and the test load should be at least the nominal design strength in accordance with Chapter 17 of ACI 318-14. In-place anchor testing performed in accordance with 2.2.4.2.5 provides the minimum available tensile strength of a single anchor, which is likely governed by pullout or bond strength in tension. Other failure modes and parameters that affect the strength of the anchors, such as proximity to edges, group effect, presence of cracks, or eccentricity of applied loads, should be considered in accordance with Chapter 17 of ACI 318-14.

C2.2.5 Default properties—Default values provided in this standard are generally conservative. Whereas the strength of steel reinforcement can be fairly consistent throughout a building, the strength of concrete in a building could be highly variable, given variability in concrete mixtures and sensitivity to water-cement ratio (w/c) and curing practices. A conservative assumption based on the field observation of the concrete compressive strength in the given range is recommended unless a higher strength is substantiated by construction documents, test reports, or material testing. For the capacity of an element in question, the lower value within the range can be conservative. It can be appropriate to use the maximum value in a given range where determining the force-controlled actions on other components.

Until approximately 1920, a variety of proprietary steel reinforcements was used. Yield strengths are likely to be in the range of 33,000 to 55,000 lb/in.², but higher values are possible and actual yield and tensile strengths can exceed minimum values. Once commonly used to designate steel reinforcement grade, the terms “structural”, “intermediate”, and “hard” became obsolete in 1968. Plain and twisted square bars were occasionally used between 1900 and 1949. Factors to convert default steel reinforcement strength to expected strength include consideration of material over-strength and strain hardening.

Table 2—Default lower-bound compressive strength of structural concrete, lb/in.²

Time frame	Footings	Beams	Slabs	Columns	Walls
1900 to 1919	1000 to 2500	2000 to 3000	1500 to 3000	1500 to 3000	1000 to 2500
1920 to 1949	1500 to 3000	2000 to 3000	2000 to 3000	2000 to 4000	2000 to 3000
1950 to 1969	2500 to 3000	3000 to 4000	3000 to 4000	3000 to 6000	2500 to 4000
1970 to present	3000 to 4000	3000 to 5000	3000 to 5000	3000 to 10,000	3000 to 5000

STANDARD

COMMENTARY

Table 3—Default lower-bound tensile and yield properties of reinforcing steel for various periods

Year	Grade	Structural*	Intermediate*	Hard*	60	65	70	75
		33	40	50				
	Minimum yield, psi	33,000	40,000	50,000	60,000	65,000	70,000	75,000
	Minimum tensile, psi	55,000	70,000	80,000	90,000	75,000	80,000	100,000
1911 to 1959		x	x	x		x		
1959 to 1966		x	x	x	x	x	x	x
1966 to 1972			x	x	x	x	x	
1972 to 1974			x	x	x	x	x	
1974 to 1987			x	x	x	x	x	
1987 to present			x	x	x	x	x	x

*The terms “structural”, “intermediate”, and “hard” became obsolete in 1968.

Notes: An entry of x indicates that the grade was available in those years.

Table 4—Default lower-bound tensile and yield properties of reinforcing steel for various ASTM specifications and periods

			ASTM grade	Structural*	Intermediate*	Hard*				
				33	40	50	60	65	70	75
			Minimum yield, psi	33,000	40,000	50,000	60,000	65,000	70,000	75,000
			Minimum tensile, psi	55,000	70,000	80,000	90,000	75,000	80,000	100,000
ASTM designation†	Steel type	Year range								
A15 (withdrawn)	Billet	1911 to 1966		x	x	x				
A16 (withdrawn)	Rail‡	1913 to 1966				x				
A61 (withdrawn)	Rail‡	1963 to 1966					x			
A160 (withdrawn)	Axle	1936 to 1964		x	x	x				
A160 (withdrawn)	Axle	1965 to 1966		x	x	x	x			
A185	WWR	1936 to present						x		
A408 (withdrawn)	Billet	1957 to 1966		x	x	x				
A431	Billet	1959 to 1966								x
A432 (withdrawn)	Billet	1959 to 1966					x			
A497	WWR	1964 to present							x	
A615/A615M (2003c)	Billet	1968 to 1972			x		x			x
A615/A615M (2003c)	Billet	1974 to 1986			x		x			
A615/A615M (2003c)	Billet	1987 to present			x		x			x
A616§ (withdrawn)	Rail‡	1968 to 1999				x	x			
A617 (withdrawn)	Axle	1968 to 1999			x		x			
A996	Rail, Axle	2000 to present			x	x	x			
A706/A706M#	Low-alloy	1974 to present					x			
A955	Stainless	1996 to present			x		x			x

Note: An entry of x indicates that the grade was available in those years.

*The terms “structural”, “intermediate”, and “hard” became obsolete in 1968.

†ASTM steel is marked with the letter W.

‡Rail bars are marked with the letter R.

§Bars marked s! (ASTM A616 [withdrawn]) have supplementary requirements for bend tests.

#ASTM A706/A706M has a minimum tensile strength of 80,000 lb/in.2, but not less than 1.25 times the actual yield strength.



STANDARD

mined by multiplying lower-bound values by an appropriate factor selected from Table 1, unless another value is justified by test data.

The default lower-bound yield strength for cast-in-place or post-installed anchor material shall be taken as 27,000 lb/in.² unless another value is justified by test data. Component actions on the connections shall be considered as force-controlled actions and default expected yield strength shall not be used.

The use of default values for prestressing steel in prestressed concrete construction shall not be permitted.

2.3—Condition assessment

2.3.1 General—A condition assessment of the existing building and site conditions shall be performed as specified in this section.

The condition assessment shall include the following:

- a) Examination of the physical condition of primary and secondary components, and the presence of any degradation shall be noted
- b) Verification of the presence and configuration of components and their connections, and the continuity of load paths between components, elements, and systems
- c) A review and documentation of other conditions, including neighboring party walls and buildings, presence of nonstructural components and mass, and prior remodeling
- d) Collection of information needed to select a knowledge factor in accordance with 4.4
- e) Confirmation of component orientation, plumbness, and physical dimensions

2.3.2 Scope and procedures—The scope of the condition assessment shall include critical structural components as described in the following subsections.

2.3.2.1 Visual condition assessment—Direct visual inspection of accessible and representative primary components and connections shall be performed to:

- a) Identify configuration issues
- b) Determine if degradation is present
- c) Establish continuity of load paths
- d) Establish the need for other test methods to quantify the presence and degree of degradation
- e) Measure dimensions of existing construction to compare with available design information and reveal any permanent deformations

A visual building inspection shall include visible portions of foundations, seismic-force-resisting members, diaphragms (slabs), and connections. As a minimum, a representative sampling of at least 20 percent of the components and connections shall be visually inspected at each floor level. If significant damage or degradation is found, the assessment sample of all similar-type critical components in the building shall be increased to 40 percent or more, as necessary, to accurately assess the performance of components and connections with degradation.

COMMENTARY

C2.3—Condition assessment

C2.3.1 General—The condition assessment also affords an opportunity to review other conditions that can influence concrete elements and systems and overall building performance. Of particular importance is the identification of other elements and components that can contribute to or impair the performance of the concrete system in question, including infills, neighboring buildings, and equipment attachments. Limitations posed by existing coverings, wall and ceiling space, infills, and other conditions shall also be defined such that prudent retrofit measures can be planned.

C2.3.2.1 Visual condition assessment—Further guidance can be found in [ACI 201.1R](#), which provides a system for reporting the condition of concrete in service.

STANDARD

If coverings or other obstructions exist, partial visual inspection through the obstruction shall be permitted to be performed using drilled holes and a fiberscope.

2.3.2.2 Comprehensive condition assessment—Exposure is defined as local minimized removal of cover concrete and other materials to inspect reinforcing system details. All damaged concrete cover shall be replaced after inspection. The following criteria shall be used for assessing primary connections in the building for comprehensive data collection:

a) If detailed design drawings exist, exposure of at least three different primary connections shall occur, with the connection sample including different types of connections (for example, beam-column, column-foundation, beam-diaphragm, and diaphragm-wall). If no deviations from the drawings exist or if the deviations from the drawings are consistently similar, it shall be permitted to consider the sample as being representative of installed conditions. If inconsistent deviations are noted, then at least 25 percent of the specific connection type shall be inspected to identify the extent of deviation.

b) In the absence of detailed design drawings, at least three connections of each primary connection type shall be exposed for inspection. If common detailing among the three connections is observed, it shall be permitted to consider this condition representative of installed conditions. If variations are observed among like connections, additional connections shall be inspected until an accurate understanding of building construction is gained.

