ACI 369R-11

Guide for Seismic Rehabilitation of Existing Concrete Frame Buildings and Commentary

Reported by ACI Committee 369

American Concrete Institute®

First Printing
February 2011

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ISBN 978-0-87031-419-3

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This guide, which was developed based on the format and content of ASCE/SEI 41-06, Chapter 6.0, "Concrete," describes methods for estimating the seismic performance of both existing and new concrete components in an existing building. The guide is intended to be used with the analysis procedures and Rehabilitation Objectives established in ASCE/SEI 41-06 for the Systematic Rehabilitation Method. The guide provides recommendations for modeling parameters and acceptance criteria for linear and nonlinear analysis of beams, columns, joints, and slab-column connections of concrete buildings and the procedures for obtaining material properties necessary for seismic rehabilitation design.

Keywords: acceptance criteria; ASCE/SEI 41; beams; columns; frames; joints; modeling parameters; retrofit; seismic rehabilitation; slab-column connections.

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ACI 369R-11 was adopted and published February 2011.

CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

Earthquake reconnaissance has clearly demonstrated that existing concrete frame buildings designed before the introduction of modern seismic codes are more vulnerable to severe damage or collapse when subjected to strong ground motion. 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.

1.2—Scope

This guide describes methods for estimating the seismic performance of concrete components in an existing building. The guide is intended to be used with the analysis procedures and Rehabilitation Objectives (ROs) established in ASCE/SEI 41-06 for the Systematic Rehabilitation Method. The methods described apply to existing concrete components of a building system, rehabilitated concrete components of a building system, and new concrete components added to an existing building system. Provisions of this guide do not apply to concrete-encased steel composite components.

Chapter 2 recommends data collection procedures for obtaining material properties and performing condition assessments. Chapter 3 provides general analysis and design requirements for concrete components. Chapter 4 provides modeling procedures, component strengths, acceptance criteria, and rehabilitation measures for cast-in-place concrete moment frames.

C1.2—Scope

This guide has been developed to provide a document that can be easily updated to reflect results from ongoing research on the seismic performance of existing concrete buildings. ACI 369R closely follows the format of Chapter 6, "Concrete" of ASCE/SEI 41-06, to make it readily accessible to engineers and to facilitate updates. Although the content in this version is similar to Chapter 6 of ASCE/SEI 41-06, this will change with timely updates specific to ongoing research. The intent is to provide a continuously updated resource document for future modifications to Chapter 6 of ASCE/SEI 41-06, similar to how the National Earthquake Hazards Reduction Program (NEHRP) Recommended 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.

The goal of developing a guide rather than a standard is to focus the updating effort on improving technical content over development of codified language. New research results reviewed by the committee can now be implemented more quickly, accelerating communication between researchers and engineers while providing design professionals with the latest recommendations for the seismic assessment and rehabilitation of concrete buildings. For this version of the guide, most sections are similar to Chapter 6 of ASCE/SEI 41 Supplement 1 (ASCE/SEI Ad Hoc Committee 2007). The most recent version, however, does not provide modeling procedures, acceptance criteria, and rehabilitation measures for precast concrete frames, infill frames, braced frames, shear walls, diaphragms, and foundations. 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 guide.

This guide should be used in conjunction with Chapters 1 through 4 of ASCE/SEI 41-06. Chapter 1 of ASCE/SEI 41-06 provides rehabilitation requirements, including description of ROs, Building Performance Levels, and seismic hazard. Chapter 2 of ASCE/SEI 41-06 provides general design requirements, including determination of as-built information, limitations for linear and nonlinear analysis procedures, definition of force- and deformation-controlled actions, procedures for construction quality assurance, and methods for determining alternative modeling parameters and acceptance criteria. Chapter 3 of ASCE/SEI 41-06 provides a detailed description of all linear and nonlinear analysis procedures referenced in ACI 369R. Chapter 4 of ASCE/SEI 41-06 provides geotechnical engineering provisions for building foundations and assessment of seismic-geologic site hazards. References to these chapters can be found throughout the guide. This guide provides short descriptions of potential seismic rehabilitation measures for each concrete building system. The design professional, however, is referred to the FEMA report, FEMA 547, for detailed information on seismic rehabilitation measures for concrete buildings. Repair techniques for earthquake-damaged concrete components are not included in ACI 369R. The 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 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 guide. 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 the American National Standards Institute (ANSI)/American Institute of Steel Construction (AISC) Code (ANSI/AISC 360). Lack of adequate confinement may result in expansion of the core concrete, which exacerbate bond slip and, consequently, 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.

CHAPTER 2—MATERIAL PROPERTIES AND CONDITION ASSESSMENT

2.1—General

Mechanical properties of materials should be obtained from available drawings, specifications, and other documents for the existing building. Where these documents fail to provide adequate information to quantify material properties, such information should be supplemented by materials testing based on recommendations of Chapter 2. The condition of the concrete components of the structure should be determined using the recommendations of Chapter 2. Section 2.2 of ASCE/SEI 41-06 provides further guidance on the assessment of as-built information for existing building structures.

Material properties of existing concrete components should be determined in accordance with Section 2.2. The use of default material properties based on historical information is permitted in accordance with Section 2.2.5. Condition assessment procedures are described in Section 2.3. Definition of a knowledge factor, based on whether materials testing and condition assessment follow the "usual" or "comprehensive" procedures described in Sections 2.2 and 2.3, is presented in Section 2.4.

C2.1—General

Chapter 2 identifies properties requiring consideration and provides guidelines for determining building properties. Also described is the need for a thorough condition assessment and utilization 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.

The form and function of concrete buildings, concrete strength and quality, reinforcing steel strength, quality and detailing, forming techniques, and concrete placement techniques have changed over the past century. These factors and changes, as a result of deterioration and prior loading history, have a significant impact on the seismic resistance of a concrete building. Innovations such as prestressed and precast concrete, post-tensioning, and lift-slab construction have created a diverse inventory of existing concrete structures.

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). Early uses of concrete did not specify design strength, and low-strength concrete was common. Early use of concrete in buildings often employed reinforcing steel 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, may be particularly difficult to assess due to 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 (WRI 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 design professional is encouraged to research and acquire all available records from original construction, including photographs, to confirm reinforcement details shown on the plans.

Design professionals seeking further guidance on the condition assessment of existing concrete buildings should refer to:

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

2.2—Properties of in-place materials and components

2.2.1 *Material properties*

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

1. Concrete compressive strength; and
2. Yield and ultimate strength of conventional and prestressing reinforcing steel and metal connection hardware.

Where materials testing is required by Section 2.2.6 of ASCE/SEI 41-06, the test methods to quantify material properties should comply with the requirements of Section 2.2.3 of ACI 369R. The frequency of sampling, including the minimum number of tests for property determination, should comply with the requirements of Section 2.2.4.

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

1. *Tensile strength and modulus of elasticity of concrete;*
2. *Ductility, toughness, and fatigue properties of concrete;*
3. *Carbon equivalent present in the reinforcing steel; and*
4. *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, Sections 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 reinforcing steel can be established from combined core and specimen sampling at similar locations, followed by laboratory testing. Core drilling should minimize damage to the existing reinforcing steel.

2.2.1.2 *Nominal or specified properties*—Nominal material properties, or properties specified in construction documents, should be taken as lower-bound material properties. Corresponding expected material properties can be calculated by multiplying lower-bound values by a factor taken from Table 2.1 to translate from lower-bound to expected values. Alternative factors may be used where justified by test data.

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

1. Cross-sectional dimensions of individual components and overall configuration of the structure;
2. Configuration of component connections, size of anchor bolts, thickness of connector material, anchorage and interconnection of embedments and the presence of bracing or stiffening components;
3. Modifications to components or overall configuration of the structure;
4. Most recent physical condition of components and connections, and the extent of any deterioration;
5. Deformations beyond those expected due to gravity loads, such as those caused by settlement or past earthquake events; and
6. Presence of other conditions that influence building performance, such as nonstructural components that may interfere with structural components during earthquake excitation.

C2.2.2 Component properties—*Component properties may be 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 lateral-force-resisting elements and systems, and their critical components and connections. If there are no drawings of the building, the design professional should perform a thorough investigation of the building to identify these elements, systems, and components (Section 2.3).*

2.2.3 *Test methods to quantify material properties*

2.2.3.1 *General*—Destructive and non-destructive test methods used to obtain in-place mechanical properties of materials identified in Section 2.2.1 and component properties identified in Section 2.2.2 are described in Section 2.2.3.1. Samples of concrete and reinforcing and connector steel should be examined for physical condition as described in Section 2.3.2.

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

Where Section 2.2.4.2.1 does not apply and the coefficient of variation is greater than 14%, the expected concrete strength should not exceed the mean less one standard deviation.

Table 2.1—Factors to translate lower-bound material properties to expected strength material properties

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

2.2.3.2 *Sampling*—For concrete material testing, the sampling program should include the removal of standard cores. Core drilling should be preceded by nondestructive location of the reinforcing steel, and core holes should be located to avoid damage to or drilling through the reinforcing steel. Core holes should be filled with concrete or grout of comparable strength. If conventional reinforcing steel is tested, sampling should include removal of local bar segments and installation of replacement spliced material to maintain continuity of the reinforcing bar for transfer of bar force.

Removal of core samples and performance of laboratory destructive testing can be used to determine existing concrete strength properties. Removal of core samples should employ the procedures included in ASTM C42/C42M. Testing should follow the procedures contained in ASTM C42/C42M, ASTM C39/C39M, and ASTM C496/C496M. Core strength should be converted to in-place concrete compressive strength by an approved procedure, such as that included in ACI 214.4R.

Removal of bar or tendon length samples and performance of laboratory destructive testing can be used to determine existing reinforcing steel strength properties. The tensile yield and ultimate strengths for reinforcing and prestressing steels should follow the procedures included in ASTM A370. Prestressing materials should meet the supplemental requirements in ASTM A416/A416M, ASTM A421/A421M, or ASTM A722/A722M, depending on material type. Properties of connector steels can be determined by wet and dry chemical composition tests, and direct tensile and compressive strength tests as specified by ASTM A370. Where strengths of embedded connectors are required, in-place testing *should satisfy the provisions of ASTM E488.*

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 may be determined from retrieved samples to assess the condition of the concrete. FEMA 274 (Section C6.3.3.2) provides references for these tests.*

The reinforcing steel system used in the construction of a specific building is usually a common grade and strength. One grade of reinforcement is occasionally used for small-diameter bars, like those used for stirrups and hoops, and another grade for large-diameter bars, like 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 (Section 2.2.5). This information can be used with laboratory and field test data to gain confidence in in-place strength properties.

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

Based on Chapter 2 of ASCE/SEI 41-06, data collection from material tests is classified as either comprehensive or usual (refer to Definitions, Section 5.2). The minimum number of tests for usual data collection is provided in Section 2.2.4.1. The minimum number of tests necessary to quantify properties by in-place testing for comprehensive data collection is provided in Section 2.2.4.2. If the existing gravity- or lateral force-resisting system is replaced during the rehabilitation process, material testing is only needed to quantify properties of existing materials at new connection points.

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 lateral 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 may influence the testing program scope. The focus of testing should be on primary lateral force-resisting components and specific properties for analysis. Test quantities provided in this section are minimal; the 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 lateral force-resisting system.

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

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

1. If the specified design strength of the concrete is known, at least one core should 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;
2. If the specified design strength of the concrete is not known, at least one core should be taken from each type of lateral force-resisting component, with a minimum of six cores taken for the entire building;
3. If the specified design strength of the reinforcing steel is known, nominal or specified material properties can be used without additional testing; and
4. If the specified design strength of the reinforcing steel is not known, at least two strength coupons of reinforcing steel should be removed from the building for testing.

2.2.4.2 *Comprehensive data collection*

2.2.4.2.1 *Coefficient of variation*—Unless specified otherwise, a minimum of three tests should be conducted to determine any property. If the coefficient of variation exceeds 14%, additional tests should be performed until the coefficient of variation is equal to or less than 14%. The number of tests in a single component should be limited so as not to compromise the integrity of the component.

2.2.4.2.2 *Concrete materials*—For each concrete element type, a minimum of three core samples should be taken and subjected to compression tests. A minimum of six total tests should be performed on a building for concrete strength determination, subject to the limitations of this section. If varying concrete classes/grades were employed in the building construction, a minimum of three samples and tests should be performed for each class. The modulus of elasticity and tensile strength can be estimated from the compressive strength testing data. Samples should be taken from components, distributed throughout the building, that are critical to the structural behavior of the building.

Tests should be performed on samples from components that are identified as damaged or degraded to quantify their condition. Test results from areas of degradation should 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 should be performed to determine the cause or identify the degree of damage or degradation.

The minimum number of tests to determine compressive strength should conform to the following criteria:

1. For concrete elements for which the specified design strength is known and test results are not available, a minimum of three cores/tests should be conducted for each floor level, 400 yd^3 (306 m^3) of concrete, or 10,000 ft^2 (930 m^2) of surface area, whichever requires the most frequent testing; or
2. For concrete elements for which the design strength is unknown and test results are not available, a minimum of six cores/tests should be conducted for each floor level, 400 yd^3 (306 m^3) of concrete, or 10,000 ft^2 (930 m^2) of surface area, whichever requires the most frequent testing. Where the results indicate that different classes of concrete were employed, the degree of testing should be increased to confirm class use.

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

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.*

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 only. Guidance for nondestructive test methods is provided in ACI 228.2R.

2.2.4.2.3 *Conventional reinforcing and connector steels*—Section 2.2.4.2.3 is a guide to the minimum number of tests required to determine reinforcing and connector steel strength properties. Connector steel is defined as additional structural steel or miscellaneous metal used to secure precast and other concrete shapes to the building structure. Tests should determine both yield and ultimate strengths of

Table 2.2—Default lower-bound compressive strength of structural concrete, psi (MPa)

Time frame	Footings	Beams	Slabs	Columns	Walls
1900-1919	1000 to 2500 (7 to 17)	2000 to 3000 (14 to 21)	1500 to 3000 (10 to 21)	1500 to 3000 (10 to 21)	1000 to 2500 (7 to 17)
1920-1949	1500 to 3000 (10 to 21)	2000 to 3000 (14 to 21)	2000 to 3000 (14 to 21)	2000 to 4000 (14 to 28)	2000 to 3000 (14 to 21)
1950-1969	2500 to 3000 (17 to 21)	3000 to 4000 (21 to 28)	3000 to 4000 (21 to 28)	3000 to 6000 (21 to 40)	2500 to 4000 (17 to 28)
1970-present	3000 to 4000 (21 to 28)	3000 to 5000 (21 to 35)	3000 to 5000 (21 to 35)	3000 to 10,000 (21 to 70)	3000 to 5000 (21 to 35)

Table 2.3—Default lower-bound tensile and yield properties of reinforcing steel for various periods*

Year	Grade	Structural† 33	Intermediate† 40	Hard† 50	60	65	70	75
	Minimum yield, psi (MPa)	33,000 (230)	40,000 (280)	50,000 (350)	60,000 (420)	65,000 (450)	70,000 (485)	75,000 (520)
	Minimum tensile, psi (MPa)	55,000 (380)	70,000 (485)	80,000 (550)	90,000 (620)	75,000 (520)	80,000 (550)	100,000 (690)
1911-1959		x	x	x		x		
1959-1966		x	x	x	x	x	x	x
1966-1972			x	x	x	x	x	
1972-1974			x	x	x	x	x	
1974-1987			x	x	x	x	x	
1987-present			x	x	x	x	x	x

*An entry of "x" indicates the grade was available in those years.
†The terms "structural," "intermediate," and "hard" became obsolete in 1968.

reinforcing and connector steel. A minimum of three tensile tests should be conducted on conventional reinforcing steel samples from a building for strength determination, subject to supplemental conditions:

1. For construction between 1972 and 1995, no testing is required if original construction documents define the reinforcing steel as ASTM A615/A615M, Grades 40 or 60, and connector steel as ASTM A36/A36M;
2. If original construction documents defining properties exist, then at least three strength coupons should be randomly removed from each element or component type and tested; or
3. 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 should be randomly removed from each element or component type for every three floors of the building; and
4. If the construction date is unknown, at least six samples/tests for every three floors should be performed.

All sampled steel should be replaced with new fully spliced and connected material unless an analysis confirms that replacement of original components is not required.

2.2.4.2.4 *Prestressing steels*—Sampling prestressing steel tendons for laboratory testing should only be performed on prestressed components that are part of the lateral force-resisting system. Prestressed components in diaphragms can be excluded.

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

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

2.2.5 *Default properties*—Default material properties to determine component strengths can be used in conjunction with the linear analysis procedures of ASCE/SEI 41-06 Chapter 3.

Default lower-bound concrete compressive strengths are given in Table 2.2. Default expected concrete compressive strengths can be determined by multiplying lower-bound values by an appropriate factor selected from Table 2.1, unless another factor is justified by test data. The appropriate default compressive strength—lower-bound or expected strength, as specified in Section 2.4.4 of ASCE/SEI 41-06—should be used to establish other strength and performance characteristics for the concrete as needed in the structural analysis.

Default lower-bound values for reinforcing steel are given for various ASTM specifications and periods in Tables 2.3 or 2.4. Default expected strength values for reinforcing steel can be determined by multiplying lower-bound values by an appropriate factor selected from Table 2.1, unless another factor is justified by test data. Where default values are assumed for existing reinforcing steel, welding or mechanical coupling of new reinforcement to the existing reinforcing steel should not be used.

The default lower-bound yield strength for steel connector material can be taken as 27,000 psi (186 MPa). The default

Table 2.4—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 (MPa)	33,000 (230)	40,000 (280)	50,000 (350)	60,000 (420)	65,000 (450)	70,000 (485)	75,000 (520)
			Minimum tensile, psi (MPa)	55,000 (380)	70,000 (485)	80,000 (550)	90,000 (620)	75,000 (520)	80,000 (550)	100,000 (690)
ASTM designation‡	Steel type	Year range								
A15 (withdrawn)	Billet	1911-1966		x	x	x				
A16 (withdrawn)	Rail§	1913-1966				x				
A61 (withdrawn)	Rail§	1963-1966					x			
A160 (withdrawn)	Axle	1936-1964		x	x	x				
A160 (withdrawn)	Axle	1965-1966		x	x	x	x			
A185	WWR	1936-present						x		
A408 (withdrawn)	Billet	1957-1966		x	x	x				
A431	Billet	1959-1966								x
A432 (withdrawn)	Billet	1959-1966					x			
A497	WWR	1964-present							x	
A615/A615M	Billet	1968-1972			x		x			x
A615/A615M	Billet	1974-1986			x		x			
A615/A615M	Billet	1987-present			x		x			x
A616‖ (withdrawn)	Rail§	1968-present				x	x			
A617 (withdrawn)	Axle	1968-present			x		x			
A706/A706M#	Low-alloy	1974-present					x			
A955	Stainless	1996-present			x		x			x

Notes:
*An entry of "x" indicates the grade was available in those years.
†The terms "structural," "intermediate," and "hard" became obsolete in 1968.
‡ASTM steel is marked with the letter "W."
§Rail bars are marked with the letter "R."
‖Bars marked "s!" (ASTM A616 (withdrawn)) have supplementary requirements for bend tests.
#ASTM A706/A706M has a minimum tensile strength of 80 ksi (550 MPa), but not less than 1.25 times the actual yield strength.

expected yield strength for steel connector material can be determined by multiplying lower-bound values by an appropriate factor selected from Table 2.1, unless another value is justified by test data.

Default values for prestressing steel in prestressed concrete construction should not be used.

C2.2.5 Default properties—*Default values provided in this standard are generally conservative. Whereas the strength of reinforcing steel may be fairly consistent throughout a building, the strength of concrete in a building could be highly variable, given variability in concrete mixture designs and sensitivity to water-cement ratio (w/cm) and curing practices. A conservative assumption of the minimum value 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; a conservative assumption would be the maximum value in a given range where determining the force-controlled actions on other components.*

Until about 1920, a variety of proprietary reinforcing steels was used. Yield strengths are likely to be in the range of 33,000 to 55,000 psi (230 to 380 MPa), but higher values are possible and actual yield and tensile strengths may exceed minimum values. Once commonly used to designate reinforcing steel 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 reinforcing steel strength to expected strength include consideration of material overstrength and strain hardening.

2.3—Condition assessment

2.3.1 *General*—A condition assessment of the existing building and site conditions should be performed as described.

The condition assessment should include:

1. Examination of the physical condition of primary and secondary components, and the presence of any degradation should be noted;
2. Verification of the presence and configuration of components and their connections, and the continuity of load paths between components, elements, and systems;
3. A review and documentation of other conditions, including neighboring party walls and buildings, presence of nonstructural components and mass, prior remodeling, and limitations for rehabilitation that may influence building performance or seismic loads;
4. Collection of information needed to select a knowledge factor in accordance with Section 2.4; and
5. Confirmation of component orientation, plumbness, and physical dimensions confirmed.

2.3.2 *Scope and procedures*—The scope of the condition assessment should 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 should be performed to:

- Identify configuration issues;
- Determine if degradation is present;
- Establish continuity of load paths;
- Establish the need for other test methods that could quantify the presence and degree of degradation; and
- Measure dimensions of existing construction to compare with available design information and reveal any permanent deformations.

A visual building inspection should include visible portions of foundations, lateral-force-resisting members, diaphragms (slabs), and connections. As a minimum, a representative sampling of at least 20% of the components and connections should 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 should be increased to 40% or more, as necessary to accurately assess the performance of components and connections with degradation.

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

C2.3.2.1 Visual condition assessment—*Design professionals seeking further guidance can consult ACI 201.1R, which provides a system for reporting the condition of concrete in service.*

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 should be replaced after inspection. The following criteria can be used for assessing primary connections in the building for comprehensive data collection:

1. If detailed design drawings exist, exposure of at least three different primary connections should occur, with the connection sample including different types of connections (for example, beam-column, column-foundation, and beam-diaphragm). If no deviations from the drawings exist or if consistent deviations from the drawings exist, it is appropriate to consider the sample as being representative of installed conditions. If inconsistent deviations are noted, then at least 25% of the specific connection type should be inspected to identify the extent of deviation; or
2. In the absence of detailed design drawings, at least three connections of each primary connection type should be exposed for inspection. If common detailing among the three connections is observed, it is appropriate to consider this condition as representative of installed conditions. If variations are observed among like connections, additional connections should 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, approved test methods should be used.

C2.3.2.3 Additional testing—*The physical condition of components and connectors will affect their performance. The need to accurately identify the physical condition may dictate the need for certain additional destructive and nondestructive test methods. Such methods may 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 may be used in the condition assessment are provided in ACI 228.1R, ACI 228.2R, FEMA 274 (Section C6.3.3.2), and FEMA 306 (Section 3.8).*

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

- *Surface NDE methods include infrared thermography, delamination sounding, surface hardness measurement, and crack mapping. These methods may be used to find surface degradation in components such as service-induced cracks, corrosion, and construction defects;*
- *Volumetric NDE methods, including radiography and ultrasonics, may be used to identify the presence of internal discontinuities and loss of section. Impact-echo ultrasonics is particularly useful because of ease of implementation and proven capability in concrete;*
- *On-line monitoring using acoustic emissions, strain gauges, in-place static or dynamic load tests, and ambient vibration tests may be used to assess structural condition 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;*
- *Electromagnetic methods using a pachometer or radiography may be used to locate, size, or perform an initial assessment of reinforcing steel. Further assessment of suspected corrosion activity should use electrical half-cell potential and resistivity measurements; and*
- *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, may be used where absolutely essential to determine the level of prestress remaining in an unbonded prestress system.

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

1. Component section properties and dimensions;
2. Component configuration and the presence of any eccentricities or permanent deformation;
3. Connection configuration and the presence of any eccentricities;
4. Presence and effect of alterations to the structural system since original construction; and
5. Interaction of nonstructural components and their involvement in lateral load resistance.

All deviations between available construction records and as-built conditions obtained from visual inspection should 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 should be assumed to be those from the design drawings after adjustment for as-built conditions. If some sectional material loss has occurred, the loss should 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 capacities and permissible deformations is selected in accordance with Section 2.2.6.4 of ASCE/SEI 41-06 with additional requirements specific to concrete components. A knowledge factor κ equal to 0.75 should be used if any of the following criteria are met:

1. 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 $\kappa = 1.0$;
2. Mechanical properties have a coefficient of variation exceeding 25%; and
3. Components contain archaic or proprietary material and the condition is uncertain.

CHAPTER 3—GENERAL ASSUMPTIONS AND REQUIREMENTS

3.1—Modeling and design

3.1.1 *General approach*—Seismic rehabilitation of a concrete building will involve the design of new components connected to the existing structure, seismic upgrading of existing components, or both. New components typically should comply with ACI 318/318M. Seismic rehabilitation methods can be selected to achieve various performance levels, including Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). When the structural performance level differs from that implied in ACI 318/318M-08, Chapter 21, new components may not need to comply with ACI 318/318M-08. In such cases, components should be assessed using the provisions of ACI 369R and design should be peer reviewed as required by the authority having jurisdiction.

Original and rehabilitated components of an existing building are not expected to satisfy provisions of ACI 318/318M, but should be assessed using the provisions of ACI 369R. Brittle or low-ductility failure modes should be identified as a part of the seismic evaluation.

Evaluation of demands and capacities of reinforced concrete components should include consideration of locations along the length where lateral 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, may produce stress concentrations that result in premature failure.

C3.1.1 General approach—*When applied to new structures, provisions of ACI 318/318M are intended to provide LS performance for the design basis earthquake. In many cases, this level of performance is appropriate for new components added to a building during seismic rehabilitation. When assessing the building for higher performance levels, such as IO, where essentially elastic response is expected, it may not be necessary to satisfy all provisions of ACI 318/318M intended to provide ductile behavior of concrete components (for example, column confinement requirements of ACI 318/318M-08, Chapter 21).*

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.

3.1.2 *Stiffness*—Component stiffnesses should be calculated considering shear, flexure, axial behavior, and reinforcement slip deformations. Stress state of the component, cracking extent due to volumetric changes from temperature and shrinkage, deformation levels under gravity, and earthquake loading should be considered.

C3.1.2 Stiffness—*For columns with low axial loads (below approximately $0.1A_g f_c'$), deformations due to bar slip can account for as much as 50% of the total deformations at yield. The design professional is referred to Elwood and Eberhard (2009) for further guidance regarding calculation of the effective stiffness of reinforced concrete columns that include the effects of flexure, shear, and bar slip.*

3.1.2.1 *Linear procedures*—Where design actions are determined using linear procedures (Chapter 3, ASCE/SEI 41-06), component effective stiffnesses should correspond to the secant value to the yield point of the component. Higher stiffnesses can be used where it is demonstrated by analysis to be appropriate for the design loading. Effective stiffness values in Table 3.1 can also be used.

C3.1.2.1 Linear procedures—*The effective flexural rigidity values in Table 3.1 for beams and columns account for the additional flexibility from reinforcement slip within the beam-column joint or foundation before yielding. The*

Table 3.1—Effective stiffness values

Component	Flexural rigidity	Shear rigidity	Axial rigidity
Beams—nonprestressed	$0.3E_cI_g$	$0.4E_cA_w$	—
Beams—prestressed	E_cI_g	$0.4E_cA_w$	—
Columns with compression due to design gravity loads $\geq 0.5A_gf_c'$	$0.7E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns with compression due to design gravity loads $\leq 0.1A_gf_c'$ or with tension	$0.3E_cI_g$	$0.4E_cA_w$	E_cA_g (compression) E_sA_s (tension)
Beam-column joints	Refer to Section 4.2.2.1		E_cA_g
Flat slabs—nonprestressed	Refer to Section 4.4.2	$0.4E_cA_g$	—
Flat slabs—prestressed	Refer to Section 4.4.2	$0.4E_cA_g$	—

Note: For T-beams, I_g can be taken as twice the value of I_g of the web alone. Otherwise, I_g should be based on the effective width as defined in Section 3.1.3. For columns with axial compression falling between the limits provided, flexural rigidity should be determined by linear interpolation. If interpolation is not performed, the more conservative effective stiffnesses should be used.

*values specified for columns were determined based on a database of 221 rectangular reinforced concrete column tests with axial loads less than 0.67*A_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 0.2*EI_g*; however, considering the scatter in the effective flexural rigidity and to avoid under-estimating the shear demand on columns with low axial loads, 0.3*EI_g *is recommended in Table 3.1 (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 due to flexure, slip, and shear (Elwood and Eberhard 2009).*

The modeling recommendations for beam-column joints (Section 4.2.2.1) do not include the influence of reinforcement slip. When the effective stiffness values for beams and columns from Table 3.1 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 (Saatcioglu 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 0.3*A_gf_c'*, the effective flexural rigidity can be estimated as 0.5*EI_g*, with linear interpolation to the value given in Table 3.1 for axial loads greater than 0.5*A_gf_c'*.*

*Due to low bond stress between concrete and plain reinforcement without deformations, components with plain longitudinal reinforcement and axial loads less than 0.5*A_gf_c' *may have lower effective flexural rigidity values than in Table 3.1.*

3.1.2.2 *Nonlinear procedures*—Where design actions are determined using nonlinear procedures (ASCE/SEI 41-06, Chapter 3), component load-deformation response should be represented by nonlinear load-deformation relations. Linear relations can be used where nonlinear response will not occur in the component. The nonlinear load-deformation relation should be based on experimental evidence or taken from quantities specified in Chapter 4. For NSP, the generalized load-deformation relation shown in Fig. 3.1 or other curves defining behavior under monotonically increasing deformation can be used. For the nonlinear dynamic procedure (NDP), load-deformation relations should define behavior under monotonically increasing lateral deformation and under multiple reversed deformation cycles as specified in Section 3.2.1.

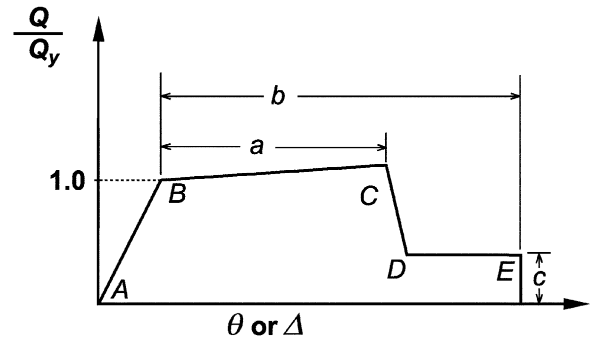


Fig. 3.1—Generalized force-deformation relation for concrete elements or components.

The generalized load-deformation relation shown in Fig. 3.1 can 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 lateral load resistance to point D, then response at reduced resistance to E, and final loss of resistance thereafter. The slope from point A to B should be determined according to Section 3.1.2.1. The slope from point B to C, ignoring effects of gravity loads acting through lateral displacements, should be taken between zero and 10% of the initial slope, unless an alternate slope is justified by experiment or analysis. Point C should 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-E) can be used if the calculated response does not exceed point C. Numerical values for the points identified in Fig. 3.1 are provided in Sections 4.2.2.2 (beams, columns, and joints) and 4.4.2.2 (slab-column connections). Other load-deformation relations are permitted if justified by experimental evidence or analysis.

C3.1.2.2 Nonlinear procedures—*Typically, the response shown in Fig. 3.1 are 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 accommodates strain hardening in the load-deformation relation as the member is deformed toward the expected strength. Where the response shown in Fig. 3.1 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 may 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. 3.1 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.*

In Fig. 3.1 deformations are expressed directly using terms such as strain, curvature, rotation, or elongation. The parameters "a" and "b" refer to deformation portions that occur after yield, or plastic deformation. The parameter c is the reduced resistance after the sudden reduction from C to D. Parameters "a," "b," and "c" are defined numerically in various tables in ACI 369R. Alternatively, parameters "a," "b," and "c" 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 Section 2.8 of ASCE/SEI 41-06.

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. 3.1 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 3.1 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.

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 should be calculated considering a width of effective flange on each side of the web equal to the smaller of:

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

Where the flange is in compression, the concrete and reinforcement within the effective width should 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 should be considered fully effective for resisting flexural and axial loads. The portion of the flange extending beyond the width of the web should be assumed ineffective in resisting shear.

In walls, effective flange width should be computed using Chapter 21 of ACI 318/318M-08.