2.3.2.3 Additional testing—If additional destructive and nondestructive testing is required to determine the degree of damage or presence of deterioration, or to understand the internal condition and quality of concrete, test methods approved by the licensed design professional shall be used.

COMMENTARY

C2.3.2.3 Additional testing—The physical condition of components and connectors affects their performance. The need to accurately identify the physical condition can dictate the need for certain additional destructive and nondestructive test methods. Such methods can be used to determine the degree of damage or presence of deterioration and to improve understanding of the internal condition and concrete quality. Further guidelines and procedures for destructive and nondestructive tests that can be used in the condition assessment are provided in [ACI 228.1R](#), [ACI 228.2R](#), [FEMA 274](#), and [FEMA 306](#).

The nondestructive examination (NDE) methods having the greatest use and applicability to condition assessment are listed in the following:

a) Surface NDE methods include infrared thermography, delamination sounding, surface hardness measurement, and crack mapping. These methods can be used to find surface degradation in components such as service-induced cracks, corrosion, and construction defects.

b) Volumetric NDE methods, including radiography and ultrasonics, can be used to identify the presence of internal discontinuities and loss of section. Impact-echo ultrasonics is often used and is a well-understood technology.

c) Online monitoring using acoustic emissions, strain gauges, in-place static or dynamic load tests, and ambient vibration tests can be used to assess structural condition.

STANDARD

COMMENTARY

2.3.3 Basis for mathematical building model—Results of the condition assessment shall be used to quantify the following items needed to create the mathematical building model:

- a) Component section properties and dimensions
- b) Component configuration and the presence of any eccentricities or permanent deformation
- c) Connection configuration and the presence of any eccentricities
- d) Presence and effect of alterations to the structural system since original construction
- e) Interaction of nonstructural components and their involvement in seismic force resistance

All deviations between available construction records and as-built conditions obtained from visual inspection shall be accounted for in the structural analysis.

Unless concrete cracking, reinforcement corrosion, or other mechanisms of degradation are observed in the condition assessment as the cause for damage or reduced capacity, the cross-sectional area and other sectional properties shall be assumed to be those from the design drawings after adjustment for as-built conditions. If some sectional material loss has occurred, the loss shall be quantified by direct measurement and sectional properties reduced accordingly using the principles of structural mechanics.

2.4—Knowledge factor

A knowledge factor κ for computation of concrete component acceptance criteria shall be selected in accordance with [ASCE 41-17](#), 6.2.4, with additional requirements specific to concrete components. A knowledge factor κ equal to 0.75 shall be used if any of the following criteria are met:

- a) Components are found to be damaged or deteriorated during assessment, and further testing is not performed to quantify their condition or justify the use of higher values of κ
- b) Mechanical properties have a coefficient of variation exceeding 20 percent
- c) Components contain archaic or proprietary material and the condition is uncertain

and performance. Monitoring is used to determine if active degradation or deformations are occurring, whereas nondestructive load testing provides direct insight on load-carrying capacity.

d) Electromagnetic methods using a pachometer or radiography can be used to locate, size, or perform an initial assessment of steel reinforcement. Further assessment of suspected corrosion activity should use electrical half-cell potential and resistivity measurements.

e) Lift-off testing (assuming original design and installation data are available), or another nondestructive method such as the coring stress relief specified in [SEI/ASCE 11](#), can be used where absolutely essential to determine the level of prestress remaining in an unbonded prestressed system.

STANDARD

COMMENTARY

CHAPTER 3—GENERAL ASSUMPTIONS AND REQUIREMENTS

3.1—Modeling and design

3.1.1 General—Seismic retrofit of a concrete building involves the design of new components connected to the existing structure, seismic upgrading of existing components, or both. New components shall comply with ACI 318, except as otherwise indicated in this standard.

Original and retrofitted components of an existing building are not expected to satisfy provisions of ACI 318 but shall be assessed using the provisions of this standard. Brittle or low-ductility failure modes shall be identified as a part of the seismic evaluation.

Evaluation of demands and capacities of reinforced concrete components shall include consideration of locations along the length where seismic force and gravity loads produce maximum effects; where changes in cross section or reinforcement result in reduced strength; and where abrupt changes in cross section or reinforcement, including splices, can produce stress concentrations that result in premature failure.

3.1.2 Stiffness—Component stiffnesses shall be calculated considering shear, flexure, axial behavior, and reinforcement slip deformations. Stress state of the component, cracking extent caused by volumetric changes from temperature and shrinkage, deformation levels under gravity loads, and seismic forces shall be considered. Gravity load effects considered for effective stiffnesses of components shall be determined using ASCE 41-17 Eq. (7-3).

3.1.2.1 Linear procedures—Where design actions are determined using the linear procedures of ASCE 41-17 Chapter 7, component effective stiffnesses shall correspond to the secant value to the yield point of the component. Alternate stiffnesses shall be permitted where it is demonstrated by analysis to be appropriate for the design loading. Alternatively, effective stiffness values in Table 5 shall be permitted.

C3.1—Modeling and design

C3.1.1 General—Brittle or low-ductility failure modes typically include behavior in direct or nearly direct compression; shear in slender components and in-component connections; torsion in slender components; and reinforcement development, splicing, and anchorage. The stresses, forces, and moments acting to cause these failure modes should be determined from a limit-state analysis, considering probable resistances at locations of nonlinear action.

C3.1.2 Stiffness—For columns with low axial loads (below approximately $0.1A_gf'_c$), deformations caused by bar slip can account for as much as 50 percent of the total deformations at yield. Further guidance regarding calculation of the effective stiffness of reinforced concrete columns that include the effects of flexure, shear, and bar slip can be found in Elwood and Eberhard (2009).

Flexure-controlled wall stiffness can vary from approximately $0.15E_cEI_g$ to $0.5E_cEI_g$, depending on wall reinforcement and axial load. A method for calculating wall stiffness, which provides compatibility with fiber section analysis, is offered in C7.2.2.

C3.1.2.1 Linear procedures—The effective flexural rigidity values in Table 5 for beams and columns account for the additional flexibility from reinforcement slip within the beam-column joint or foundation before yielding. The values specified for columns were determined based on a database of 221 rectangular reinforced concrete column tests with axial loads less than $0.67A_gf'_c$ and shear span-depth ratios greater than 1.4. Measured effective stiffnesses from the laboratory test data suggest that the effective flexural rigidity for low axial loads could be approximated as $0.2EI_g$; however, considering the scatter in the effective flexural rigidity and to avoid underestimating the shear demand on columns with low axial loads, $0.3EI_g$ is recommended in Table 5 (Elwood et al. 2007). In addition to axial load, the shear span-depth ratio of the column influences the effective flexural rigidity. A more refined estimate of the effective flexural rigidity can be determined by calculating the displacement at yield caused by flexure, slip, and shear (Elwood and Eberhard 2009).

The modeling recommendations for beam-column joints (6.2.2.1) do not include the influence of reinforcement slip.