3.2—Strength and deformability

3.2.1 *General*—Actions in a structure should 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 4.1, 4.2, 4.4, and Tables 4.6 through 4.10. Where linear and nonlinear acceptance criteria are not specified in the tables, actions should be taken as force-controlled unless component testing is performed in accordance with Section 2.8 of ASCE/SEI 41-06. Design strengths for deformation-controlled and force-controlled actions should be calculated in accordance with Sections 3.2.2 and 3.2.3 of this guide, respectively.

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

Where strength and deformation capacities are derived from test data, the tests should be representative of proportions, details, and stress levels for the component and comply with Section 2.8.1 of ASCE/SEI 41-06. Further guidance on the testing of moment-frame components can be found in ACI 374.1.

The strength and deformation capacities of concrete members should 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.

C3.2.1 General—*ASCE/SEI 41-06 and this guide classify actions as either deformation-controlled or force-controlled. Actions are considered to be deformation-controlled where the component behavior is well documented by test results. Where linear or nonlinear acceptance criteria are tabulated in this guide, 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 design professional may opt to perform testing to validate the classification of deformation-controlled. Section 2.8 of ASCE/SEI 41-06 provides guidance on procedures to be followed during testing and Section 2.4.4.3 of ASCE/SEI 41-06 provides a methodology based on the test data to distinguishing force-controlled from deformation-controlled actions.*

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

3.2.2 *Deformation-controlled actions*—Strengths used for deformation-controlled actions should be taken as equal to expected strengths Q_{CE} obtained experimentally, or calculated using accepted principles of mechanics. Expected strength is defined as the mean maximum resistance expected over the range of deformations that a concrete component is likely subjected to. Where calculations are used to define expected strength, expected material properties should be used as well. Unless specified in this guide, other procedures specified in ACI 318/318M to calculate design strengths may be used, except that the strength reduction factor ϕ should be taken equal to unity. Deformation capacities for acceptance of deformation-controlled actions calculated by nonlinear procedures should be as specified in Chapter 4. For components constructed of lightweight concrete, Q_{CE} should be modified in accordance with ACI 318/318M procedures for lightweight concrete.

C3.2.2 *Deformation-controlled actions—Expected yield strength of reinforcing steel as specified in Section 2.2.1.2 of this guide includes material overstrength considerations.*

3.2.3 *Force-controlled actions*—Strengths used for force-controlled actions should 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 should be used. Unless other procedures are specified in this standard, procedures specified in ACI 318/318M to calculate design strengths may be used, except that the strength reduction factor ϕ should be taken equal to unity. For components constructed of lightweight concrete, Q_{CL} should be modified in accordance with ACI 318/318M procedures for lightweight concrete.

3.2.4 *Component ductility demand classification*—Table 3.2 provides classification of component ductility demand as low, moderate, or high based on the maximum value of the demand capacity ratio (DCR) (refer to Section 2.4.1 of ASCE/SEI 41-06) for linear procedures or the calculated displacement ductility for nonlinear procedures.

3.3—Flexure and axial loads

Flexural strength of members with and without axial loads should be calculated according to ACI 318/318M 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 should be calculated considering shear, flexure, and reinforcement slip deformations, or may be determined based on acceptance criteria given in Chapter 4 of ACI 369R. Strengths and deformation capacities of components with monolithic flanges should be calculated considering concrete and developed longitudinal reinforcement within the effective flange width (Section 3.1.3).

Strength and deformation capacities should 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 should be calculated based on limiting stress capacity of the embedded bar as defined in Section 3.5.

Where flexural deformation capacities are calculated from basic principles of mechanics, reductions in deformation capacity due to applied shear should be considered. Where using analytical models for flexural deformability that do not directly consider effect of shear and design shear equals or exceeds $6\sqrt{f_c'}A_w$, psi ($0.5\sqrt{f_c'}A_w$, MPa), the design value should not exceed 80% of the value calculated using the analytical model.

Table 3.2—Component ductility demand classification

Maximum value of DCR or displacement ductility	Descriptor
< 2	Low ductility demand
2 to 4	Moderate ductility demand
> 4	High ductility demand

For concrete columns under combined axial load and biaxial bending, the combined strength should be evaluated considering biaxial bending. When using linear procedures, the design axial load P_{UF} should be calculated as a force-controlled action (Section 3.4.2, ASCE/SEI 41-06). The design moments M_{UD} should be calculated about each principal axis (Section 3.4, ASCE/SEI 41-06). Acceptance should be based on

$$\left(\frac{M_{UDx}}{m_x \kappa M_{CEx}}\right)^2 + \left(\frac{M_{UDy}}{m_y \kappa M_{CEy}}\right)^2 \leq 1 \qquad (3\text{-}1)$$

Alternative approaches based on principles of mechanics can be used.

C3.3—Flexure and axial loads

Laboratory tests indicate that flexural deformability may be reduced as coexisting shear forces increase. As flexural ductility demands increase, shear capacity decreases, which may result in a shear failure before theoretical flexural deformation capacities are reached. Use caution where flexural deformation capacities are determined by calculation. FEMA 306 (Section 5.2) is a resource for guidance on the interaction between shear and flexure.

3.3.1 *Usable strain limits*—Without confining transverse reinforcement, the maximum usable strain at the extreme concrete compression fiber should not exceed 0.002 for components in nearly pure compression and 0.005 for other components, unless larger strains are substantiated by experimental evidence and approved by the authority having jurisdiction. Maximum usable compressive strains for confined concrete should 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. Maximum compressive strains in longitudinal reinforcement should not exceed 0.02 and maximum tensile strains in longitudinal reinforcement should not exceed 0.05. Monotonic coupon test results should not be used to determine reinforcement strain limits. If experimental evidence is used to determine strain limits, the effects of low-cycle fatigue and transverse reinforcement spacing and size should be included in testing procedures (Brown and Kunnath 2004). Results are subject to the approval of the authority having jurisdiction.

C3.3.1 *Usable strain limits—Reinforcement tensile strain limit 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. The California Department of Transportation (Caltrans) Seismic Design Criteria (Caltrans 2006) recommends an ultimate tensile strain of 0.09 for No. 10 (No. 32) bars and smaller, and 0.06 for No. 11 (No. 36) bars and larger, for ASTM A706/A706M 60 ksi (420 MPa) reinforcing bars. A lower bound is selected here considering the variability in materials and details typically found in existing structures.*

3.4—Shear and torsion

Strengths in shear and torsion should be calculated according to ACI 318/318M, except as modified in this guide.

Within yielding regions of components with moderate or high ductility demands, shear and torsional strength should be calculated according to procedures for ductile components, such as the provisions in Chapter 21 of ACI 318/318M-08. Within yielding regions of components with low ductility demands (Table 3.2) and outside yielding regions for all ductility demands, procedures for effective elastic response, such as the provisions in Chapter 11 of ACI 318/318M-08, can be used to calculate the design shear strength.

Where the longitudinal spacing of transverse reinforcement exceeds half the component effective depth measured in the direction of shear, transverse reinforcement should be assumed not more than 50% effective in resisting shear or torsion. Where the longitudinal spacing of transverse reinforcement exceeds the component effective depth measured in the direction of shear, transverse reinforcement should be assumed ineffective in resisting shear or torsion. For beams and columns, lap-spliced transverse reinforcement should be assumed not more than 50% effective in regions of moderate ductility demand and ineffective in regions of high ductility demand.

Shear friction strength should be calculated according to ACI 318/318M, considering the expected axial load from gravity and earthquake effects. Where rehabilitation involves the addition of concrete requiring overhead work with dry-pack, the shear friction coefficient μ should be taken as equal to 70% of the value specified by ACI 318/318M.

3.5—Development and splices of reinforcement

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

1. Deformed straight, hooked, and lap-spliced bars should meet the development requirements of Chapter 12 of ACI 318/318M, except for lap splices, which should be the same as those for straight development of bars in tension without consideration of lap splice classifications (Orangun et al. 1977);
2. Where existing deformed straight bars, hooked bars, and lap-spliced bars do not meet the development requirements of (1) above, the capacity of existing reinforcement should be calculated using Eq. (3-2)

$$f_s = 1.25\left(\frac{\ell_b}{\ell_d}\right)^{2/3} f_{yL} \tag{3-2}$$

   but should not exceed the expected or lower-bound yield strength, as applicable.

   Where transverse reinforcement is distributed along the development length with spacing not exceeding one-third of the effective component depth, it can be assumed that the reinforcement retains the calculated maximum stress to high ductility demands. For larger spacings of transverse reinforcement, the developed stress should 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;
3. Strength of deformed straight, discontinuous bars embedded in concrete sections or beam-column joints, with clear cover over the embedded bar not less than $3d_b$, should be calculated according to Eq. (3-3)

$$f_s = \frac{2500}{d_b}\ell_e \le f_y \quad \text{(psi units)}$$

$$\tag{3-3}$$

$$f_s = \frac{17}{d_b}\ell_e \le f_y \quad \text{(psi units)}$$

   where f_s is less than f_y, and the calculated stress in the bar due to design loads equals or exceeds f_s, the maximum developed stress should be assumed to degrade from $1.0f_s$ to $0.2f_s$ at a ductility demand or DCR equal to 2.0. In beams with short bottom bar embedments into beam-column joints, flexural strength should be calculated considering the stress limitation of Eq. (3-3);
4. For plain straight, hooked, and lap-spliced bars, development and splice lengths should be taken as twice the values determined in accordance with ACI 318/318M, unless other lengths are justified by approved tests or calculations considering only the chemical bond between the bar and concrete; and
5. Doweled bars added in seismic rehabilitation can be assumed to develop yield stress where all the following conditions are satisfied:
   a) Drilled holes for dowel bars are cleaned with a stiff brush that extends the length of the hole;
   b) Embedment length ℓ_e is not less than $10d_b$;
   c) Minimum dowel bar spacing is not less than $4\ell_e$ and minimum edge distance is not less than $2\ell_e$. Design values for dowel bars not satisfying these conditions should be verified by test data. Field samples should be obtained to ensure design strengths are developed in accordance with Chapter 3.

C3.5—Development and splices of reinforcement

Development requirements in accordance with Chapter 12 of ACI 318/318M will be applicable to development of bars in all components. Chapter 21 of ACI 318/318M 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 21 of ACI 318/318M-08. Chapter 12 of ACI 318/318M-08 permits reductions in lengths if minimum cover and confinement are present in an existing component.

Equation (3-2), which is a modified version of the model presented by Cho and Pincheira (2006), reflects the intent of ACI code development and splice equations to develop 1.25 times the nominal bar strength, referred to in this guide as the lower-bound 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.

For buildings constructed prior to 1950, the bond strength developed between reinforcing steel and concrete may be less than present-day strength. Present equations for development and splices of reinforcement account for mechanical bond from deformations present in deformed bars as well as chemical bond. The length required to develop plain bars will be 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 reinforcing steel are found in Concrete Reinforcing Steel Institute (CRSI) Engineering Data Report 48 (CRSI DA24).

3.5.1 *Square reinforcing bars*—Square reinforcing bars in a building should be classified as either twisted or straight. The developed strength of twisted square bars should be as specified for deformed bars in Section 3.5, using an effective diameter calculated based on the gross area of the square bar. Straight square bars should be considered as plain bars and the developed strength should be as specified for plain bars (Section 3.5).

3.6—Connections to existing concrete

Connections used to connect two or more components can be classified according to their anchoring systems as cast-in-place or as post-installed, and designed according to Appendix D of ACI 318/318M as modified in Section 3.6. These provisions do not apply to connections in plastic hinge zones.

C3.6—Connections to existing concrete

Appendix D of ACI 318/318M 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 Appendix D is applicable. ACI 355.2 describes 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.

ASCE/SEI 41-06 Section 6.3.6.1 requires 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, prior to the introduction of ACI 318/318M Appendix D. Because cracking is now accounted for by Appendix D, the 0.5 factor is not required in Section 3.6 of ACI 369R.

3.6.1 *Cast-in-place systems*—Component actions on cast-in-place connection systems, including shear forces, tension forces, bending moments, and prying actions, should be considered force-controlled. Lower-bound strength of connections should be ultimate values as specified in Appendix D of ACI 318/318M with $\phi = 1.0$.

3.6.2 *Post-installed anchors*—Component actions on post-installed anchor connection systems should be considered force-controlled. The lower-bound capacity of post-installed anchors should be ultimate values as specified in Appendix D of ACI 318/318M with $\phi = 1.0$, or mean less one standard deviation of ultimate values published in approved test reports.

3.6.3 *Quality assurance*—Connections between existing concrete components and new ones added for structural seismic rehabilitation should be subject to quality assurance provisions specified in Section 2.7 of ASCE/SEI 41-06. The design professional should specify the required inspection and testing of cast-in-place and post-installed anchors as part of the quality assurance plan.

3.7—Rehabilitation: general requirements

Concrete components in an existing building that are determined deficient for the selected RO should be addressed through seismic rehabilitation. This may include replacement or rehabilitation of the component or modification of the structure so the component is no longer deficient for the selected RO. If component replacement is selected, the new component should be designed in accordance with this guide and detailed and constructed in compliance with a building code approved by the authority having jurisdiction.

Rehabilitation measures should be evaluated as required by this guide to assure that the completed rehabilitation achieves the selected RO. The effects of rehabilitation on stiffness, strength, and deformability should be taken into account in an analytical model of the rehabilitated structure. The compatibility of new and existing components should be checked at displacements consistent with the selected performance level.

Connections required between existing and new components should satisfy the requirements of Section 3.6 and other requirements of this guide.

CHAPTER 4—CONCRETE MOMENT FRAMES

4.1—Types of concrete moment frames

Concrete moment frames are defined as elements comprised primarily of horizontal framing components, such as beams and slabs, or both; vertical framing components, such as columns; and joints connecting horizontal and vertical framing components. To resist lateral 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 Chapter 4. Frames addressed include reinforced concrete beam-column moment frames, post-tensioned concrete beam-column moment frames, and slab-column moment frames. Precast concrete frames, concrete frames with infills, and concrete braced frames are not addressed in this guide.

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

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

1. Framing components are beams (with or without slabs), columns, and their connections;
2. Frames are of monolithic construction that provides for moment and shear transfer between beams and columns; and
3. Primary reinforcement in components contributing to lateral load resistance is nonprestressed.

Special moment frames, intermediate moment frames, and ordinary moment frames, as defined in ASCE/ACI 7 and ACI 318/318M, are deemed to satisfy the above conditions.

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

1. Framing components are beams (with or without slabs), columns, and their connections;
2. Frames of monolithic construction provide for moment and shear transfer between beams and columns; and
3. Primary reinforcement in beams contributing to lateral load-resistance includes post-tensioned reinforcement with or without mild reinforcement.

4.1.3 *Slab-column moment frames*—Slab-column moment frames (Section 4.4) are defined by the following conditions:

1. Framing components are slabs (with or without beams in the transverse direction), columns, and their connections;
2. Frames of monolithic construction provide for moment and shear transfer between slabs and columns; and
3. Primary reinforcement in slabs contributing to lateral load-resistance includes nonprestressed reinforcement, prestressed reinforcement, or both.

4.2—Reinforced concrete beam-column moment frames

4.2.1 *General considerations*—The analytical model for a beam-column frame element should 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 should be considered. Interaction with other elements, including nonstructural components, should be included.

Analytical models representing a beam-column frame using line elements with properties concentrated at component centerlines are acceptable. Where beam and column centerlines do not intersect, the eccentricity effects between framing centerlines should be considered. Where the centerline of the narrower component falls within the middle third of the adjacent framing component measured transverse to the framing direction, this eccentricity need not be considered. Where larger eccentricities occur, the effect should 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 should be modeled according to Section 4.2.2 or as justified by experimental evidence. The model of the connection between columns and foundation should 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 should be represented. Action of the slab as a composite beam flange should be considered in developing stiffness, strength, and deformation capacities of the beam component model (Section 3.1.3).

Inelastic action should be restricted to those components and actions listed in Tables 4.1, 4.2, and 4.4, 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 Section 4.2.4.

C4.2.1 General considerations—*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. ASCE/SEI 41-06 Section 3.2.2.3 suggests that nonstructural components should be included if their lateral stiffness exceeds 10% 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.*

4.2.2 *Stiffness for analysis*

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

1. For $\Sigma M_{nc}/\Sigma M_{nb} > 1.2$, column offsets are rigid and beam offsets are not;
2. For $\Sigma M_{nc}/\Sigma M_{nb} < 0.8$, beam offsets are rigid and column offsets are not; and
3. For $0.8 \leq \Sigma M_{nc}/\Sigma M_{nb} \leq 1.2$, half of the beam and column offsets are considered rigid (Fig. 4.1).

M_{nc} should be calculated considering axial force from design gravity loads. As this modeling approach accounts only for joint shear flexibility, stiffness values used for the beams and columns should include the flexibility resulting from bar slip.

Table 4.1—Modeling parameters and numerical acceptance criteria for nonlinear procedures—reinforced concrete beams

Conditions			Modeling parameters*			Acceptance criteria*†				
						Plastic rotations angle, radians				
						Performance level				
			Plastic rotations angle, radians		Residual strength ratio		Component type			
							Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
Condition i. Beams controlled by flexure‡										
$\frac{\rho - \rho'}{\rho_{bal}}$	Transverse reinforcement†	$\frac{V}{b_w d \sqrt{f'_c}}$ ‖								
≤ 0.0	C	≤ 3 (0.25)‖	0.025	0.05	0.2	0.010	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6 (0.5)‖	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3 (0.25)‖	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6 (0.5)‖	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3 (0.25)‖	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6 (0.5)‖	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3 (0.25)‖	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6 (0.5)‖	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01
Condition ii. Beams controlled by shear‡										
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.0020	0.0030	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	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.0020	0.0030	0.005	0.01
Condition iv. Beams controlled by inadequate embedment into beam-column joint‡										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

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

†Primary and secondary component demands should be within secondary component acceptance criteria where the full backbone curve is explicitly modeled, including strength degradation and residual strength, in accordance with Section 3.4.3.2 of ASCE/SEI 41-06.

‡Where more than one of the 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_s) is at least 3/4 of the design shear. Otherwise, the transverse reinforcement is considered nonconforming.

‖V is the design shear force from NSP or NDP.

Note: f'_c in psi (MPa) units.

C4.2.2.1 Linear static and dynamic procedures—*Various approaches to explicitly model beam-column joints are available in the literature (El-Metwally and Chen 1988; Ghobarah and Biddah 1999; Shin and LaFave 2004; Mitra and Lowes 2007). For simplicity, implementation in commercial structural analysis software and agreement with calibration studies performed in the development of ACI 369R, this section defines an implicit beam-column joint modeling technique using centerline models with semi-rigid joint offsets. Figure 4.1 shows an example of an explicit joint model and illustrates the implicit joint modeling approach. In the implicit joint model, only a portion of the beam and column, or both, within the geometric joint region is defined as rigid. In typical commercial software packages, this portion can range from 0, in which case the model is a true centerline model, to 1.0, where the entire joint region is rigid. Further commentary is provided in Section C3.1.2.1 and background material is provided in Elwood et al. (2007) and Birely et al. (2009).*

4.2.2.2 *Nonlinear static procedure*—Nonlinear load-deformation relations should comply with Section 3.1.2. Nonlinear modeling parameters for beams, columns, and beam-column joints are provided in Tables 4.1, 4.2, and 4.4, respectively.

Beams and columns should be modeled using concentrated or distributed plastic hinge models. Other models whose behavior represents the behavior of reinforced concrete beam and column components subjected to lateral loading are acceptable. The beam and column model should be capable of representing inelastic response along the component length, except where it is shown by equilibrium that yielding is restricted to the component ends. Where nonlinear response is expected in a mode other than flexure, the model should be established to represent such effects.

Monotonic load-deformation relations coincide with the generalized load-deformation relation shown in Fig. 3.1, with the exception that different relations are acceptable where verified by experiments. The overall load-deformation relation should be established so that maximum resistance is consistent with the design strength specifications of Sections 3.2 and 4.2.3.

For beams and columns, the generalized deformation in Fig. 3.1 is plastic hinge rotation. For beam-column joints, the

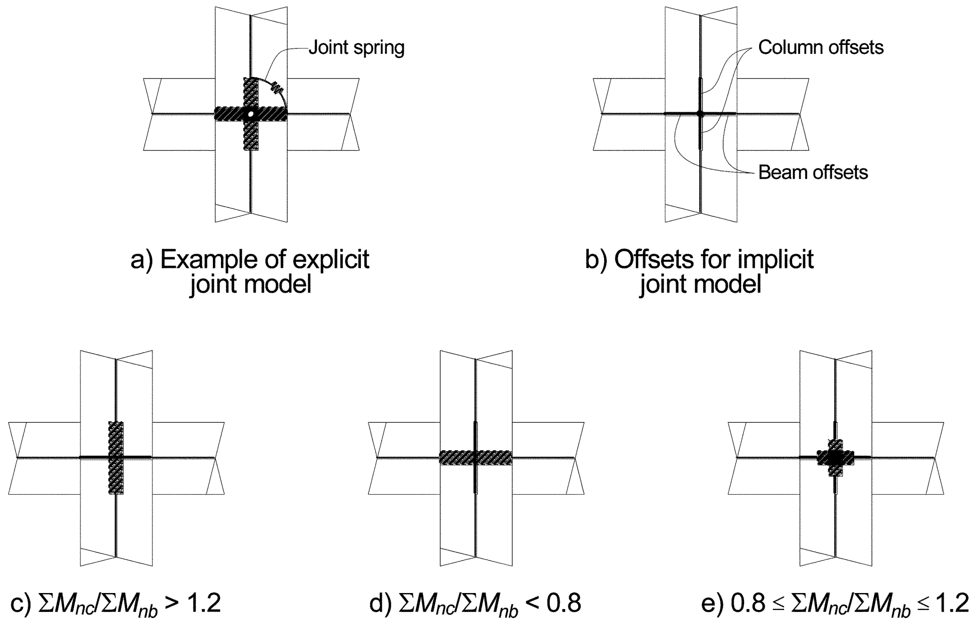


Fig. 4.1—Beam-column joint modeling (note: hatched portions indicate rigid element).

generalized deformation is shear strain. Values of the generalized deformation at points B, C, and D should be derived from experiments or rational analyses, and should take into account the interactions between flexure, axial load, and shear.

Columns not controlled by inadequate splices (Condition i, ii, or iii in Table 4.2) are classified based on V_o (Eq. (4-1), Section 4.2.3.1, using expected material properties), the plastic shear demand on the column (V_p is shear demand at flexural yielding of plastic hinges), and the transverse reinforcement detailing, as shown in Table 4.3.

C4.2.2.2 Nonlinear static procedure—*The modeling parameters and acceptance criteria specified in Table 4.2 reflect results from recent research on reinforced concrete columns. Refer to Elwood et al. (2007) for a detailed description of the derivation of this table. Section 4.2.2.2 and Table 4.3 provide the criteria to determine which condition in Table 4.2 should be used to select the modeling parameters and acceptance criteria. For columns with transverse reinforcement including 135-degree hooks, the specified conditions approximately correspond to the following failure modes:*

- *Condition i: Flexure failure;*
- *Condition ii: Flexure-shear failure, where yielding in flexure is expected before shear failure; and*
- *Condition iii: Shear failure.*

For $V_p/V_o \geq 0.6$, the condition is adjusted from Condition i to ii for columns with 90-degree hooks or lap-spliced transverse reinforcement to reflect the observation from experiments that poor transverse reinforcement details can result in decreased deformation capacity. For $1.0 \geq V_p/V_o > 0.6$, the condition is adjusted from Condition ii to iii only for lap-spliced transverse reinforcement since the database used to evaluate the parameters for Condition ii includes columns with transverse reinforcement having 90-degree hooks. The classification of columns based on V_p/V_o as described in Section 4.2.2.2 may conservatively classify some columns with $V_p/V_o \approx 1.0$ as shear failures, although some flexural yielding may occur before shear failure. Likewise, columns with $0.6 < V_p/V_o \leq 0.7$ may in fact experience flexural failures (without shear degradation) but have been conservatively classified in Section 4.2.2.2 as flexure-shear failures to ensure columns are not erroneously classified in a better performing category. Experimental evidence may be used to determine the expected failure mode and select the appropriate modeling parameters.

The acceptance criteria in Table 4.2 are determined based on the modeling parameters "a" and "b" and the requirements of ASCE/SEI 41-06 Chapter 2. The modeling parameters in Table 4.2 define the plastic rotations according to Fig. 3.1. As shown in Fig. 3.1, modeling parameter "a" provides the plastic rotation at significant loss of lateral load capacity. For the purposes of determining "a" values based on test data, it was assumed that this point represented a 20% or greater reduction in the lateral load resistance from the measured peak shear capacity. For columns expected to experience flexural failures (Condition i), such loss of lateral load resistance can be caused by concrete crushing, bar buckling, and other flexural damage mechanisms. For columns expected to experience shear failures, either before or after flexural yielding (Conditions ii or iii), loss of lateral load resistance is commonly caused by severe diagonal cracking indicative of shear damage. Consistent with ASCE/SEI 41-06 Section 2.4.4.3, modeling parameter "b" provides an estimate of the plastic rotation at the loss of gravity load support, that is, axial load failure. Experimental evidence suggests that axial load failure can occur suddenly after lateral load failure for columns with axial loads above $0.6A_g f_c'$ (Sezen and Moehle 2006; Bayrak and Sheikh 1997). Based on this observation, the "a" and "b" parameters in Table 4.2 converge to a single value for high axial loads.

Table 4.2—Modeling parameters an numerical acceptance criteria for nonlinear procedures—reinforced concrete columns

Conditions			Modeling parameters*			Acceptance criteria*†				
						Plastic rotations angle, radians				
						Performance level				
							Component type			
			Plastic rotations angle, radians		Residual strength ratio		Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
Condition i.‡										
$\frac{P}{A_g f'_c}$ §	$\rho = \frac{A_v}{b_w s}$									
≤ 0.1	≥ 0.006		0.035	0.060	0.2	0.005	0.026	0.035	0.045	0.060
≥ 0.6	≥ 0.006		0.010	0.010	0.0	0.003	0.008	0.009	0.009	0.010
≤ 0.1	= 0.002		0.027	0.034	0.2	0.005	0.020	0.027	0.027	0.034
≥ 0.6	= 0.002		0.005	0.005	0.0	0.002	0.003	0.004	0.004	0.005
Condition ii.‡										
$\frac{P}{A_g f'_c}$ §	$\rho = \frac{A_v}{b_w s}$	$\frac{V}{b_w d \sqrt{f'_c}}$ ‖								
≤ 0.1	≥ 0.006	≤ 3 (0.25)‖	0.032	0.060	0.2	0.005	0.024	0.032	0.045	0.060
≤ 0.1	≥ 0.006	≥ 6 (0.5)‖	0.025	0.060	0.2	0.005	0.019	0.025	0.045	0.060
≥ 0.6	≥ 0.006	≤ 3 (0.25)‖	0.010	0.010	0.2	0.003	0.008	0.009	0.009	0.010
≥ 0.6	≥ 0.006	≥ 6 (0.5)‖	0.008	0.008	0.2	0.003	0.006	0.007	0.007	0.008
≤ 0.1	≤ 0.0005	≤ 3 (0.25)‖	0.012	0.012	0.0	0.005	0.009	0.010	0.010	0.012
≤ 0.1	≤ 0.0005	≥ 6 (0.5)‖	0.006	0.006	0.0	0.004	0.005	0.005	0.005	0.006
≥ 0.6	≤ 0.0005	≤ 3 (0.25)‖	0.004	0.004	0.0	0.002	0.003	0.003	0.003	0.004
≥ 0.6	≤ 0.0005	≥ 6 (0.5)‖	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Condition iii.‡										
$\frac{P}{A_g f'_c}$ §	$\rho = \frac{A_v}{b_w s}$									
≤ 0.1	≥ 0.006		0.0	0.060	0.0	0.0	0.0	0.0	0.045	0.060
≥ 0.6	≥ 0.006		0.0	0.008	0.0	0.0	0.0	0.0	0.007	0.008
≤ 0.1	≤ 0.0005		0.0	0.006	0.0	0.0	0.0	0.0	0.005	0.006
≥ 0.6	≤ 0.0005		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Condition iv. Columns controlled by inadequate development or splicing along the clear height‡										
$\frac{P}{A_g f'_c}$ §	$\rho = \frac{A_v}{b_w s}$									
≤ 0.1	≥ 0.006		0.0	0.060	0.4	0.0	0.0	0.0	0.045	0.060
≥ 0.6	≥ 0.006		0.0	0.008	0.4	0.0	0.0	0.0	0.007	0.008
≤ 0.1	≤ 0.0005		0.0	0.006	0.2	0.0	0.0	0.0	0.005	0.006
≥ 0.6	≤ 0.0005		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

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

†Primary and secondary component demands should be within secondary component acceptance criteria where the full backbone curve is explicitly modeled, including strength degradation and residual strength, in accordance with Section 3.4.3.2 of ASCE/SEI 41-06.

‡Refer to Section 4.2.2.2 for definition of Conditions i, ii, and iii. 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. (3-2). Where more than one of the Conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.

§Where $P > 0.7A_g f'_c$, the plastic rotation angles should be taken as zero for all performance levels unless the column has transverse reinforcement consisting of hoops with 135-degree hooks spaced at $\leq d/3$ and the strength provided by the hoops (V_s) is at least 3/4 of the design shear. Axial load P should be based on the maximum expected axial loads due to gravity and earthquake loads.

‖V is the design shear force from NSP or NDP.

Note: f'_c is in psi (MPa) units.

Table 4.3—Transverse reinforcement details: Condition to be used for columns in Table 4.2

	ACI 318/318M conforming seismic details with 135-degree hooks	Closed hoops with 90-degree hooks	Other (including lap-spliced transverse reinforcement)
$V_p/V_o \leq 0.6$	i*	ii	ii
$1.0 \geq V_p/V_o > 0.6$	ii	ii	iii
$V_p/V_o > 1.0$	iii	iii	iii

*To qualify for Condition i, a column should have $A_v/b_w s \geq 0.002$ and $s/d \leq 0.5$ within flexural plastic hinge region. Otherwise, the column is assigned to Condition ii.

For an appropriate estimate of the deformation capacities, interpolation between the values given in Table 4.2 is required. For Condition ii, the interpolation is performed on three variables in any order.

Considerable scatter exists in results from reinforced concrete columns tested to lateral load and axial load failure, making it inappropriate to specify median or mean values for the plastic rotations in Table 4.2. The goal in selecting values for parameter "a" given in Table 4.2 was to achieve a high level of safety—probability of failure P_f *less*

Table 4.4—Modeling parameters and numerical acceptance criteria for nonlinear procedures—reinforced concrete beam-column joints

Conditions			Modeling parameters*			Acceptance criteria*†				
						Plastic rotations angle, radians				
						Performance level				
			Plastic rotations angle, radians		Residual strength ratio		Component type			
							Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
Condition i. Interior joints (Note: for classification of joints, refer to Fig. 4.2)										
$\frac{P}{A_g f_c'}$ ‡	Transverse reinforcement§	$\frac{V}{V_n}$ ‖								
≤ 0.1	C	≤ 1.2	0.015	0.03	0.2	0.0	0.0	0.0	0.02	0.03
≤ 0.1	C	≥ 1.5	0.015	0.03	0.2	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≤ 1.2	0.015	0.025	0.2	0.0	0.0	0.0	0.015	0.025
≥ 0.4	C	≥ 1.5	0.015	0.2	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≤ 1.2	0.005	0.2	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≤ 1.2	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
Condition ii. Other joints (Note: for classification for joints, refer to Fig. 4.2)										
$\frac{P}{A_g f_c'}$ ‡	Transverse reinforcement§	$\frac{V}{V_n}$ ‖								
≤ 0.1	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≤ 0.1	NC	≤ 1.2	0.005	0.01	0.2	0.0	0.0	0.0	0.0075	0.01
≤ 0.1	NC	≥ 1.5	0.005	0.01	0.2	0.0	0.0	0.0	0.0075	0.01
≥ 0.4	NC	≤ 1.2	0.0	0.0075	0.0	0.0	0.0	0.0	0.005	0.0075
≥ 0.4	NC	≥ 1.5	0.0	0.0075	0.0	0.0	0.0	0.0	0.005	0.0075

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

†Primary and secondary component demands should be within secondary component acceptance criteria where the full backbone curve is explicitly modeled, including strength degradation and residual strength, in accordance with Section 3.4.3.2 of ASCE/SEI 41-06.

‡P is the design axial force on the column above the joint calculated using limit-state analysis procedures in accordance with Section 4.2.4, and A_g is the gross cross-sectional area of the joint.