STANDARD

COMMENTARY

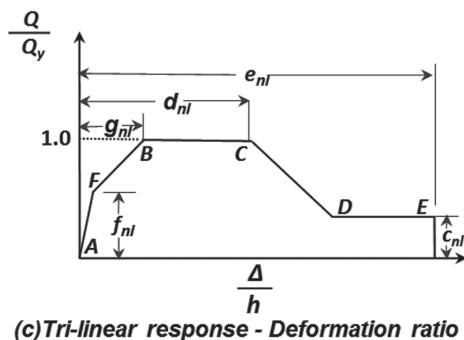
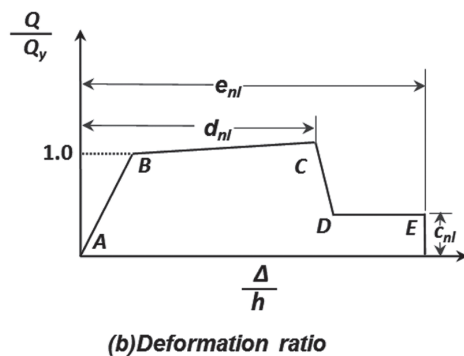
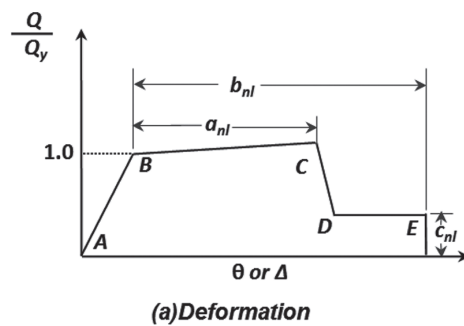
Table 5—Effective stiffness values

Component	Flexural rigidity	Shear rigidity	Axial rigidity
Beams—nonprestressed*	$0.3E_cE I_g$	$0.4E_cE A_w$	—
Beams—prestressed*	$E_cE I_g$	$0.4E_cE A_w$	—
Columns with compression caused by design gravity loads $\geq 0.5A_g f'_c$ [†]	$0.7E_cE I_g$	$0.4E_cE A_w$	$E_cE A_g$
Columns with compression caused by design gravity loads $\leq 0.1A_g f'_c$ or with tension [‡]	$0.3E_cE I_g$	$0.4E_cE A_w$	$E_cE A_g$ (compression) $E_s A_s$ (tension)
Beam-column joints	Refer to 4.2.2.1		$E_cE A_g$
Flat slabs—nonprestressed	Refer to 4.4.2	$0.4E_cE A_g$	—
Flat slabs—prestressed	Refer to 4.4.2	$0.4E_cE A_g$	—
Walls—cracked [‡]	$0.35E_cE I_g$	$0.4E_cE A_w$	$E_cE A_g$ (compression) $E_s A_s$ (tension)

*For T-beams, I_g can be taken as twice the value of I_g of the web alone. Otherwise, I_g shall be based on the effective width as defined in 3.1.3.

[†]For columns with axial compression falling between the limits provided, flexural rigidity shall be determined by linear interpolation. If interpolation is not performed, the more conservative effective stiffnesses shall be used. An imposed axial load N_{UG} is permitted to be used for stiffness evaluations.

[‡]Refer to 7.2.2.



When the effective stiffness values for beams and columns from Table 5 are used in combination with the modeling recommendations for beam-column joints, the overall stiffness is in close agreement with results from beam-column subassembly tests (Elwood et al. 2007).

The effect of reinforcement slip can be accounted for by including rotational springs at the ends of the beam or column elements (Saaticioglu et al. 1992). If this modeling option is selected, the effective flexural rigidity of the column element should reflect only the flexibility from flexural deformations. In this case, for axial loads less than $0.3A_g f'_c$, the effective flexural rigidity can be estimated as $0.5E_cE I_g$, with linear interpolation to the value given in Table 5 for axial loads greater than $0.5A_g f'_c$.

Because of low bond stress between concrete and plain reinforcement without deformations, components with plain longitudinal reinforcement and axial loads less than $0.5A_g f'_c$ can have lower effective flexural rigidity values than in Table 5.

Fig. 1—Generalized force-deformation relation for concrete elements or components.

STANDARD

3.1.2.2 Nonlinear procedures—Where design actions are determined using the nonlinear procedures of **ASCE 41-17** Chapter 7, component load-deformation response shall be represented by nonlinear load-deformation relations. Linear relations shall be permitted where nonlinear response does not occur in the component. The nonlinear load-deformation relation shall be based on experimental evidence or taken from quantities specified in **Chapters 4** through **12**. For the nonlinear static procedure (NSP), the generalized load-deformation relation shown in Fig. 1 or other curves defining behavior under monotonically increasing deformation shall be permitted. For the nonlinear dynamic procedure (NDP), load-deformation relations shall define behavior under monotonically increasing lateral deformation and under multiple reversed deformation cycles as specified in 3.2.1.

The generalized load-deformation relation shown in Fig. 1 shall be described by linear response from A (unloaded component) to an effective yield B, then a linear response at reduced stiffness from Point B to C, then sudden reduction in seismic force resistance to Point D, then response at reduced resistance to E, and final loss of resistance thereafter. The slope from Point A to B shall be determined according to 3.1.2.1. The slope from Point B to C, ignoring effects of gravity loads acting through lateral displacements, shall be taken between 0 and 10 percent of the initial slope, unless an alternate slope is justified by experiment or analysis. Point C shall have an ordinate equal to the strength of the component and an abscissa equal to the deformation at which significant strength degradation begins. Representation of the load-deformation relation by Points A, B, and C only (rather than all Points A through E) shall be permitted if the calculated response does not exceed Point C. Numerical values for the points identified in Fig. 1 shall be as specified in 3.2.2.2 for beams, columns, and joints; 3.3.2.2 for post-tensioned beams; 3.4.2.2 for slab-column connections; and **7.2.2** for shear walls, wall segments, and coupling beams. Other load-deformation relations shall be permitted if justified by experimental evidence or analysis.

COMMENTARY

C3.1.2.2 Nonlinear procedures—Typically, the response shown in Fig. 1 is associated with flexural response or tension response. In this case, the resistance at $Q/Q_y = 1.0$ is the yield value, and subsequent strain hardening is accommodated by hardening in the load-deformation relation as the member is deformed toward the expected strength. Where the response shown in Fig. 1 is associated with compression, the resistance at $Q/Q_y = 1.0$ typically is the value where concrete begins to spall, and strain hardening in well-confined sections can be associated with strain hardening of the longitudinal reinforcement and an increase in strength from the confinement of concrete. Where the response shown in Fig. 1 is associated with shear, the resistance at $Q/Q_y = 1.0$ typically is the value at which the design shear strength is reached and, typically, no strain hardening follows.

The deformations used for the load-deformation relation of Fig. 1 should be defined in one of two ways, as follows:

a) **Deformation, or Type I:** In this curve, deformations are expressed directly using terms such as strain, curvature, rotation, or elongation. The parameters a_{nl} and b_{nl} refer to deformation portions that occur after yield, or plastic deformation. The parameter c_{nl} is the reduced resistance after the sudden reduction from C to D. Parameters a_{nl} , b_{nl} , and c_{nl} are defined numerically in various tables in this standard. Alternatively, parameters a_{nl} , b_{nl} , and c_{nl} can be determined directly by analytical procedures justified by experimental evidence.

b) **Deformation ratio, or Type II:** In this curve, deformations are expressed in terms such as shear angle and tangential drift ratio. The parameters d_{nl} and e_{nl} refer to total deformations measured from the origin. Parameters c_{nl} , d_{nl} , and e_{nl} are defined numerically in various tables in this standard. Alternatively, parameters c_{nl} , d_{nl} , and e_{nl} can be determined directly by analytical procedures justified by experimental evidence.

Provisions for determining alternative modeling parameters and acceptance criteria based on experimental evidence are given in **ASCE 41-17**, 7.6.

Displacement demands determined from nonlinear dynamic analysis are sensitive to the rate of strength degradation included in the structural model. Unless there is experimental evidence of sudden strength loss for a particular component under consideration, the use of a model with a sudden strength loss from Point C to D in Fig. 1 can result in overestimation of the drift demands for a structural system and individual components. A more realistic model for many concrete components would have a linear degradation in resistance from Point C to E.

Strength loss that occurs within a single cycle can result in dynamic instability of the structure, whereas strength loss that occurs between cycles is unlikely to cause such instability. Figure 1 does not distinguish between these types of strength degradation and may not accurately predict the displacement demands if the two forms of strength degradation are not properly considered.

STANDARD

3.1.3 Flanged construction—In beams consisting of a web and flange that act integrally, the combined stiffness and strength for flexural and axial loading shall be calculated considering a width of effective flange on each side of the web equal to the smallest of:

1. The provided flange width
2. Eight times the flange thickness
3. Half the distance to the next web
4. One-fifth of the beam span length

Where the flange is in compression, the concrete and reinforcement within the effective width shall be considered effective in resisting flexure and axial load. Where the flange is in tension, longitudinal reinforcement within the effective width of the flange and developed beyond the critical section shall be considered fully effective for resisting flexural and axial loads. The portion of the flange extending beyond the width of the web shall be assumed ineffective in resisting shear.

In walls, effective flange width should be computed using Chapter 18 of [ACI 318-14](#).