§"C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. Joint transverse reinforcement is conforming if hoops are spaced at ≤ $h_c/2$ within the joint. Otherwise, the transverse reinforcement is considered nonconforming.

‖V is the design shear force from NSP or NDP, and V_n is the shear strength for the joint. The shear strength should be calculated according to Section 4.2.3.

than 15%—for columns that may experience shear failures, while allowing a slightly lower level of safety, P_f < 35%, for columns expected to experience flexural failures. Given the potential of collapse resulting from axial load failure of individual columns, a high level of safety, P_f < 15%, was desired for parameter "b." Target limits for probabilities of failure were selected based on the judgment of the ASCE/SEI 41 Supplement 1 Ad Hoc Committee (ASCE/SEI 41 Ad Hoc Committee 2007) responsible for the development of Table 4.2.

To assess the degree of safety provided by Table 4.2, the tabulated values were interpolated and compared with data from laboratory tests on reinforced concrete columns appropriate for each of the conditions defined in Section 4.2.2.2. The results are assessed and summarized in Table C4.1. Actual probabilities of failure achieved by the limits in Table 4.2 are considerably lower in many cases than the target probabilities of failure given above. Insufficient data exist to assess the probability of failure for parameter "b" for Conditions i, iii, and iv; limited experimental evidence suggests however, that the drift ratios for such columns will be greater than those for flexure-shear columns (Melek and Wallace 2004; Yoshimura et al. 2004). Therefore, the "b" values for Condition ii are conservatively used for all conditions. Most laboratory tests ignore some factors that may influence the drift capacity, such as loading history; the probabilities of failure in Table C4.1 may therefore be larger if these factors are considered.

The database for modeling parameter "a" for Condition i only considered columns with $A_v/b_w s \geq 0.002$ and $s/d \leq 0.5$; these limitations, therefore, have been placed on the applicability of the modeling parameters for Condition i.

For columns expected to experience shear failure before flexural yielding (Condition iii), the deformation at shear failure is given by the effective stiffness of the component and shear strength of the column (V_o from Eq. (4-1)). Significant plastic deformations cannot be relied on before shear failure; parameter "a," therefore, is set to zero. This assumption is conservative for some columns because the classification method in Section 4.2.2.2 may result in some flexure-shear columns being classified as Condition iii and

Table C4.1—Database results for modeling parameters in Table 4.2

Modeling parameter	No. of tests	Mean($\theta_{p\ meas}/\theta_{p\ table}$)	β($\theta_{p\ meas}/\theta_{p\ table}$)	Probability of failure*
"a" for Condition i	141	1.44	0.50	30%
"a" for Condition ii	31	2.23	0.47	6%
"a" for Condition iii	34	4.66	0.48	0.1%
"b" for Condition ii	28	1.97	0.50	13%

*Assuming a lognormal distribution for ($\theta_{p\ meas}/\theta_{p\ calc}$).

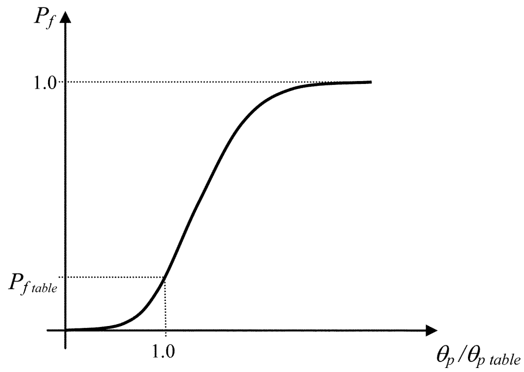


Fig. C4.1—Fragility curve.

most will have some limited plastic rotation capacity before shear failure. Except for columns with high axial loads and very light transverse reinforcement, deformations beyond shear failure are expected before axial load failure.

Elwood and Moehle (2005b) have demonstrated that the drift at axial failure decreases as the nondimensional parameter increases

$$\alpha = \frac{P}{A_v f_{yt} d_c / s} \qquad (C4\text{-}1)$$

The database used to assess the probability of failure for parameter "b" includes columns with $\alpha \leq 33$. Use caution when applying the values from Table 4.2 to columns with $\alpha > 33$.

The probabilities of failure in Table C4.1 were determined by considering ($\theta_{p\ meas}/\theta_{p\ table}$) as a random variable with a lognormal distribution. Equation (C4-2) allows for the determination of the expected plastic rotation for a higher probability of failure, $P_{f\ new}$.

$$\theta_p(P_{f\ new}) = \theta_{p\ table} exp[\zeta[\Phi^{-1}(P_{f\ new}) - \Phi^{-1}(P_{f\ table})] \qquad (C4\text{-}2)$$

where $\zeta = \sqrt{\ln(1+\beta^2)}$, β is the coefficient of variation based on test data given in Table C4.1, $P_{f\ table}$ is the probability of failure given in Table C4.1, and Φ^{-1} is the inverse standard normal cumulative distribution function, with a zero mean and unit standard deviation. The inverse standard normal cumulative distribution function Φ^{-1} is found in basic statistics textbooks and is available as a function in most spreadsheet programs.

Equation (C4-2) can be used to establish the fragility curve (Fig. C4.1) of the column, which provides the probability of failure for a given normalized plastic rotation demand, $\theta_p/\theta_{p\ table}$. Note that P_f is the probability of failure for a column given a plastic rotation demand equal to θ_p. The probability of failure considering the uncertainty in the ground motion is much lower than P_f.

Databases used to assess the model conservatism consist of rectangular columns subjected to unidirectional lateral loads parallel to one face of the column. Actual columns have configurations and loadings that differ from those used in the columns database; additional scatter in results, therefore, may be anticipated. Note that bidirectional loading on corner columns is expected to result in lower drift capacities. Limited data exist, however, to assess the degree of reduction anticipated.

The design professional is referred to the following reports for further guidance regarding determination of modeling parameters and acceptance criteria for reinforced concrete columns: Berry and Eberhard (2005); Elwood and Moehle (2004, 2005a,b); Fardis and Biskinis (2003); Biskinis et al. (2004); Panagiotakos and Fardis (2001); Lynn et al. (1996); and Sezen (2002).

4.2.2.3 *Nonlinear dynamic procedure*—For NDP, the complete hysteretic behavior of each component should be modeled using properties verified by experimental evidence. The use of the generalized load-deformation relation described by Fig. 3.1 to represent the envelope relation for the analysis is acceptable. Refer to Section 4.2.2.2 for the application of parameters for columns in Table 4.2. Unloading and reloading properties should represent significant stiffness and strength-degradation characteristics.

4.2.3 *Strength*—Component strengths should be computed according to the general requirements of Section 3.2, as modified in this section.

The maximum component strength should be determined considering potential failure in flexure, axial load, shear, torsion, development, and other actions at all points along the length of the component, under the actions of design gravity and earthquake load combinations.

4.2.3.1 *Columns*—For columns, the shear strength V_n can be calculated using Eq. (4-1).

$$V_n = kV_o = k\left(\frac{A_v f_y d}{s} + \lambda\left(\frac{6\sqrt{f'_c}}{M/Vd}\sqrt{1+\frac{N_u}{6\sqrt{f'_c}A_g}}\right)0.8A_g\right) \text{ (psi units)}$$

(4-1)

$$V_n = kV_o = k\left(\frac{A_v f_y d}{s} + \lambda\left(\frac{0.5\sqrt{f'_c}}{M/Vd}\sqrt{1+\frac{N_u}{0.5\sqrt{f'_c}A_g}}\right)0.8A_g\right) \text{ (MPa units)}$$

in which k = 1.0 in regions where displacement ductility demand is less than or equal to 2, 0.7 in regions where displacement ductility is greater than or equal to 6, and varies linearly for displacement ductility between 2 and 6; $\lambda = 0.75$ for lightweight aggregate concrete and 1.0 for normalweight aggregate concrete; N_u is the axial compression force (set to zero for tension force); M/Vd is the largest ratio of moment to shear times effective depth under design loadings for the column, but should not be taken greater than 4 or less than 2; d is the effective depth; and A_g is the gross cross-sectional area of the column. It is acceptable to assume $d = 0.8h$, where h is the dimension of the column in the direction of shear. Where axial force is calculated from the linear procedures of ASCE/SEI 41-06 Chapter 3, the maximum compressive axial load in Eq. (4-1) should be taken as equal to the value calculated using ASCE/SEI 41-06 Eq. (3-3) considering design gravity load only and the minimum compression axial load calculated using ASCE/SEI 41-06 Eq. (3-18). Alternatively, the limit analysis specified in ASCE/SEI 41-06 Section 3.4.2.1.2 can be used to determine design axial loads with the linear analysis procedures of ASCE/SEI 41-06 Chapter 3. Alternative formulations for column strength that consider effects of reversed cyclic, inelastic deformations and that are verified by experimental evidence are acceptable.

For columns satisfying the detailing and proportioning requirements of ACI 318/318M, Chapter 21, the shear strength equations of ACI 318/318M can be used.

C4.2.3.1 Columns—*For the assessment of columns in Section 4.2.2.2 of this guide, it is unnecessary to determine k and the displacement ductility demand.*

As discussed in Section C3.3, experimental evidence indicates 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. Caution should be exercised when flexural deformation capacities are determined by calculation.

Equation (4-1) illustrates the reduction in column shear capacity with increasing nonlinear deformations and provides an estimate of the mean observed shear strength for 51 rectangular reinforced concrete columns subjected to unidirectional lateral loads parallel to one face of the column (Sezen and Moehle 2004). The coefficient of variation for the ratio of measured to calculated shear strength is 0.15.

For a column experiencing flexural yielding before shear failure ($V_p < V_o$), displacement ductility demand is defined as the ratio of maximum displacement demand to yield displacement. The yield displacement is the lateral displacement of the column, determined using the effective rigidities from Table 3.1, at a shear demand resulting in flexural yielding of the plastic hinges, V_p. The maximum displacement demand for the column can be estimated as the maximum interstory displacement demand. Alternatively, the interstory displacement demand can be refined by accounting for the interstory displacements caused by rigid-body rotations at the column's base and top. Further discussion on displacement ductility demand is found in Sezen and Moehle (2004). Equation (4-1) should not be used to determine displacement ductility (Elwood and Moehle 2005a).

The design professional is referred to Earthquake Engineering Research Institute/Pacific Earthquake Engineering Research (EERI/PEER) (2006) for a comparison of test data with several column shear strength equations.

4.2.3.2 *Beam-column joints*—For beam-column joints, the nominal cross-sectional area A_j is defined by a joint depth equal to the column dimension in the direction of framing and a joint width equal to the smaller of:

1. The column width;
2. The beam width plus the joint depth; and
3. Twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side.

Design forces should be calculated based on development of flexural plastic hinges in adjacent framing members, including effective slab width, but need not exceed values calculated from design gravity and earthquake-load combinations. Nominal joint shear strength V_n should be calculated using the general procedures of ACI 318/318M, as modified by Eq. (4-2)

$$V_n = \lambda\gamma\sqrt{f_c'}\,A_j \text{ (psi units)}$$

$$\text{(4-2)}$$

$$V_n = 0.083\lambda\gamma\sqrt{f_c'}\,A_j \text{ (MPa units)}$$

in which $\lambda = 0.75$ for lightweight aggregate concrete and 1.0 for normalweight aggregate concrete, A_j is the effective horizontal joint area with dimensions as defined above, and γ is defined in Table 4.5.

4.2.4 *Acceptance criteria*

4.2.4.1 *Linear static and dynamic procedures*—All actions should be classified as being either deformation-controlled or force-controlled, as defined in Section 3.2.1.

Design actions on components should be determined based on ASCE/SEI 41-06 Chapter 3. Where the calculated DCR values exceed unity, the following design actions should be determined using the limit analysis principles in ASCE/SEI 41-06 Chapter 3:

1. Moments, shears, torsions, and development and splice actions corresponding to development of component strength in beams and columns;
2. Joint shears corresponding to strength development in adjacent beams and columns; and
3. Axial load in columns and joints, considering likely plastic action in components above the level in question.

Design actions should be compared with design strengths in accordance with ASCE/SEI 41-06 Section 3.4.2.2, with the m-factors selected from Tables 4.6, 4.7, and 4.8 for beams, columns, and beam-column joints, respectively. Components satisfying ASCE/SEI 41-06 Eq. (3-20) or (3-21), as applicable, should comply with the performance criteria.

Where the average DCR for columns at a level exceeds the average value for beams at the same level, and exceeds the greater of 1.0 and $m/2$ for all columns, at all levels, the level

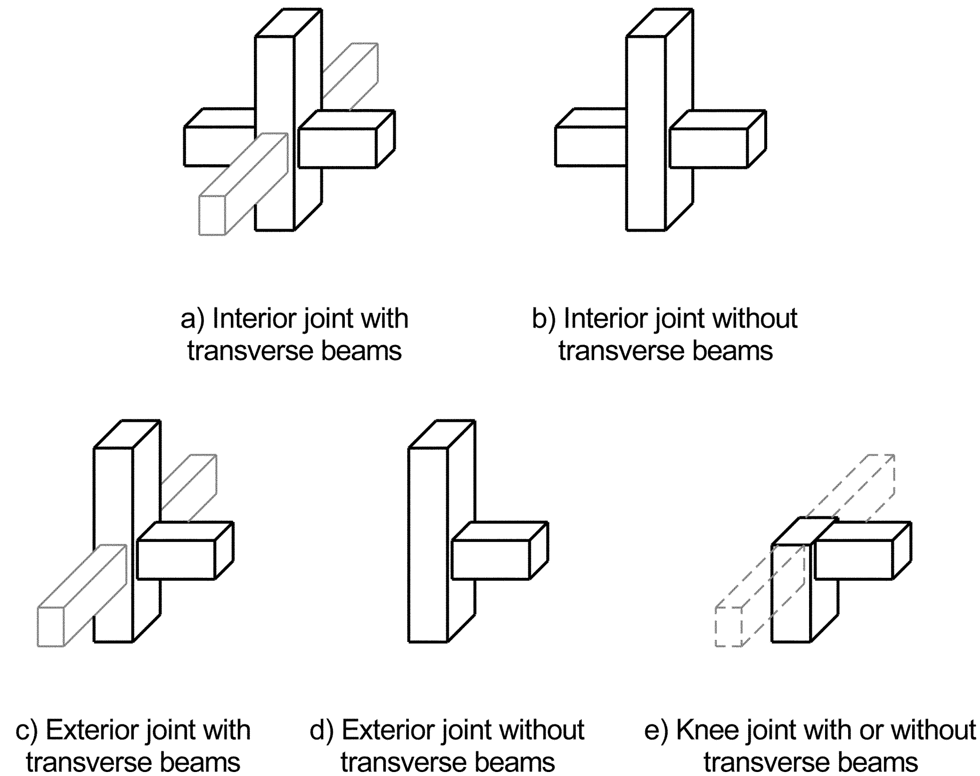


Fig. 4.2—Joint classification (for response in the plane of the page).

Table 4.5—Values of γ for joint strength calculation

Transverse reinforcement†	Value of γ				
	Condition i: interior joints*		Condition ii: other joints		
	Interior joint with transverse beams	Interior joint without transverse beams	Exterior joint with transverse beams	Exterior joint without transverse beams	Knee joint with or without transverse beams
C	20	15	15	12	8
NC	12	10	8	6	4

*For classification of joints, refer to Fig. 4.2

†"C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. Joint transverse reinforcement is conforming if hoops are spaced at $\leq h_c/2$ within the joint. Otherwise, the transverse reinforcement is considered nonconforming.

is defined as a weak story element. For weak story elements, one of the following should be satisfied:

1. The check of average DCR values at the level should be repeated, considering all primary and secondary components at the level with a weak story element at the level. If the average DCR values for vertical components exceeds the average value for horizontal components at the level, and exceeds 2.0, the structure should be reanalyzed using a nonlinear procedure or rehabilitated to eliminate this deficiency;
2. The structure should be reanalyzed using either the NSP or the NDP of ASCE/SEI 41-06 Chapter 3; or
3. The structure should be rehabilitated to eliminate the weak story element condition.