3.2—Strength and deformability

3.2.1 General—Actions in a structure shall be classified as being either deformation-controlled or force-controlled. Deformation-controlled actions are defined by the designation of linear and nonlinear acceptance criteria in Tables 7 through 10 and 13 through 22. Where linear and nonlinear acceptance criteria are not specified in the tables, actions shall be taken as force-controlled unless component testing is performed in accordance with [ASCE 41-17](#), 7.6. Strengths for deformation-controlled and force-controlled actions shall be calculated in accordance with 3.2.2 and 3.2.3, respectively.

Components shall be classified as having low, moderate, or high ductility demands, according to 3.2.4.

Where strength and deformation capacities are derived from test data, the tests shall be representative of proportions, details, and stress levels for the component and comply with 7.6.1 of [ASCE 41-17](#).

The strength and deformation capacities of concrete members shall correspond to values resulting from a loading protocol involving three fully reversed cycles to the design deformation level, in addition to similar cycles to lesser deformation levels, unless a larger or smaller number of deformation cycles is determined considering earthquake duration and dynamic properties of the structure.

3.2.2 Deformation-controlled actions—Strengths used for deformation-controlled actions shall be taken as equal to expected strengths Q_{CE} obtained experimentally or calculated using accepted principles of mechanics. Unless specified in this standard, other procedures specified in [ACI 318](#) to calculate strengths shall be permitted, except that the strength reduction factor ϕ shall be taken equal to unity. Deformation capacities for acceptance of deformation-

COMMENTARY

C3.2—Strength and deformability

C3.2.1 General—In this standard, actions are classified as either deformation-controlled or force-controlled. Actions are considered deformation-controlled where the component behavior is well documented by test results. Where linear or nonlinear acceptance criteria are tabulated in this standard, the committee has judged the action to be deformation-controlled, and expected material properties should be used. Where such acceptance criteria are not specified, the action should be assumed force-controlled, thereby requiring the use of lower-bound material properties, or the licensed design professional can opt to perform testing to validate the classification of deformation-controlled. [ASCE 41-17](#), 7.6, provides guidance on procedures to be followed during testing, and [ASCE 41-17](#), 7.5.1.2 provides a methodology based on the test data to distinguish force-controlled from deformation-controlled actions. Further guidance on the testing of moment-frame components can be found in [ACI 374.1](#).

In some cases, including short-period buildings and those subjected to a long-duration design earthquake, a building can be expected to be subjected to additional cycles to the design deformation levels beyond the three cycles recommended in 3.2.1. The increased number of cycles can lead to reductions in resistance and deformation capacity. The effects on strength and deformation capacity of additional deformation cycles should be considered in design.

C3.2.2 Deformation-controlled actions—Expected yield strength of steel reinforcement, as specified in [2.2.1.2](#), includes material overstrength considerations.

STANDARD

COMMENTARY

controlled actions calculated by nonlinear procedures shall be as specified in **Chapters 4 through 12** of this standard. For components constructed of lightweight concrete, Q_{CE} shall be modified in accordance with **ACI 318** procedures for lightweight concrete.

3.2.3 Force-controlled actions—Strengths used for force-controlled actions shall be taken as lower-bound strengths Q_{CL} , obtained experimentally or calculated using established principles of mechanics. Lower-bound strength is defined as the mean less one standard deviation of resistance expected over the range of deformations and loading cycles to which the concrete component is likely to be subjected. Where calculations are used to define lower-bound strengths, lower-bound estimates of material properties shall be used. Unless other procedures are specified in this standard, procedures specified in ACI 318 to calculate strengths shall be permitted, except that the strength reduction factor ϕ shall be taken equal to unity. For components constructed of lightweight concrete, Q_{CL} shall be modified in accordance with ACI 318 procedures for lightweight concrete.

3.2.4 Component ductility demand classification—Table 6 provides classification of component ductility demands as low, moderate, or high based on the maximum value of the demand-capacity ratio (DCR) defined in **ASCE 41-17**, 7.3.1.1, for linear procedures or the calculated displacement ductility for nonlinear procedures.

Table 6—Component ductility demand classification

Maximum value of demand capacity ratio (DCR) or displacement ductility	Descriptor
Less than 2	Low ductility demand
2 to 4	Moderate ductility demand
Greater than 4	High ductility demand

3.3—Flexure and axial loads

Flexural strength of members with and without axial loads shall be calculated according to ACI 318 or by other demonstrated rational methods, such as sectional analysis using appropriate concrete and steel constitutive models. Deformation capacity of members with and without axial loads shall be calculated considering shear, flexure, and reinforcement slip deformations, or based on acceptance criteria given in this standard. Strengths and deformation capacities of components with monolithic flanges shall be calculated considering concrete and developed longitudinal reinforcement within the effective flange width, as defined in 3.1.3.

Strength and deformation capacities shall be determined based on the available development of longitudinal reinforcement. Where longitudinal reinforcement has embedment or development length that is insufficient for reinforcement strength development, flexural strength shall be

C3.3—Flexure and axial loads

Laboratory tests indicate that flexural deformability can be reduced as coexisting shear forces increase. As flexural ductility demands increase, shear capacity decreases, which can result in a shear failure before theoretical flexural deformation capacities are reached. Use caution where flexural deformation capacities are determined by calculation. **FEMA 306** and **ASCE 41-17**, 5.2 are resources for guidance on the interaction between shear and flexure.

The combined strength under uniaxial or biaxial bending with axial load is difficult to generalize in a closed-form solution, given the range of column section geometries encountered. For a particular class of rectangular column sections, closed-form solutions based on section capacities about the principal axes have been developed that provide excellent agreement when compared to a more generalized analysis (**Hsu 1988**; **Furlong et al. 2004**). A circular section provides a poor prediction of the strength for all but circular

STANDARD

calculated based on limiting stress capacity of the embedded bar as defined in 3.5.

Where flexural deformation capacities are calculated from basic principles of mechanics, reductions in deformation capacity caused by applied shear shall be considered. Where using analytical models for flexural deformability that do not directly account for the effect of shear on deformation capacity and if the design shear equals or exceeds $6\sqrt{f'_c} A_w$, lb/in.², the design flexural deformation capacity shall not exceed 80 percent of the value calculated using the analytical model.

For concrete columns or walls under combined axial load and biaxial bending, the combined strength shall be evaluated considering biaxial bending. When using linear procedures, the axial load P_{UF} or P_{UD} shall be calculated as a force-controlled action or deformation-controlled action per **ASCE 41-17**, 7.5.2. The design moments M_{UD} should be calculated about each of two orthogonal axes. Combined strength shall be based on principles of mechanics with applied bending moments calculated as $M_{UDx}/(m_x\kappa)$ and $M_{UDy}/(m_y\kappa)$ about the x - and y -axes, respectively. Acceptance shall be based on the applied bending moments lying within the expected strength envelope calculated at an axial load level of P_{UF} if the member is in compression, or $P_{UD}/[(\text{minimum of } m_x \text{ and } m_y)\kappa]$ if the member is in tension.

3.3.1 Usable strain limits—For deformation- and force-controlled actions in elements without confining transverse reinforcement, the maximum usable strain at the extreme concrete compression fiber used to calculate the moment and axial strength shall not exceed:

- a) 0.002 for members in nearly pure compression
- b) 0.005 for other members

Larger values of maximum usable strain in the extreme compression fiber shall be allowed where substantiated by experimental evidence.

For deformation- and force-controlled actions in elements with confined concrete, the maximum usable strain at the extreme concrete compression fiber used to calculate moment and axial strength shall be based on experimental evidence and consider limitations posed by transverse reinforcement fracture, longitudinal reinforcement buckling, and degradation of component resistance at large deformation levels. In the case of force-controlled actions in elements with confined concrete, it shall be permitted to adopt usable strain limits for unconfined concrete.

For deformation-controlled actions, the maximum compressive strains in the longitudinal reinforcement used to calculate the moment and axial strength shall not exceed 0.02, and maximum tensile strains in longitudinal reinforcement shall not exceed 0.05. Monotonic coupon test results shall not be used to determine reinforcement strain limits. If experimental evidence is used to determine strain limits for reinforcement, the effects of low-cycle fatigue and transverse reinforcement spacing and size shall be included in testing procedures.

COMMENTARY

columns. For general sections, the strength envelope should be developed based on principles of mechanics.

When flexural strength of an axially loaded member needs to be calculated in the linear procedure, compressive load level should be considered as a force-controlled action due to its nonductile nature whereas tensile load level should be considered as a deformation-controlled action because the tensile strength and stiffness of the member are based on steel reinforcement contribution only. The m -factor for the flexural behavior can be conservatively used to estimate the deformation-controlled action due to the tension.

C3.3.1 Usable strain limits—Early research on the stress-strain behavior of unconfined concrete (**Hognestad 1952**) has shown that the stress-strain behavior of concrete is different in members subjected to flexure than in members subjected to nearly pure compression. Concrete subjected to concentric compression exhibits crushing shortly after the maximum stress is reached at strains of approximately 0.0015 to 0.0020 (Hognestad 1952), whereas crushing in the extreme compression fiber of members subjected to flexure and axial load is observed at higher strains, ranging between 0.003 to 0.005 (Hognestad 1952). The maximum usable strain limits established in this section are intended to caution engineers when using stress-strain relationships for concrete to calculate moment and axial strengths. In members subjected to nearly pure compression, redistribution of stresses within the compression zone after the strain in the concrete exceeds the strain corresponding to peak stress (0.0015 to 0.0020 for unconfined concrete) (Hognestad 1952) is not possible because most of the concrete in the cross section will be on the descending branch of the stress-strain curve for concrete.

Usable strain limits specified in this section do not preclude engineers from using the provisions in 22.2 of **ACI 318-14**. Section 22.2.2.1 of **ACI 318-14** stipulates that to calculate the moment and axial strength of reinforced concrete members, the maximum usable strain in the extreme compression fiber of reinforced concrete shall be assumed to be 0.003. This usable strain is within the limit of 0.005 specified in 3.3.1 herein. In the case of members subjected to nearly pure compression, provisions in 22.4.2 of **ACI 318-14** establish that the design axial strength of columns shall not exceed 80 percent of the nominal axial

STANDARD

COMMENTARY

3.4—Shear and torsion

Strengths in shear and torsion shall be calculated according to ACI 318, except as modified in this standard.

Within yielding regions of components with moderate or high ductility demands, shear and torsional strength shall be calculated according to procedures for ductile components, such as the provisions in Chapter 18 of ACI 318. Within yielding regions of components with low ductility demands per Table 6 and outside yielding regions for all ductility demands, procedures for effective elastic response, such as the provisions in Chapter 22 of ACI 318-14, shall be permitted to calculate the design shear strength.

Unless otherwise noted, where the longitudinal spacing of transverse reinforcement exceeds half the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed to have reduced effectiveness in resisting shear or torsion by a factor of $2(1 - s/d)$. Where the longitudinal spacing of transverse reinforcement exceeds the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed ineffective in resisting shear or torsion. For beams and columns, lap-spliced transverse reinforcement shall be assumed not more

strength. According to the commentary of 22.4.2.1 of ACI 318-14, the reduced nominal axial strength corresponds to a minimum eccentricity of 5 percent of the column depth. The usable strain limit of 0.002 specified in 3.3.1 herein is intended to prevent overestimating the flexural strength of columns with very small eccentricities, so the provisions in 22.4.2.1 of ACI 318-14 can be used in place of calculating the axial and moment strength based on stress-strain models for concrete.

While provisions in 21.2.2 of ACI 318-14 establish that, for tension-controlled members, the strain in the reinforcement at failure shall be at least 0.005, there is no upper limit in the code for the usable strain in the reinforcement of beams and columns. Although an upper limit in the strain at failure of beams and columns is implied in the provisions for minimum reinforcement in 9.6 and 10.6 of ACI 318-14, those limits are not intended for members that will be subjected to deformation cycles in the nonlinear range of response. The reinforcement tensile strain limit in 5.3.1 herein is based on consideration of the effects of material properties and low-cycle fatigue. Low-cycle fatigue is influenced by spacing and size of transverse reinforcement and strain history. Using extrapolated monotonic test results to develop tensile strains greater than those specified above is not recommended. Caltrans (2006) recommends an ultimate tensile strain of 0.09 for No. 10 bars and smaller, and 0.06 for No. 11 bars and larger, for ASTM A706/A706M 60 kip/in.² reinforcing bars. A lower bound is selected herein considering the variability in materials and details typically found in existing structures.

Refer to Brown and Kunnath (2004) for incorporating the effects of low-cycle fatigue and transverse reinforcement for determining strain limits based on testing.

C3.4—Shear and torsion

The reduction in the effectiveness of transverse reinforcement in this section accounts for the limited number of ties expected to cross an inclined crack when ties are provided at large spacing. Furthermore, reduction in the effectiveness of the transverse reinforcement is needed because the widely spaced ties may not be fully developed both above and below the crack. For tie spacing equal to the effective depth of the member, it is possible to develop an inclined crack that does not cross any ties and, hence, the contribution of the transverse reinforcement should be ignored.

STANDARD

than 50 percent effective in regions of moderate ductility demand and ineffective in regions of high ductility demand, and applies in addition to the effectiveness factor due to spacing.

Shear friction strength shall be calculated according to **ACI 318**, considering the expected axial load from gravity and earthquake effects. Where retrofit involves the addition of concrete requiring overhead work with dry pack, the shear friction coefficient μ shall be taken equal to 70 percent of the value specified by ACI 318.

3.5—Development and splices of reinforcement

Development of straight bars, hooked bars, and lap-spliced bars shall be calculated according to the provisions of ACI 318, with the following modifications:

a) Deformed straight, hooked, and lap-spliced bars satisfying the development requirements of Chapter 25 of ACI 318-14 using expected material properties shall be deemed capable of developing their yield strength, except as adjusted in the following:

i. The development of lapped straight bars in tension without consideration of lap splice classifications is permitted to be used as the required lap splice length.

ii. For columns, where deformed straight and lap-spliced bars pass through regions where inelastic deformations and damage are expected, the bar length within those regions shall be considered effective for anchorage only until inelastic deformations occur. In such cases, the development length obtained using ACI 318 procedures shall be compared with a degraded available development length ℓ_{b-deg} , as defined in (b) in the following.

b) Where existing deformed straight bars, hooked bars, and lap-spliced bars do not meet the development requirements of (a), the capacity of existing reinforcement shall be calculated as follows

$$f_s = 1.25 \left(\frac{\ell_b}{\ell_d} \right)^{2/3} f_{yL} \leq f_{yL/E} \quad (1a)$$

If the maximum applied bar stress is larger than f_s given in Eq. (1a), members shall be deemed controlled by inadequate development or splicing.

For columns where deformed straight and lap-spliced longitudinal bars pass through regions where inelastic deformations and damage are expected, the bar length within those regions shall be considered effective for anchorage only until inelastic deformations occur. In such cases, if $f_s = f_{yL/E}$ from Eq. (1a), the degraded reinforcement capacity f_{s-deg} accounting for the loss of anchorage in the damaged region shall be evaluated using a degraded available development length (ℓ_{b-deg}). ℓ_{b-deg} shall be evaluated by subtracting from ℓ_b a distance of $2/3d$ from the point of maximum flexural demand in any direction damage is anticipated within the column.

COMMENTARY

C3.5—Development and splices of reinforcement

Development requirements in accordance with Chapter 25 of **ACI 318-14** are applicable to development of bars in all components. Chapter 18 of ACI 318-14 provides development requirements that are intended only for use in yielding components of reinforced concrete moment frames that comply with the cover and confinement provisions of Chapter 18 of ACI 318-14. Chapter 25 of ACI 318-14 permits reductions in lengths if minimum cover and confinement are present in an existing component. For additional information on development and lap splices, refer to **ACI 408R**, and for hooked anchorage, refer to **Sperry et al. (2005)**.

Equation (1a), which is a modified version of the model presented by **Cho and Pincheira (2006)**, reflects the intent of ACI 318 development and splice equations to develop 1.25 times the nominal bar strength, referred to in this standard as the expected yield strength. The nonlinear relation between developed stress and development length reflects the effect of increasing slip and, hence, reduced unit bond strength, for longer development lengths. Refer to **Elwood et al. (2007)** for more details.