4.2.4.2 *Nonlinear static and dynamic procedures*—Calculated component actions should satisfy the requirements of ASCE/SEI 41-06 Section 3.4.3.2. Where the generalized deformation is taken as rotation in the flexural plastic hinge zone in beams and columns, the plastic hinge rotation capacities are defined by Tables 4.1 and 4.2. Where the generalized deformation is shear distortion of the beam-column joint, shear angle capacities are defined by Table 4.4. Where inelastic action is indicated for a component or action not listed in Tables 4.1, 4.2, and 4.4, the performance is deemed unacceptable. Alternative approaches or values are acceptable where justified by experimental evidence and analysis.

C4.2.4.2 Nonlinear static and dynamic procedures—*Refer to C4.2.2.2 and C4.2.3.1 for discussion of Table 4.2 and acceptance criteria for reinforced concrete columns.*

4.2.5 *Rehabilitation measures*—Concrete beam-column moment frame components that do not meet the acceptance criteria for the selected RO should be rehabilitated. Rehabilitation measures should meet the requirements of Section 3.7 and other provisions of ACI 369R.

C4.2.5 Rehabilitation measures—*Chapter 12 of FEMA 547 provides detailed descriptions of effective rehabilitation measures for use with concrete moment frames, including considerations such as constructibility, disruption for building occupants, and costs.*

Table 4.6—Numerical acceptance criteria for linear procedures—reinforced concrete beams

Conditions			*m*-factors*				
			Performance level				
				Component type			
				Primary		Secondary	
			IO	LS	CP	LS	CP
Condition i. Beams controlled by flexure†							
$\frac{\rho-\rho'}{\rho_{bal}}$	Transverse reinforcement‡	$\frac{V}{b_w d\sqrt{f'_c}}$ §					
≤ 0.0	C	≤ 3 (0.25)§	3	6	7	6	10
≤ 0.0	C	≥ 6 (0.5)§	2	3	4	3	5
≥ 0.5	C	≤ 3 (0.25)§	2	3	4	3	5
≥ 0.5	C	≥ 6 (0.5)§	2	2	3	2	4
≤ 0.0	NC	≤ 3 (0.25)§	2	3	4	3	5
≤ 0.0	NC	≥ 6 (0.5)§	1.25	2	3	2	4
≥ 0.5	NC	≤ 3 (0.25)§	2	3	3	3	4
≥ 0.5	NC	≥ 6 (0.5)§	1.25	2	2	2	3
Condition ii. Beams controlled by shear†							
Stirrup spacing ≤ $d/2$			1.25	1.5	1.75	3	4
Stirrup spacing > $d/2$			1.25	1.5	1.75	2	3
Condition iii. Beams controlled by inadequate development or splicing along the span†							
Stirrup spacing ≤ $d/2$			1.25	1.5	1.75	3	4
Stirrup spacing > $d/2$			1.25	1.5	1.75	2	3
Condition iv. Beams controlled by inadequate embedment into beam-column joint†							
			2	2	3	3	4

*Values between those listed in the table should be determined by linear interpolation.
†Where more than one of the Conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
‡"C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. 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_s) is at least 3/4 of the design shear. Otherwise, the transverse reinforcement is considered nonconforming.
§V is the design shear force calculated using limit-state analysis procedures in accordance with Section 4.2.4.1.
Note: f'_c in psi (MPa) units.

Rehabilitation measures that can be effective in rehabilitating reinforced concrete beam-column moment frames are:

1. *Jacketing existing beams, columns, or joints with new reinforced concrete, steel, or fiber-reinforced polymer wrap overlays. Where reinforced concrete jackets are used, the design should provide detailing to enhance ductility. Component strength should not exceed any limiting strength of connections with adjacent components. Jackets should be designed to provide increased connection strength and improved continuity between adjacent components (FEMA 547 Sections 12.4.4, 12.4.5, and 12.4.6);*
2. *Post-tensioning existing beams, columns, or joints using external post-tensioning reinforcement. Post-tensioned reinforcement should be unbonded within a distance equal to twice the effective depth from sections where inelastic action is expected. Anchorages should be located away from regions where inelastic action is anticipated and designed with consideration of possible force variations from earthquake loading;*
3. *Modifying the element by selective material removal from existing element. Examples include: a) Where nonstructural components interfere with the frame eliminate this interference by removing or separating the nonstructural component from the frame; b) Weakening from concrete removal or severing longitudinal reinforcement to change the response from a nonductile to a more ductile mode; for example, weakening beams to promote formation of a strong-column, weak-beam system; and c) segmenting walls to change stiffness and strength;*
4. *Improving deficient existing reinforcement details. Removal of cover concrete to modify existing reinforcement details should avoid damage to core concrete and the bond between existing reinforcement and core concrete. New cover concrete should be designed and constructed to achieve fully composite action with the existing materials (FEMA 547 Sections 12.4.4, 12.4.5, and 12.4.6);*
5. *Changing the building system to reduce demands on the existing elements. Examples include addition of supplementary lateral force-resisting elements such as walls or buttresses, seismic isolation, and mass reduction (FEMA 547 Chapter 24); and*

Table 4.7—Numerical acceptance criteria for linear procedures—reinforced concrete columns

Conditions			*m*-factors*				
			Performance level				
				Component type			
				Primary		Secondary	
			IO	LS	CP	LS	CP
Condition i†							
$\frac{P}{A_g f_c'}$ ‡	$\rho = \frac{A_v}{b_w s}$						
≤ 0.1	≥ 0.006		2	2.5	3	4	5
≥ 0.6	≥ 0.006		1.25	1.8	1.9	1.9	2
≤ 0.1	≤ 0.002		2	2	2.6	2.6	3
≥ 0.6	≤ 0.002		1.1	1.1	1.2	1.2	1.4
Condition ii†							
$\frac{P}{A_g f_c'}$ ‡	$\rho = \frac{A_v}{b_w s}$	$\frac{V}{b_w d \sqrt{f_c'}}$ §					
≤ 0.1	≥ 0.006	≤ 3 (0.25)§	2	2.5	3	4	5
≤ 0.1	≥ 0.006	≥ 6 (0.5)§	2	2	2.5	4	5
≥ 0.6	≥ 0.006	≤ 3 (0.25)§	1.25	1.8	1.9	1.9	2
≥ 0.6	≥ 0.006	≥ 6 (0.5)§	1.25	1.5	1.6	1.6	1.8
≤ 0.1	≤ 0.0005	≤ 3 (0.25)§	1.2	1.3	1.4	1.4	1.6
≤ 0.1	≤ 0.0005	≥ 6 (0.5)§	1	1	1.1	1.1	1.2
≥ 0.6	≤ 0.0005	≤ 3 (0.25)§	1	1	1.1	1.1	1.2
≥ 0.6	≤ 0.0005	≥ 6 (0.5)§	1	1	1	1	1
Condition iii†							
$\frac{P}{A_g f_c'}$ ‡	$\rho = \frac{A_v}{b_w s}$						
≤ 0.1	≥ 0.006		1	1	1	4	5
≥ 0.6	≥ 0.006		1	1	1	1.6	1.8
≤ 0.1	≤ 0.002		1	1	1	1.1	1.2
≥ 0.6	≤ 0.002		1	1	1	1	1
Condition iv. Columns controlled by inadequate development or splicing along the clear height†							
$\frac{P}{A_g f_c'}$ ‡	$\rho = \frac{A_v}{b_w s}$						
≤ 0.1	≥ 0.006		1	1	1	4	5
≥ 0.6	≥ 0.006		1	1	1	1.6	1.8
≤ 0.1	≤ 0.002		1	1	1	1.1	1.2
≥ 0.6	≤ 0.002		1	1	1	1	1

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

†Refer to Section 4.2.2.2 for definition of Conditions i, ii, and iii. 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. (3-2). Where more than one of the Conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.

‡Where $P > 0.7A_g f_c'$, the *m*-factor should be taken as unity for all performance levels unless the column has transverse reinforcement consisting of hoops with 135-degree hooks spaced at ≤ $d/3$ and the strength provided by the hoops (V_s) is at least 3/4 of the design shear. P is the design axial force in the member. Alternatively, axial loads determined based on a limit-state analysis can be used.

§V is the design shear force calculated using limit-state analysis procedures in accordance with Section 4.2.4.1.

Note: f_c' is in psi (MPa) units.

6. Changing the frame element to a shear wall, infilled frame, or braced frame element by adding new material. Connections between new and existing materials should be designed to transfer the anticipated forces based on the design-load combinations. Where the existing concrete frame columns and beams act as boundary components and collectors for the new shear wall or braced frame, these should be checked for adequacy, considering strength, reinforcement development, and deformability. Diaphragms, including ties and collectors, should be evaluated and if necessary, rehabilitated to ensure a complete load path to the new shear wall or braced frame element (FEMA 547 Sections 12.4.1 and 12.4.2).

Table 4.8—Numerical acceptance criteria for linear procedures—reinforced concrete beam-column joints

Conditions			*m*-factors*				
			Performance level				
				Component type			
				Primary		Secondary	
			IO	LS	CP	LS	CP
Condition i. Interior joints (Note: for classification of joints, refer to Fig. 4.2)							
$\frac{P}{A_g f'_c}$ †	Transverse reinforcement§	$\frac{V}{V_n}$ ‡					
≤ 0.1	C	≤ 1.2	1	1	1	3	4
≤ 0.1	C	≥ 1.5	1	1	1	2	3
≥ 0.4	C	≤ 1.2	1	1	1	3	4
≥ 0.4	C	≥ 1.5	1	1	1	2	3
≤ 0.1	NC	≤ 1.2	1	1	1	2	3
≤ 0.1	NC	≥ 1.5	1	1	1	2	3
≥ 0.4	NC	≤ 1.2	1	1	1	2	3
≥ 0.4	NC	≥ 1.5	1	1	1	2	3
Condition ii. Other joints (Note: for classification of joints, refer to Fig. 4.2)							
$\frac{P}{A_g f'_c}$ †	Transverse reinforcement§	$\frac{V}{V_n}$ ‡					
≤ 0.1	C	≤ 1.2	1	1	1	3	4
≤ 0.1	C	≥ 1.5	1	1	1	2	3
≥ 0.4	C	≤ 1.2	1	1	1	3	4
≥ 0.4	C	≥ 1.5	1	1	1	2	3
≤ 0.1	NC	≤ 1.2	1	1	1	2	3
≤ 0.1	NC	≥ 1.5	1	1	1	2	3
≥ 0.4	NC	≤ 1.2	1	1	1	1.5	2
≥ 0.4	NC	≥ 1.5	1	1	1	1.5	2

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

†P is the design axial force on the column above the joint calculated using limit-state analysis procedures in accordance with Section 4.2.4. A_g is the gross cross-sectional area of the joint.

‡V is the design shear force and V_n is the shear strength for the joint. The design shear force and shear strength should be calculated according to Section 4.2.4.1 and Section 4.2.3, respectively.

§"C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement, respectively. Transverse reinforcement is conforming if hoops are spaced at ≤ $h_c/2$ within the joint. Otherwise, the transverse reinforcement is considered nonconforming.

4.3—Post-tensioned concrete beam-column moment frames

4.3.1 *General considerations*—The analytical model for a post-tensioned concrete beam-column frame element should be established as specified in Section 4.2.1 for reinforced concrete beam-column moment frames. In addition to potential failure modes described in Section 4.2.1, the analysis model should consider potential failure of tendon anchorages.

The analysis procedures described in ASCE/SEI 41-06 Chapter 3 apply to frames with post-tensioned beams satisfying the following conditions:

1. The average prestress f_{pc} calculated for an area equal to the product of the shortest and the perpendicular cross-sectional dimensions of the beam does not exceed the greater of 750 psi (5 MPa) or $f'_c/12$ at locations of nonlinear action;
2. Prestressing tendons do not provide more than one-quarter of the strength at the joint face for both positive and negative moments; and
3. Anchorages for tendons are demonstrated to have performed satisfactorily for seismic forces in compliance with ACI 318/318M requirements. These anchorages should occur outside hinging areas or joints, except in existing components where experimental evidence demonstrates the connection will meet the performance objectives under design loadings.

Alternative procedures should be used where these conditions are not satisfied.

4.3.2 *Stiffness*

4.3.2.1 *Linear static and dynamic procedures*—Beams should be modeled considering flexural and shear stiffnesses, including the effect of the slab acting as a flange in monolithic and composite construction. Columns should be modeled considering flexural, shear, and axial stiffnesses. Refer to Section 3.1.2 for effective stiffness computations. Refer to Section 4.2.2.1 for modeling of joint stiffness.

4.3.2.2 *Nonlinear static procedure*—Nonlinear load-deformation relations should comply with Section 3.1.2 and reinforced concrete frame requirements to Section 4.2.2.2.

Values of the generalized deformation at points B, C, and D in Fig. 3.1 should be derived either from experiments or from approved rational analyses, considering the interactions between flexure, axial load, and shear. Alternatively, where the generalized deformation is taken as rotation in the flexural plastic hinge zone and the three conditions of Section 4.3.1 are satisfied, beam plastic hinge rotation capacities can be as defined in Table 4.1. Columns and joints should be modeled as described in Section 4.2.2.

4.3.2.3 *Nonlinear dynamic procedure*—For the NDP, the complete hysteretic behavior of each component should be modeled using properties verified by experimental evidence. The relation of Fig. 3.1 should be taken to represent the envelope relation for the analysis. Unloading and reloading properties should represent significant stiffness and strength degradation characteristics as influenced by prestressing.

4.3.3 *Strength*—Component strengths should be computed according to the general requirements of Section 3.2 and additional requirements of Section 4.2.3. Effects of prestressing on strength should be considered.

For deformation-controlled actions, prestress should be assumed effective to determine the maximum actions that can be developed in association with nonlinear response of the frame. For force-controlled actions, the effects on strength of prestress loss should be considered as a design condition, where such losses are possible under design load combinations including inelastic deformation reversals.

4.3.4 *Acceptance criteria*—Acceptance criteria for post-tensioned concrete beam-column moment frames should follow the criteria for reinforced concrete beam-column frames specified in Section 4.2.4.

Modeling parameters and acceptance criteria should be based on Tables 4.1, 4.2, 4.4, and Tables 4.6 through 4.8.

4.3.5 *Rehabilitation measures*—Post-tensioned concrete beam-column moment frame components that do not meet acceptance criteria for the selected RO should be rehabilitated. Rehabilitation measures should meet the requirements of Section 3.7 and other provisions of this guide.

C4.3.5 Rehabilitation measures—*Rehabilitation measures described in Section C4.2.5 for reinforced concrete beam-column moment frames can be effective in rehabilitating post-tensioned concrete beam-column moment frames. Further rehabilitation measures can be found in FEMA 547.*

4.4—Slab-column moment frames

4.4.1 *General considerations*—The analytical model for a slab-column frame element should represent strength, stiffness, and deformation capacity of slabs, columns, slab-column connections, and other components of the frame. The connection between the columns and foundation should be modeled based on the details of the column-foundation connection and rigidity of the foundation-soil system. Potential failure in flexure, shear, shear-moment transfer (punching shear), and reinforcement development at any section along the component length should be considered. The effects of changes in cross section, slab openings, and interaction with structural and nonstructural components should be considered.

An analytical model of the slab-column frame based on any of the following approaches can be used.

1. Effective beam width model: Columns and slabs are represented by line elements rigidly interconnected at the slab-column connection, where the slab width included in the model is adjusted to account for flexibility of the slab-column connection;
2. Equivalent frame model: Columns and slabs are represented by line elements and stiffness of column or slab elements is adjusted to account for flexibility of the slab-column connection; and
3. Finite element model: Columns are represented by line elements and the slab by plate-bending elements.

C4.4.1 General considerations—*The stiffness of a slab-column frame is highly dependent on the ratio of the column cross section dimensions (c_1 and c_2) to the slab plan dimensions (ℓ_1 and ℓ_2).*

Approaches for modeling slab-column frame systems differ primarily in how slab stiffness is incorporated in the analytical model.

1. *Effective beam width model: Slab element width reduced to adjust the elastic stiffness to more closely match measured values (Pecknold 1975). Column behavior and slab-column moment and shear transfer are modeled separately;*
2. *Equivalent frame model: Shear and flexure in the slab beyond the width of the column are assumed to be transferred to the column through torsional elements perpendicular to the slab span direction (Vanderbilt and Corley 1983). Flexibility of the torsional elements reduces the elastic stiffness of the overall frame. Although it is possible to model them separately, torsional elements are typically lumped with columns or the slab to produce a frame with equivalent stiffness (Chapter 13 ACI 318/318M-08); and*
3. *Finite element model: The slab distortion is modeled explicitly using finite elements.*

Each approach is considered acceptable for analytical modeling of slab-column frames. Research has shown that the effective beam approach tends to overestimate lateral stiffness, whereas the equivalent frame approach tends to underestimate lateral stiffness of slab-column systems responding in the elastic range (Hwang and Moehle 2000). For either approach, the elastic stiffness should be reduced further to account for cracking in slab-column systems responding in the inelastic range (Hwang and Moehle 2000; Luo et al. 1994; and Dovich and Wight 2005).