Bond strength can be significantly curtailed in damaged regions within plastic hinges (**Sokoli and Ghannoum 2016; Ichinose 1995**). The length where bond capacity is curtailed during inelastic deformations is recommended to be two-thirds of the section effective depth d (Sokoli and Ghannoum 2016). If f_s , evaluated using Eq. (1a), equals $f_{yL/E}$, then bond failure is not expected prior to inelastic hinging and the bar under consideration can be expected to resist the full yield stress $f_{yL/E}$. However, f_s should be re-evaluated using a degraded effective anchorage length ℓ_{b-deg} using Eq. (1b), which is reduced by the bar length within the region expected to be damaged. If f_{s-deg} remains equal to $f_{yL/E}$, even after the anchorage length is reduced, then no anchorage failure is expected even during inelastic deformations. If, however, f_{s-deg} becomes smaller than $f_{yL/E}$ when the available anchorage length is reduced, then anchorage failure is expected, but only after inelastic deformations occur. In such cases, the limiting stress in longitudinal bars will be $f_{yL/E}$ but the modeling parameters in Tables 8 and 9 for columns with inadequate development or splicing should be used.

For buildings constructed before 1950, the bond strength developed between steel reinforcement and concrete can be less than present-day strength. Present-day strength of development and splices of reinforcement should account for mechanical

STANDARD

COMMENTARY

Table 7—Modeling parameters and numerical acceptance criteria for nonlinear procedures: reinforced concrete beams

Conditions			Modeling parameters*			Acceptance criteria*		
			Plastic rotations angle, rad		Residual strength ratio	Plastic rotations angle, rad		
						Performance level		
			a	b	c	IO	LS	CP
Condition i. beams controlled by flexure†								
$\frac{\rho - \rho'}{\rho_{bal}}$	Transverse reinforcement‡	$\frac{V}{b_w d \sqrt{f'_{ce}}}$ §						
≤0.0	C	≤3	0.025	0.05	0.2	0.010	0.025	0.05
≤0.0	C	≥6	0.02	0.04	0.2	0.005	0.02	0.04
≥0.5	C	≤3	0.02	0.03	0.2	0.005	0.02	0.03
≥0.5	C	≥6	0.015	0.02	0.2	0.005	0.015	0.02
≤0.0	NC	≤3	0.02	0.03	0.2	0.005	0.02	0.03
≤0.0	NC	≥6	0.01	0.015	0.2	0.0015	0.01	0.015
≥0.5	NC	≤3	0.01	0.015	0.2	0.005	0.01	0.015
≥0.5	NC	≥6	0.005	0.01	0.2	0.0015	0.005	0.01
Condition ii. beams controlled by shear†								
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.005	0.01
Condition iii. beams controlled by inadequate development or splicing along the span†								
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.005	0.01
Condition iv. beams controlled by inadequate embedment into beam-column joint†								
			0.015	0.03	0.2	0.01	0.02	0.03

*Values between those listed in the table shall be determined by linear interpolation.

†Where more than one of Conditions i, ii, iii, and iv occur for a given component, use the minimum appropriate numerical value from the table.

‡C and NC are abbreviations for conforming and nonconforming transverse reinforcement, respectively. Transverse reinforcement is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ d/3, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_h) is at least three-fourths of the design shear. Otherwise, the transverse reinforcement is considered nonconforming.§ V is the shear force from NSP or NDP.Note: f'_c in lb/in.².

$$f_s = 1.25 \left(\frac{\ell_{b-deg}}{\ell_d} \right)^{2/3} f_{yL} \leq f_{yL/E} \quad (1b)$$

In cases where $f_s = f_{yL/E}$ from Eq. (1a) but the maximum applied longitudinal bar stress is larger than f_{s-deg} given in Eq. (1b), columns shall be deemed controlled by inadequate development or splicing and the capacity of the existing reinforcement taken as $f_{yL/E}$.

c) For inadequate development or splicing of straight bars in beams and columns: for nonlinear procedures, it shall be permitted to assume that the reinforcement retains the calculated maximum stress evaluated using Eq. (1a) up to the deformation levels defined by $a_{n\ell}$ in Tables 7 through 9; for linear procedures, the calculated maximum stress evaluated using Eq. (1a) shall be used for strength calculations. For members other than beams and columns controlled by inad-

bond from deformations present in deformed bars as well as chemical bond. The length required to develop plain bars is much greater than for deformed bars and more sensitive to cracking in concrete. Testing and assessment procedures for tensile lap splices and development length for plain steel reinforcement are found in **Concrete Reinforcing Steel Institute (1981)**.

STANDARD

COMMENTARY

Table 8—Modeling parameters and numerical acceptance criteria for nonlinear procedures: reinforced concrete columns other than circular with spiral reinforcement or seismic hoops as defined in ACI 318

Modeling parameters	Acceptance criteria		
Plastic rotation angles a_{nl} and b_{nl} , rad Residual strength ratio c_{nl}	Plastic rotation angle, rad		
	Performance level		
	IO	LS	CP
Columns not controlled by inadequate development or splicing along the clear height*			
$a_{nl} = \left(0.042 - 0.043 \frac{N_{UD}}{A_g f'_{cE}} + 0.63 \rho_t - 0.023 \frac{V_{yE}}{V_{ColOE}} \right) \geq 0.0$	$0.15 a_{nl} \leq 0.005$	$0.5 b_{nl}^{\dagger}$	$0.7 b_{nl}^{\dagger}$
For $\frac{N_{UD}}{A_g f'_{cE}} \leq 0.5$ $b_{nl} = \frac{0.5}{5 + \frac{N_{UD}}{0.8 A_g f'_{cE}} \frac{1}{\rho_t f_{yE}}} - 0.01 \geq a_{nl}$ †			
$c_{nl} = 0.24 - 0.4 \frac{N_{UD}}{A_g f'_{cE}} \geq 0.0$			
Columns controlled by inadequate development or splicing along the clear height§			
$a_{nl} = \left(\frac{1}{8} \frac{\rho_t f_{yE}}{\rho_t f_{yE}} \right) \geq 0.0 \parallel \leq 0.025$	0.0	$0.5 b_{nl}$	$0.7 b_{nl}$
$b_{nl} = \left(0.012 - 0.085 \frac{N_{UD}}{A_g f'_{cE}} + 12 \rho_t \right) \geq 0.0 \# \leq a_{nl} \leq 0.06$			
$c_{nl} = 0.15 + 36 \rho_t \leq 0.4$			

^{*} ρ_t shall not be taken greater than 0.0175 in any case nor greater than 0.0075 when ties are not adequately anchored in the core. Equations in the table are not valid for columns with ρ_t smaller than 0.0005. V_{yE}/V_{ColOE} shall not be taken less than 0.2. N_{UD} shall be the maximum compressive axial load accounting for the effects of lateral forces as described in Eq. (7-34) of ASCE 41-17. Alternatively, it shall be permitted to evaluate N_{UD} based on a limit-state analysis.

[†] b_{nl} shall be reduced linearly for $N_{UD}/(A_g f'_{cE}) > 0.5$ from its value at $N_{UD}/(A_g f'_{cE}) = 0.5$ to zero at $N_{UD}/(A_g f'_{cE}) = 0.7$ but shall not be smaller than a_{nl} .

[‡] $N_{UD}/(A_g f'_{cE})$ shall not be taken smaller than 0.1.

[§]Columns are considered to be controlled by inadequate development or splices where the calculated steel stress at the splice exceeds the steel stress specified by Eq. (1a) or (1b). Modeling parameter for columns controlled by inadequate development or splicing shall never exceed those of columns not controlled by inadequate development or splicing.

^{||} a_{nl} for columns controlled by inadequate development or splicing shall be taken as zero if the splice region is not crossed by at least two tie groups over its length.