4.4.2 *Stiffness*

4.4.2.1 *Linear static and dynamic procedures*—Slabs should be modeled considering flexural, shear, and torsional (in the slab adjacent to the column) stiffnesses. Columns should be modeled considering flexural, shear, and axial stiffnesses. Slab-column connections should be modeled as stiff or rigid components. Although effective component stiffnesses should be determined according to the general principles of Section 3.1.2, adjustments can be made based on experimental evidence.

C4.4.2.1 Linear static and dynamic procedures:

1. Effective beam width model: Allen and Darvall (1977) provide tables of effective width coefficients for different combinations of plate aspect ratios (ℓ_1/ℓ_2) and column width-to-slab span ratios (c_1/ℓ_1 or c_2/ℓ_1). Research indicates that the effective width of exterior bays should be less than the *effective width of interior bays due to the higher flexibility of one-sided slab-column connections at the frame end. Hwang and Moehle (2000) provide equations for effective width that show the relationship between exterior and interior bays is about 1/2.*

Equation (C4-3) can be used instead of tables from Allen and Darvall (1977)

For interior bays: $b_{eff} = 2c_1 + \ell_1/3$ *(C4-3)*

For exterior bays: $b_{eff} = c_1 + \ell_1/6$

where b_{eff} *is the effective slab width*

To account for cracking from temperature, shrinkage, or nonlinear response, slab stiffness determined using gross section properties based on the above guidance should be reduced by an effective stiffness factor β_{eff}. *There is general agreement that* $\beta_{eff} = 1/3$ *is appropriate for non-prestressed slabs (Vanderbilt and Corley 1983). Somewhat higher, yet conservative, values can be obtained using Eq. (C4-4) from Hwang and Moehle (2000)*

$$\beta_{eff} = 4c_1/\ell_1 \geq 1/3 \qquad \text{(C4-4)}$$

For prestressed post-tensioned slabs, it is generally agreed that higher values of β_{eff} *are appropriate (*$\beta_{eff} = 1/2$*) because of reduced cracking due to prestressing (Kang and Wallace 2005).*

2. Equivalent frame model: Column, slab-beam, and torsional connection element properties for the equivalent frame model are defined in Chapter 13 of ACI 318/318M-08. To account for cracking due to temperature, shrinkage, or nonlinear response, the stiffness of the torsional connection element based on gross section properties defined in ACI 318/318M should be multiplied by a factor of 1/3.

4.4.2.2 *Nonlinear static procedure*—Nonlinear load-deformation relations should comply with the requirements of Section 3.1.2. Nonlinear modeling parameters for slab-column connections are provided in Table 4.9.

Nonlinear static models should be capable of representing inelastic response along the component length, except where it is shown by equilibrium that yielding is restricted to the component ends.

Idealized load-deformation relations should be modeled using the generalized relation shown in Fig. 3.1. The overall load-deformation relation should be established so that the maximum resistance is consistent with the design strength specifications of Sections 3.2 and 4.4.3. For columns, the generalized deformation shown in Fig. 3.1 is flexural plastic hinge rotation with parameters as defined in Table 4.2. For slabs and slab-column connections, the generalized deformation shown in Fig. 3.1 is plastic rotation with parameters as defined in Table 4.9. Different relations are acceptable where verified by experimentally obtained cyclic response relations of slab-column subassemblies.

C4.4.2.2 Nonlinear static procedure—*The values provided in Table 4.9 are used to assess punching failures at slab-column connections. Elwood et al. (2007) provides a comparison of the modeling parameters in Table 4.9 and test data summarized by Kang and Wallace (2006). Lateral drift ratio is typically reported for test data; therefore, plastic rotations were derived from the test data assuming column deformations were negligible and yield rotations of 0.01 and 0.015 radians for reinforced concrete and post-tensioned slabs, respectively. The larger rotation value for post-tensioned connections reflects the larger span-to-slab thickness ratios common for this type of construction. Continuity reinforcement for reinforced concrete connections is based on Joint ACI-ASCE Committee 352 recommendations (ACI 352R).*

Plastic rotation values are approximately mean and mean less one standard deviation values for connections with and without continuity reinforcement, respectively. Mean less one standard deviation values give total (yield plus plastic) rotation values that are close to the maximum drift values allowed by ACI 318/318M-08 Section 21.13.5, without the use of slab shear reinforcement. Few data exist for reinforced concrete connections subjected to gravity shear ratios greater than 0.6 and for post-tensioned connections subjected to reverse cyclic loading. The residual strength capacity for post-tensioned connections is based on test results reported by Qaisrani (1993). Although relatively few tests have been reported for edge connections, the limited data available suggest the relationship between rotation and gravity shear ratio for exterior connections is similar to the trend for interior connections.

Modeling of slab-column connections is commonly accomplished using beam elements to represent the slab and a rigid-plastic torsional member to represent moment and shear transfer at the connection between slab and column (Fig. C4.2) (Elwood et al. (2007)). If the punching capacity of the slab-column connection is insufficient to develop the nominal capacity for the developed slab flexural reinforcement provided within the column strip, then all yielding is assumed to occur in the torsional element using the modeling parameters provided in Table 4.9. For strong connections where yielding of slab reinforcement within the column strip is expected, plastic rotations should be modeled only within the beam elements framing into the torsional element using the plastic rotation modeling parameters provided in Table 4.9 to define the plastic hinges at the beam ends.

4.4.2.3 *Nonlinear dynamic procedure*—The requirements of Sections 3.2 and 4.2.2.3 for reinforced concrete beam-column moment frames apply to slab-column moment frames.

4.4.3 *Strength*—Component strengths should be computed according to the general requirements of Section 4.2, as modified in this section. For columns, evaluation of shear strength according to Section 4.2.3 can be used.

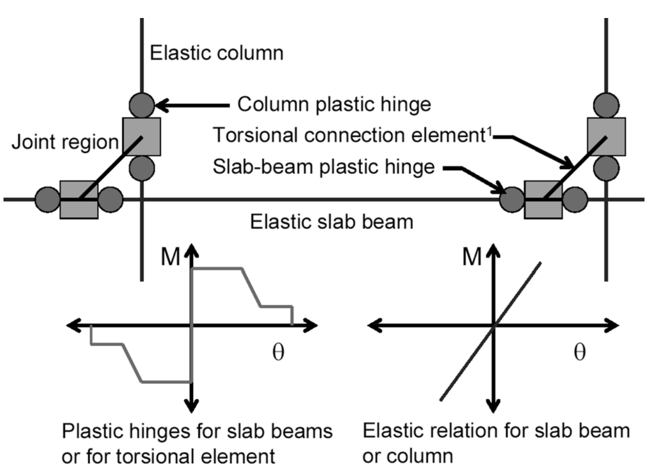
The flexural strength of a slab to resist moment due to lateral deformations should be calculated as $M_{nCS} - M_{gCS}$, where M_{nCS} is the design flexural strength of the column strip and M_{gCS} is the column strip moment due to gravity loads. M_{gCS} should be calculated according to the procedures of ACI 318/318M for the design gravity load specified in ASCE/SEI 41-06 Chapter 3.

Slab-column connections should be investigated for potential failure in shear and moment transfer considering

Table 4.9—Modeling parameters and numerical acceptance criteria for nonlinear procedures—two-way slabs and slab-column connections

Conditions		Modeling parameters[*]: Plastic rotation angle, radians		Modeling parameters[*]: Residual strength ratio	Acceptance criteria[*†]: Plastic rotation angle, radians; Performance level				
						Component type: Primary		Component type: Secondary	
		a	b	c	IO	LS	CP	LS	CP
Condition i. Reinforced concrete slab-column connections[‡]									
$\frac{V_g}{V_o}$ [§]	Continuity reinforcement[‖]								
0	Yes	0.035	0.05	0.2	0.01	0.026	0.035	0.035	0.05
0.2	Yes	0.03	0.04	0.2	0.01	0.023	0.03	0.03	0.04
0.4	Yes	0.02	0.03	0.2	0	0.015	0.02	0.02	0.03
≥ 0.6	Yes	0	0.02	0	0	0	0	0	0.02
0	No	0.025	0.025	0	0.01	0.02	0.02	0.02	0.025
0.2	No	0.02	0.02	0	0.01	0.015	0.015	0.015	0.02
0.4	No	0.01	0.01	0	0	0.008	0.008	0.008	0.01
0.6	No	0	0	0	0	0	0	0	0
> 0.6	No	0	0	0	—[#]	—[#]	—[#]	—[#]	—[#]
Condition ii. Post-tensioned slab-column connections[‡]									
$\frac{V_g}{V_o}$ [§]	Continuity reinforcement[‖]								
0	Yes	0.035	0.05	0.4	0.01	0.026	0.035	0.035	0.05
0.6	Yes	0.005	0.03	0.2	0	0.003	0.005	0.025	0.03
> 0.6	Yes	0	0.02	0.2	0	0	0	0.015	0.02
0	No	0.025	0.025	0	0.01	0.02	0.02	0.02	0.025
0.6	No	0	0	0	0	0	0	0	0
> 0.6	No	0	0	0	—[#]	—[#]	—[#]	—[#]	—[#]
Condition iii. Slabs controlled by inadequate development or splicing along the span[‡]									
		0	0.02	0	0	0	0	0.01	0.02
Condition iv. Slabs controlled by inadequate embedment into slab-column joint[‡]									
		0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

[*]Values between those listed in the table should be determined by linear interpolation.
[†]Primary and secondary component demands should be within secondary component acceptance criteria where the full backbone curve is explicitly modeled, including strength degradation and residual strength, in accordance with Section 3.4.3.2 of ASCE/SEI 41-06.
[‡]Where more than one of the Conditions i, ii, iii, and iv occur for a given component, use the minimum appropriate numerical value from the table.
[§]V_g is the gravity shear acting on the slab critical section as defined by ACI 318/318M, and V_o is the direct punching shear strength as defined by ACI 318/318M.
[‖]"Yes" should be used where the area of effectively continuous main bottom bars passing through the column cage in each direction is greater than or equal to $0.5V_g/(\phi f_y)$. Where the slab is post-tensioned, "Yes" should be used where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, "No" should be used.
[#]Action should be treated as force-controlled.



[1]Slab-beams and columns only connected by rigid-plastic torsional connection element.

Fig. C4.2—Modeling of slab-column connection.

the combined action of flexure, shear, and torsion acting in the slab at the connection with the column.

For interior connections without transverse beams and exterior connections with moment about an axis perpendicular to the slab edge, the shear and moment transfer strength, or the "torsional" element strength, can be calculated as the minimum of:

1. Strength calculated considering eccentricity of shear on a slab critical section due to combined shear and moment (ACI 318/318M); and
2. Moment transfer strength equal to $\Sigma M_n/\gamma_f$, where ΣM_n is the sum of positive and negative flexural strengths of a section of slab between lines that are two and one-half slab or drop panel thicknesses ($2.5h$) outside opposite faces of the column or capital; γ_f is the fraction of the moment resisted by flexure per ACI 318/318M; and h is slab thickness.

Table 4.10—Numerical acceptance criteria for linear procedures—two-way slabs and slab-column connections

Conditions		m-factors*				
		Performance level				
			Component type			
			Primary		Secondary	
		IO	LS	CP	LS	CP
Condition i. Reinforced concrete slab-column connections†						
$\frac{V_g}{V_o}$ ‡	Continuity reinforcement§					
0	Yes	2	2.75	3.5	3.5	4.5
0.2	Yes	1.5	2.5	3	3	3.75
0.4	Yes	1	2	2.25	2.25	3
≥ 0.6	Yes	1	1	1	1	2.25
0	No	2	2.25	2.25	2.25	2.75
0.2	No	1.5	2	2	2	2.25
0.4	No	1	1.5	1.5	1.5	1.75
0.6	No	1	1	1	1	1
> 0.6	No	—‖	—‖	—‖	—‖	—‖
Condition ii. Post-tensioned slab-column connections†						
$\frac{V_g}{V_o}$ ‡	Continuity reinforcement§					
0	Yes	1.5	2	2.5	2.5	3.25
0.6	Yes	1	1	1	2	2.25
> 0.6	Yes	1	1	1	1.5	1.75
0	No	1.25	1.75	1.75	1.75	2
0.6	No	1	1	1	1	1
> 0.6	No	—‖	—‖	—‖	—‖	—‖
Condition iii. Slabs controlled by inadequate development or splicing along the span†						
		—‖	—‖	—‖	3	4
Condition iv. Slabs controlled by inadequate embedment into slab-column joint†						
		2	2	3	3	4

*Values between those listed in the table should be determined by linear interpolation.
†Where more than one of the Conditions i, ii, iii, and iv occur for a given component, use the minimum appropriate numerical value from the table.
‡V_g is the gravity shear acting on the slab critical section as defined by ACI 318/318M, and V_o is the direct punching shear strength as defined by ACI 318/318M.
§"Yes" should be used where the area of effectively continuous main bottom bars passing through the column cage in each direction is greater than or equal to $0.5V_g/(\phi f_y)$. Where the slab is post-tensioned, "Yes" should be used where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, "No" should be used.
‖Action should be treated as force-controlled.

For moment about an axis parallel to slab edge at exterior connections without transverse beams, where the shear on the slab critical section due to gravity loads does not exceed $0.75V_c$ or the shear at a corner support does not exceed $0.5V_c$, the moment transfer strength can be taken as equal to the flexural strength of a section of slab between lines that are a distance c_1 outside opposite faces of the column or capital. V_c is the direct punching shear strength defined by ACI 318/318M.

C4.4.3 Strength—*Alternative expressions for calculating moment transfer strength of interior and exterior slab-column connections can be found in Luo et al. (1994), and detailed modeling recommendations for reinforced and post-tensioned concrete slab-column frames, as well as comparisons with shake table tests, can be found in Kang et al. (2009).*

4.4.4 *Acceptance criteria*

4.4.4.1 *Linear static and dynamic procedures*—Component actions should be classified as being deformation-controlled or force-controlled, as defined in Section 3.2.1. In primary components, deformation-controlled actions should be restricted to flexure in slabs and columns, and shear and moment transfer in slab-column connections. In secondary components, deformation-controlled actions are acceptable in shear and reinforcement development (Table 4.10). All other actions should be classified as force-controlled.

Design actions on components should be determined as prescribed in ASCE/SEI 41-06, Chapter 3. Where the calculated DCR values exceed unity, the following design actions should be determined using limit analysis principles as prescribed in ASCE/SEI 41-06 Chapter 3:

1. Moments, shears, torsions, and development and splice actions corresponding to the development of component strength in slabs and columns; and
2. Axial load in columns, considering likely plastic action in components above the level in question.

Design actions should be compared with design strengths in accordance with ASCE/SEI 41-06 Section 3.4.2.2 and m-factors for slab-column frame components should be selected from Tables 4.7 and 4.10.

Where the average DCRs for columns at a level exceeds the average value for slabs at the same level, and exceeds the greater of 1.0 and $m/2$, the element should be defined as a weak story element and evaluated by the procedure for weak story element (Section 4.2.4.1).

4.4.4.2 *Nonlinear static and dynamic procedures*—Inelastic response should be restricted to actions in Tables 4.2 and 4.9, except where it is demonstrated by experimental evidence and analysis that other inelastic actions are acceptable for the selected performance levels. Other actions should be defined as force-controlled.

Calculated component actions should satisfy the requirements of ASCE/SEI 41-06 Section 3.4.3.2. Maximum permissible inelastic deformations should be taken from Tables 4.2 and 4.9. Alternative values can be used where justified by experimental evidence and analysis.