[#] ρ_t shall not be taken greater than 0.0075.

equate development or splicing and hooked anchorage, the developed stress shall be assumed to degrade from $1.0f_s$ at a ductility demand or DCR equal to 1.0, to $0.2f_s$ at a ductility demand or DCR equal to 2.0.

d) The strength of deformed straight, discontinuous bars embedded in concrete sections or beam-column joints, with clear cover over the embedded bar not less than $3d_b$, shall be calculated according to Eq. (2)

$$f_s = \frac{2500}{d_b} \ell_e \leq f_{yL/E} \quad (\text{psi})$$

$$f_s = \frac{17}{d_b} \ell_e \leq f_{yL/E} \quad (\text{MPa}) \quad (2)$$

where f_s is less than $f_{yL/E}$ and the calculated stress in the bar caused by design loads equals or exceeds f_s , the maximum developed stress shall be assumed to degrade from $1.0f_s$ at a

STANDARD

COMMENTARY

Table 9—Modeling parameters and numerical acceptance criteria for nonlinear procedures: reinforced concrete circular columns with spiral reinforcement or seismic hoops as defined in ACI 318

Modeling parameters	Acceptance criteria		
Plastic rotation angles a_{nt} and b_{nt} , rad Residual strength ratio c_{nt}	Plastic rotation angle, rad		
	Performance level		
	IO	LS	CP
Columns not controlled by inadequate development or splicing along the clear height [*]			
$a_{nt} = \left(0.06 - 0.06 \frac{N_{UD}}{A_g f'_{cE}} + 1.3 \rho_t - 0.037 \frac{V_{yE}}{V_{CoOE}} \right) \geq 0.0$	0.15 $a_{nt} \leq 0.005$	0.5 $b_{nt}^{\ddagger}$	0.7 $b_{nt}^{\ddagger}$
For $\frac{N_{UD}}{A_g f'_{cE}} \leq 0.5$ $b_{nt} = \frac{0.65}{5 + \frac{N_{UD}}{0.8 A_g f'_{cE}} \frac{1}{\rho_t f_{yE}}} - 0.01 \geq a_{nt}$ [†]			
$c_{nt} = 0.24 - 0.4 \frac{N_{UD}}{A_g f'_{cE}} \geq 0.0$			
Columns controlled by inadequate development or splicing along the clear height [§]			
$a_{nt} = \left(\frac{1}{8} \frac{\rho_t f_{yE}}{\rho_t f_{yE}} \right) \geq 0.0$ ≤ 0.025	0.0	0.5 b_{nt} [®]	0.7 b_{nt}
$b_{nt} = \left(0.012 - 0.085 \frac{N_{UD}}{A_g f'_{cE}} + 12 \rho_t \right) \geq 0.0$ [#] ≤ 0.06			
$c_{nt} = 0.15 + 36 \rho_t \leq 0.4$			

^{*} ρ_t shall not be taken greater than 0.0175 in any case nor greater than 0.0075 when ties are not adequately anchored in the core. Equations in the table are not valid for columns with ρ_t smaller than 0.0005. V_{yE}/V_{CoOE} shall not be taken less than 0.2.

N_{UD} shall be the maximum compressive axial load accounting for the effects of lateral forces as described in Eq. (7-34) of ASCE 41-17. Alternatively, it shall be permitted to evaluate N_{UD} based on a limit-state analysis.

[†] b_{nt} shall be reduced linearly for $N_{UD}/(A_g f'_{cE}) > 0.5$ from its value at $N_{UD}/(A_g f'_{cE}) = 0.5$ to zero at $N_{UD}/(A_g f'_{cE}) = 0.7$ but shall not be smaller than a_{nt} .

[‡] $N_{UD}/(A_g f'_{cE})$ shall not be taken smaller than 0.1.

[§]Columns are considered to be controlled by inadequate development or splices where the calculated steel stress at the splice exceeds the steel stress specified by Eq. (1a) or (1b). Modeling parameter for columns controlled by inadequate development or splicing shall never exceed those of columns not controlled by inadequate development or splicing.

^{||} a_{nt} for columns controlled by inadequate development or splicing shall be taken as zero if the splice region is not crossed by at least two tie groups over its length.

[#] ρ_t shall not be taken greater than 0.0075.

ductility demand or DCR equal to 1.0, to 0.2 f_s at a ductility demand or DCR equal to 2.0. In beams with bottom bar embedment length into beam-column joints less than the requirements of ACI 318, flexural strength shall be calculated considering the stress limitation of Eq. (2).

e) For plain straight, hooked, and lap-spliced bars, development and splice lengths shall be taken as twice the values determined in accordance with ACI 318, unless other lengths are justified by approved tests.

f) Doweled bars added in seismic retrofit shall be assumed to develop yield stress where all the following conditions are satisfied:

- Drilled holes for dowel bars are cleaned
- Embedment length ℓ_e is not less than $10d_b$
- Minimum dowel bar spacing is not less than $4\ell_e$ and minimum edge distance is not less than $2\ell_e$

STANDARD

Design values for dowel bars not satisfying these conditions shall be verified by test data. Field samples shall be obtained to ensure that design strengths are developed in accordance with Chapter 3.

g) Square reinforcing bars in a building should be classified as either twisted or straight. The developed strength of twisted square bars shall be as specified for deformed bars in this section, using an effective diameter calculated based on the area of the square bar. Straight square bars shall be considered plain bars, and the developed strength shall be as specified for plain bars in this section.

3.6—Connections to existing concrete

Connections used to connect two or more components shall be classified according to their anchoring systems as cast-in-place or as post-installed and shall be evaluated and designed according to Chapter 17 of **ACI 318-14** as modified in this section. The properties of the existing anchors and connection systems obtained in accordance with **2.2** herein shall be considered in the evaluation. These provisions do not apply to connections in plastic hinge zones.

COMMENTARY

C3.6—Connections to existing concrete

Chapter 17 of **ACI 318-14** accounts for the influence of cracking on the load capacity of connectors; however, cracking and spalling expected in plastic hinge zones is likely to be more severe than the level of damage for which Chapter 17 is applicable. **ACI 355.2** and **ACI 355.4** describe simulated seismic tests that can be used for qualification of post-installed anchors. Such tests do not simulate the conditions expected in plastic hinge zones.

The 2006 version of **ASCE 41** required the load capacity of anchors placed in areas where cracking is expected to be reduced by a factor of 0.5. This provision was included in **FEMA 273** for both cast-in-place and post-installed anchors, before the introduction of **ACI 318-02** Appendix D. Because cracking is now accounted for in **ACI 318**, the 0.5 factor is not required in 3.6 of this standard.

Capacities of existing anchors should be evaluated based on the obtained properties in accordance with **2.2**, and Chapter 17 of **ACI 318-14**. If the anchors are not tested to failure but to a load based on the force-controlled action determined by the engineer for the seismic hazard under consideration, the procedure in Chapter 17 of **ACI 318-14** can be used to calculate available strength based on the test results and the geometry of anchors measured or assumed by the engineer.

To evaluate the capacity of existing cast-in-place and post-installed anchors using **ACI 318-14** Chapter 17, it is necessary to know the geometry of the anchor (that is, embedment, edge distance, spacing, and anchor diameter) and material properties. Edge distance, spacing, and anchor diameter can be established from construction documents or by visual inspection. Unless known from construction documents, embedment and material properties of the anchor are more difficult to determine. Where failure of the anchor is not critical to meeting the target performance level, embedment of post-installed anchors can be assumed equal to the minimum embedment required by manufacturer's specifications for the anchor type in question. For cast-in-place anchors, embedment can be taken as less than or equal to the minimum embedment from the original design code for an embedded bolt of the same diameter. It is recommended that where the consequence of failure of an anchor is critical to satisfying the target performance level, anchor embedment not known from construction documents be determined by nondestructive testing (for example, ultrasonic testing).

STANDARD

COMMENTARY

3.6.1 Cast-in-place anchors and connection systems—All component actions on cast-in-place anchors and connection systems shall be considered force-controlled. Lower-bound strength of the anchors and connections shall be nominal strength as specified in Chapter 17 of ACI 318-14 for the connections of structural components. The amplification factor to account for the seismic overstrength, Ω_0 , shall be taken equal to unity for the connections of structural components.

A strength reduction factor ϕ and amplification factor Ω_0 shall be used for the connections of nonstructural components.

3.6.2 Post-installed anchors—All component actions on post-installed anchor connection systems shall be considered force-controlled. The lower-bound capacity of post-installed anchors shall be nominal strength, as specified in Chapter 17 of ACI 318-14, or mean less one standard deviation of ultimate values published in approved test reports for the connections of structural components. The amplification factor to account for the seismic overstrength, Ω_0 , shall be taken equal to unity for the connections of structural components.

A strength reduction factor ϕ and amplification factor Ω_0 shall be used for the connections of nonstructural components.

Lower-bound properties for steel connector materials and concrete strength based on default values, construction documents, or test values can be assumed for anchor strength calculations. It is noted that direct testing of anchors can provide greater certainty and can provide higher capacities. Judgment should be exercised in the use of default lower-bound material properties because doing so may not yield a conservative estimate of anchor capacity in cases where the steel strength is determined to govern the anchor capacity, and additional requirements of ACI 318-14 Chapter 17 for ductile behavior are waived as a result.

Not all manufacturers of post-installed anchors publish information on the mean and the standard deviation of the ultimate anchor capacity. Older testing for existing post-installed anchors is often reported at allowable stress design levels and may not comply with the requirements of Chapter 17 of ACI 318-14 for simulated seismic tests. Care and judgment should be used in determining pullout strength for anchors, particularly those that are critical to satisfying the target performance level. Where necessary, in-place strengths of anchors can be obtained or verified by static testing of representative anchors. ACI 355.2 and ACI 355.4 can be used for guidance on testing.

Proper installation of post-installed anchors is critical to their performance and should be verified in all cases.

C3.6.1 Cast-in-place anchors and connection systems—The strength reduction factor ϕ in ACI 318 is taken equal to unity for the lower-bound connection strength of structural components, but the requirements in 17.2.3 of ACI 318-14 need to be satisfied, including the reduction of the strength due to cracked concrete and cyclic loading. The component actions on the anchors and connection systems for structural components are considered as force-controlled actions according to 7.5.2 and 7.5.3 of ASCE 41-17, so further amplification of the seismic demand is not necessary.

However, the seismic demand on nonstructural components in Chapter 13 of ASCE 41-17 is based on that in ASCE 7. A strength reduction factor ϕ and amplification factor Ω_0 should be consistent with the demand.

C3.6.2 Post-installed anchors—The strength reduction factor ϕ in ACI 318 is taken equal to unity for the lower-bound connection strength of structural components, but the requirements in 17.2.3 of ACI 318-14 need to be satisfied, including the reduction of the strength due to cracked concrete and cyclic loading. The component actions on post-installed anchors for structural components are considered force-controlled actions according to 7.5.2 and 7.5.3 of ASCE 41-17, so further amplification of the seismic demand is not necessary.

However, the seismic demand on nonstructural components in Chapter 13 of ASCE 41-17 is based on that in ASCE 7. Strength reduction factor ϕ and amplification factor Ω_0 should be consistent with the demand.

STANDARD

COMMENTARY

3.7—Retrofit measures

Seismic retrofit measures for concrete buildings shall meet the requirements of this section and other provisions of this standard.

Retrofit measures shall include replacement or retrofit of the component or modification of the structure so that the component is no longer deficient for the selected performance objective. If component replacement is selected, the new component shall be designed in accordance with this standard and detailed and constructed in compliance with the applicable building code.

Retrofit measures shall be evaluated to ensure that the completed retrofit achieves the selected performance objective. The effects of retrofit on stiffness, strength, and deformability shall be taken into account in an analytical model of the rehabilitated structure. The compatibility of new and existing components shall be checked at displacements consistent with the selected performance level.

Connections required between existing and new components shall satisfy the requirements of 3.6 and other requirements of this standard.



STANDARD

COMMENTARY

CHAPTER 4—CONCRETE MOMENT FRAMES

4.1—Types of concrete moment frames

Concrete moment frames are defined as elements composed primarily of horizontal frame components, such as beams, slabs, or both; vertical frame components, such as columns; and joints connecting horizontal and vertical frame components. To resist seismic forces, these elements act alone or in conjunction with shear walls, braced frames, or other elements.

Frames that are cast monolithically, including monolithic concrete frames created by the addition of new material, are addressed in this chapter. Frames addressed include reinforced concrete beam-column moment frames, post-tensioned concrete beam-column moment frames, and slab-column moment frames.

The frame classifications in 4.1.1 through 4.1.3 include existing construction, new construction, existing construction that has been retrofitted, frames intended as part of the seismic-force-resisting system, and frames not intended as part of the seismic-force-resisting system in the original design.

4.1.1 Reinforced concrete beam-column moment frames—Reinforced concrete beam-column moment frames, addressed in 4.2, are defined by the following conditions:

- a) Frame components are beams with or without slabs, columns, and their connections.
- b) Frames are of monolithic construction that provide for moment and shear transfer between beams and columns.
- c) Primary reinforcement in components contributing to seismic-force resistance is nonprestressed.

4.1.2 Post-tensioned concrete beam-column moment frames—Post-tensioned concrete beam-column moment frames, addressed in 4.3, are defined by the following conditions:

- a) Frame components are beams (with or without slabs), columns, and their connections.
- b) Frames are of monolithic construction that provide for moment and shear transfer between beams and columns.
- c) Primary reinforcement in beams contributing to seismic force resistance includes post-tensioned reinforcement with or without nonprestressed reinforcement.

4.1.3 Slab-column moment frames—Slab-column moment frames, addressed in 2.4, are defined by the following conditions:

- a) Frame components are slabs with or without beams in the transverse direction, columns, and their connections.
- b) Frames are of monolithic construction that provide for moment and shear transfer between slabs and columns.
- c) Primary reinforcement in slabs contributing to seismic force resistance includes nonprestressed reinforcement, prestressed reinforcement, or both.

STANDARD

4.2—Reinforced concrete beam-column moment frames

4.2.1 General—The analytical model for a beam-column frame element shall represent strength, stiffness, and deformation capacity of beams, columns, beam-column joints, and other components of the frame, including connections with other elements. Potential failure in flexure, shear, and reinforcement development at any section along the component length shall be considered. Interaction with other elements, including nonstructural components, shall be included.

Analytical models representing a beam-column frame using line elements with properties concentrated at component centerlines shall be permitted. Where beam and column centerlines do not intersect, the eccentricity effects between frame centerlines shall be considered. Where the centerline of the narrower component falls within the middle one-third of the adjacent frame component measured transverse to the framing direction, this eccentricity need not be considered. Where larger eccentricities occur, the effect shall be represented either by reductions in effective stiffness, strength, and deformation capacity or by direct modeling of the eccentricity.

The beam-column joint in monolithic construction is the zone having horizontal dimensions equal to the column cross-sectional dimensions and vertical dimension equal to the beam depth. A wider joint is acceptable where the beam is wider than the column. The beam-column joint shall be modeled according to 4.2.2 or as justified by experimental evidence. The model of the connection between columns and foundation shall be selected based on details of the column-foundation connection and rigidity of the foundation-soil system.

Action of the slab as a diaphragm interconnecting vertical components shall be considered. Action of the slab as a composite beam flange shall be considered in developing stiffness, strength, and deformation capacities of the beam component model per 3.1.3.

Inelastic action shall be restricted to those components and actions listed in Tables 7 through 9, except where it is demonstrated by experimental evidence and analysis that other inelastic action is acceptable for the selected performance level. Acceptance criteria are specified in 4.2.4.

4.2.2 Stiffness of reinforced concrete beam-column moment frames

4.2.2.1 Linear static and dynamic procedures—Beams shall be modeled considering flexural and shear stiffnesses, including the effect of the slab acting as a flange in monolithic construction according to the provisions in 3.1.3. Columns shall be modeled considering flexural, shear, and axial stiffnesses. Refer to 3.1.2 to compute the effective stiffnesses. Where joint stiffness is not modeled explicitly, it shall be permitted to be modeled implicitly by adjusting a centerline model (Fig. 2):

a) For $\sum M_{ColE} / \sum M_{BE} > 1.2$, column offsets are rigid and beam offsets are not.

COMMENTARY

C4.2—Reinforced concrete beam-column moment frames

C4.2.1 General—Nonstructural components should be included in the analytical model if such elements contribute significantly to building stiffness, modify dynamic properties, or have significant impact on the behavior of adjacent structural elements. Section 7.2.3.3 of [ASCE 41-17](#) suggests that nonstructural components should be included if their lateral stiffness exceeds 10 percent of the total initial lateral stiffness of a story. Partial infill walls and staircases are examples of nonstructural elements that can alter the behavior of adjacent concrete structural elements.

C4.2.2 Stiffness of reinforced concrete beam-column moment frames

C4.2.2.1 Linear static and dynamic procedures—Various approaches to explicitly model beam-column joints are available in [El-Metwally and Chen \(1988\)](#), [Ghobarah and Biddah \(1999\)](#), [Shin and LaFave \(2004\)](#), [Mittra and Lowes \(2007\)](#), and [Lin and Restrepo \(2002\)](#). For simplicity of implementation in commercial structural analysis software and agreement with calibration studies performed in the development of this standard, this section defines an implicit beam-column joint modeling technique using centerline models with semi-rigid joint offsets. Figure 2.1.1 is an example of an explicit joint model and illustrates the implicit