C4.4.4.2 Nonlinear static and dynamic procedures—*Refer to Section C4.4.2.2 for discussion of Table 4.9 and acceptance criteria for reinforced concrete slab-column connections. Refer to Section C4.2.2.2 for discussion of Table 4.2 and acceptance criteria for reinforced concrete columns.*

4.4.5 *Rehabilitation measures*—Reinforced concrete slab-column moment frame components that do not meet acceptance criteria for the selected RO should be rehabilitated. Rehabilitation measures should meet the requirements of Section 3.7 and other provisions of this guide.

C4.4.5 Rehabilitation measures—*Rehabilitation measures for reinforced concrete beam-column moment frames can be effective in rehabilitating reinforced concrete slab-column moment frames. Further rehabilitation measures are found in FEMA 547.*

CHAPTER 5—NOTATION, DEFINITIONS, AND ACRONYMS

5.1—Notation

A_g = gross area of column, Eq. (4-1)

A_j = effective cross-sectional area of a beam-column joint, in a plane parallel to plane of reinforcement generating shear in the joint, as defined in Section 4.2.3.2, Eq. (4-2)

A_v = area of shear reinforcement, Eq. (4-1)

A_w = gross area of the web cross section, $= b_w d$

a = parameter used to measure deformation capacity in component load-deformation curves, Fig. 3.1

b = parameter used to measure deformation capacity in component load-deformation curves, Fig. 3.1

b_{eff} = effective slab width

b_w = web width or diameter of circular section

c = parameter used to measure residual strength, Fig. 3.1

c_1 = column dimension parallel to span, Section 4.4

c_2 = column dimension perpendicular to span, Section 4.4

d = distance from extreme compression fiber to centroid of tension reinforcement

d_b = nominal diameter of bar, Eq. (3-3)

d_c = depth of column core in direction of applied shear (centerline-to-centerline of ties)

E = Young's modulus of elasticity

E_c = modulus of elasticity of concrete

E_s = modulus of elasticity of reinforcement

f_c' = compressive strength of concrete (lower-bound or expected strength, as appropriate)

f_{pc} = average compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses)

f_s = maximum stress that can be developed in the bar for straight development, hook development, or lap splice length ℓ_b (Eq. (3-2)) or embedded length ℓ_e (Eq. (3-3))

f_y = yield strength of reinforcement

f_{yL} = lower-bound yield strength of reinforcement, Eq. (3-2)

f_{yt} = yield strength of transverse reinforcement, Section C4.2.2.2

h_c = average height of the beams framing into the joint in the direction of applied shear

I = moment of inertia

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement

k = ductility-related coefficient used for calculation of column shear strength, Eq. (4-1)

L = length of member along which deformations are assumed to occur

ℓ_1 = center-to-center span length in the direction under consideration, Section 4.4

ℓ_2 = center-to-center span length perpendicular to the direction under consideration, Section 4.4

ℓ_b = provided length of straight development, lap splice, or standard hook, Eq. (3-2)

ℓ_d = required length by Chapter 12 of ACI 318/318M-08 for straight bar development, hook development, or lap splice, except that required splice lengths may be taken as straight bar development lengths in tension, Eq. (3-2)

ℓ_e = length of embedment of reinforcement, Eq. (3-3)

M_n = nominal design moment strength at section

M/Vd = largest ratio of column moment to shear times effective depth under design loadings, Eq. (4-1)

M_{CEx} = expected bending strength of a member about the x-axis for axial load P_{UF}, Eq. (3-1)

M_{CEy} = expected bending strength of a member about the y-axis for axial load P_{UF}, Eq. (3-1)

M_{gCS} = moment acting on the slab column strip due to gravity loads, Section 4.4.3

M_{UDx} = deformation-controlled design bending moment due to gravity and earthquake loads about x-axis

M_{UDy} = deformation-controlled design bending moment due to gravity and earthquake loads about y-axis

ΣM_n = sum of positive and negative flexural strengths of a section of slab between lines that are two and one-

half slab or drop panel thicknesses outside opposite faces of the column or capital, Section 4.4.3
ΣM_{nb} = sum of nominal moment capacities of all beams framing into a joint, Section 4.2.2.1
ΣM_{nc} = sum of nominal moment capacities of all columns framing into a joint, Section 4.2.2.1
M_{nCS} = nominal design moment strength of the slab column strip, Section 4.4.3
M_{UD} = deformation-controlled design moment due to gravity and earthquake loads
M_{UDx}= deformation-controlled design bending moment due to gravity and earthquake loads about x-axis for axial load P_{UF}, Eq. (3-1)
M_{UDy}= deformation-controlled design bending moment due to gravity and earthquake loads about y-axis for axial load P_{UF}, Eq. (3-1)
m = component demand modification factor used in linear procedures to account for expected ductility associated with an action at the selected structural performance level
m_x = m-factor for column for bending about x-axis in accordance with Table 4.7, Eq. (3-1)
m_y = m-factor for column for bending about y-axis in accordance with Table 4.7, Eq. (3-1)
N_u = column axial load occurring simultaneously with the column shear demand under consideration, Eq. (4-1)
P = axial load on column
P_f = probability of failure, Section C4.2.2.2
P_o = nominal axial load strength at zero eccentricity
P_{UF} = force-controlled design axial force in a member due to gravity and earthquake loads determined based on procedures defined in Section 3.4.2 of ASCE/SEI 41-06
Q = generalized force in a component, Fig. 3.1
Q_{CE} = expected strength of a component at the deformation level under consideration
Q_{CL} = lower-bound estimate of the strength of a component at the deformation level under consideration
Q_y = yield strength of a component
s = spacing of shear reinforcement
V = design shear force at section
V_g = shear acting on slab critical section due to gravity loads
V_n = nominal shear strength at section, Eq. (4-1); shear strength of the beam-column joint, Eq. (4-2), Tables 4.4 and 4.8
V_o = nominal column shear strength at zero displacement ductility, Eq. (4-1); direct punching shear strength as defined by ACI 318/318M-08, Tables 4.9 and 4.10
V_p = column shear demand at flexural yielding of plastic hinges, Section 4.2.2.2
β = coefficient of variation, Section C4.2.2.2
β_{eff} = effective stiffness factor for slab-column connections, Section C4.4.2.1
γ = coefficient for calculation of joint shear strength, Eq. (4-2) and Table 4.5
γ_f = fraction of unbalanced moment transferred by flexure at slab-column connections, Section 4.4.3
θ = generalized deformation, radians, Fig. 3.1
θ_p = plastic rotation, Section C4.2.2.2
$\theta_{p\ table}$= plastic rotation capacity determined from Table 4.2, Section C4.2.2.2
$\theta_{p\ meas}$= measured plastic rotation from test data, Section C4.2.2.2
κ = a knowledge factor used to reduce component strength based on the level of knowledge obtained for individual components during data collection
λ = correction factor related to unit weight of concrete, Eq. (4-2)
ρ = ratio of non-prestressed tension reinforcement, Table 4.1
ρ' = ratio of non-prestressed compression reinforcement, Table 4.1
ρ_{bal} = reinforcement ratio producing balanced strain conditions, Table 4.1
ϕ = strength reduction factor
Φ^{-1} = inverse standard normal cumulative distribution function, Section C4.2.2.2

5.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, "ACI Concrete Terminology," http://terminology.concrete.org. Definitions provided herein complement that resource.

acceptance criteria—limiting values of properties such as drift, strength demand, and inelastic deformation used to determine the acceptability of a component at a given performance level.

action—an internal force or deformation corresponding to a demand; designated as force- or deformation-controlled.

action, deformation-controlled—an action for which acceptance criteria are based on the deformation demands on the component. Deformation-controlled actions are defined in Section 2.4.4.3 of ASCE/SEI 41-06.

action, force-controlled—an action for which acceptance criteria are based on the force demands on the component. Force-controlled actions are defined in Section 2.4.4.3 of ASCE/SEI 41-06.

braced frame—a vertical lateral force-resisting element consisting of vertical, horizontal, and diagonal components joined by concentric or eccentric connections.

Building Performance Level—a limiting damage state for a building, considering structural and nonstructural components, used in the definition of RO.

closed stirrups or ties—transverse reinforcement defined in Chapter 7 of ACI 318/318M-08 consisting of standard stirrups or ties with 90-degree hooks and lap splices in a pattern that encloses longitudinal reinforcement.

coefficient of variation—for a sample of data, the ratio of the standard deviation for the sample to the mean value for the sample.

component—a part of the structural system of a building, including beams, columns, and slabs; designated as primary or secondary.

component, primary—a structural component that is required to resist seismic forces for the structure to achieve the selected performance level.

component, secondary—a structural component that is not required to resist seismic forces in order for the structure to achieve the selected performance level.

comprehensive knowledge—see **knowledge, comprehensive**.

connection—a link that transmits actions from one component or element to another component or element.

connectors—embedded steel sections, shear plates, headed studs, and welds used to link components to other components.

deformation-controlled action—see **action, deformation-controlled**.

demand—the amount of force or deformation imposed on an element or component.

demand capacity ratio—ratio of force demands from gravity and earthquake loading (determined from linear analysis procedures in Section 3.4.2 of ASCE/SEI 41-06) to the yield strength of a component based on expected or lower-bound strengths, as appropriate.

effective stiffness—an approximation of the elastic stiffness of a component (Table 3.1).

element—an assembly of structural components that act together in resisting forces, including gravity frames, moment-resisting frames, braced frames, shear walls, and diaphragms.

expected strength—see **strength, expected**.

force-controlled action—see **action, force-controlled**.

hoops—transverse reinforcement defined in Chapter 21 of ACI 318/318M-08 consisting of closed ties with 135-degree hooks embedded into the core and no lap splices.

infill—a panel of masonry placed within a concrete frame.

jacketing—the repair or strengthening of a concrete member by encasement using steel, concrete, or fiber-reinforced polymer.

knee joint—a joint that has one column and one beam in the direction of framing.

knowledge, comprehensive—a level of knowledge of the structural condition consistent with that known for new structures. Data collection requirements are defined in Section 2.2.6 of ASCE/SEI 41-06 and Sections 2.2.4.2 and 2.3.2.2 of this guide.

knowledge, usual—a basic level of knowledge of the structural condition needed to proceed with a seismic rehabilitation based on nonlinear procedures. Data collection requirements are defined in Section 2.2.6 of ASCE/SEI 41-06 and 2.2.4.1 of this guide.

knowledge factor—a factor used to account for the uncertainty in the quality of as-built data. The knowledge factor is defined in Section 2.2.6.4 of ASCE/SEI 41-06 and Section 2.4 of this guide.

lateral force-resisting system—elements of the structural system that provide its basic lateral strength and stiffness.

lightweight concrete—structural concrete that has an air-dry unit weight not exceeding 115 lb/ft^3 (1840 kg/m^3).

lower-bound strength—see **strength, lower-bound**.

moment frame—a building frame system in which lateral forces are resisted by shear and flexure in members and joints of the frame.

performance level, building—see **Building Performance Level**.

primary component—see **component, primary**.

rehabilitation measures—modifications to existing components, or installation of new components, that correct deficiencies identified in a seismic evaluation as part of a scheme to rehabilitate a building to achieve a selected RO.

rehabilitation method—one or more measures and strategies for improving the seismic performance of existing buildings.

Rehabilitation Objective—one or more rehabilitation goals with each goal consisting of the selection of a target Building Performance Level and an Earthquake Hazard Level, as defined in Section 1.4 of ASCE/SEI 41-06.

secondary component—see **component, secondary**.

seismic evaluation—a process or methodology of identifying and evaluating deficiencies that impact the seismic performance of the building.

shear wall—wall that resists lateral forces applied parallel with its plane.

special moment frame—a moment frame system that meets the special requirements for frames as defined in seismic provisions for new construction.

strength—the maximum axial force, shear force, or moment that can be resisted by a component.

strength, expected—the mean maximum resistance expected over the range of deformations to which a component is likely to be subjected, including consideration of the variability in material strength, as well as strain hardening and plastic section development.

strength, lower-bound—for a population of test results for similar components, the mean strength less one standard deviation. Nominal material properties, or properties specified in construction documents, can be used to define lower-bound strengths.

structural component—see **component**.

structural system—an assemblage of structural components that are joined together to provide regular interaction or interdependence.

Systematic Rehabilitation Method—an approach to rehabilitation in which complete analysis of the response of the building to earthquake hazards is performed as defined in Section 2.3.2 of ASCE/SEI 41-06.

usual knowledge—see **knowledge, usual**.

5.3—Acronyms

ACI—American Concrete Institute
AISC—American Institute of Steel Construction
ANSI—American National Standards Institute
ASCE—American Society of Civil Engineers
ASTM—American Society for Testing and Materials
CP—Collapse Prevention
DCR—demand capacity ratio
EERI—Earthquake Engineering Research Institute
FEMA—Federal Emergency Management Agency
IO—Immediate Occupancy

LS—Life Safety
NDE—nondestructive examination
NDP—nonlinear dynamic procedure
NSP—nonlinear static procedure
PEER— Pacific Earthquake Engineering Research
RO—Rehabilitation Objective
SEI—Structural Engineering Institute
SMF—special moment frame

CHAPTER 6—REFERENCES

6.1—Referenced standards and reports

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

American Concrete Institute

201.1R Guide for Conducting a Visual Inspection of Concrete in Service
214.4R Guide for Obtaining Cores and Interpreting Compressive Strength Results
228.1R In-Place Methods to Estimate Concrete Strength
228.2R Nondestructive Test Methods for Evaluation of Concrete in Structures
318/318M Building Code Requirements for Structural Concrete and Commentary
352R Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures
355.2 Qualification of Post-Installed Mechanical Anchors in Concrete and Commentary
364.1R Guide for Evaluation of Concrete Structures before Rehabilitation
374.1 Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary
437R Strength Evaluation of Existing Concrete Buildings

American National Standards Institute/American Institute of Steel Construction

ANSI/AISC 360 Specification for Structural Steel Buildings

American Society of Civil Engineers/Structural Engineering Institute

7 Minimum Design Loads for Buildings and Other Structure
41-06 Seismic Rehabilitation of Existing Buildings

ASTM International

A15 Specification for Billet-Steel Bars for Concrete Reinforcement (withdrawn, replaced by A615/A615M)
A16 Specification for Rail-Steel Bars of Concrete Reinforcement (withdrawn 1969, replaced by A616/A616M)
A36/A36M Standard Specification for Carbon Structural Steel
A61 Specification for Deformed Rail Steel Bars for Concrete Reinforcement with 60,000 psi Minimum Yield Strength (withdrawn 1969, replaced by A616/A616M)
A160 Specification for Axle-Steel Bars for Concrete Reinforcement (withdrawn 1969, replaced by A617/A617M)
A185/A185M Standard Specification for Steel Welded Wire Reinforcement, Plain, for Concrete
A370 Standard Test Methods and Definitions for Mechanical Testing of Steel Products
A408 Specification for Special Large Size Deformed Billet-Steel Bars for Concrete Reinforcement (withdrawn 1968, replaced by A615/A615M)
A416/A416M Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete
A421/A421M Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete
A431 Specification for High-Strength Deformed Billet-Steel Bars for Concrete Reinforcement with 75,000 psi Minimum Yield Strength (withdrawn 1968, replaced by A615/A615M)
A432 Specification for Deformed Billet Steel Bars for Concrete Reinforcement with 60,000 psi Minimum Yield Point (withdrawn 1968, no replacement)
A497/A497M Standard Specification for Steel Welded Wire Reinforcement, Deformed, for Concrete
A615/A615M Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement
A616 Standard Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement (withdrawn 1999, replaced by A996/A996M)
A617 Standard Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement (withdrawn 1999, replaced by A996/A996M)
A706/A706M Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement
A722/A722M Standard Specification for Uncoated High-Strength Steel Bars for Prestressing Concrete
A955/A955M Standard Specification for Deformed and Plain Stainless-Steel Bars for Concrete Reinforcement
C39/C39M Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
C42/C42M Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C496/C496M Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
E488 Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements

Concrete Reinforcing Steel Institute

DA24 Evaluation of Reinforcing Steel Systems in Old Reinforced Concrete Structures, Engineering Data Report No. 48

Federal Emergency Management Agency

273 NEHRP Guidelines for the Seismic Rehabilitation of Buildings

274 NEHRP Commentary on the Guidelines for Seismic Rehabilitation of Buildings

306 Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings—Basic Procedures Manual

307 Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings—Technical Resources

308 Repair of Earthquake Damaged Concrete and Masonry Wall Buildings

450 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 2003 Edition, Part 1: Provisions and Part 2: Commentary

547 Techniques for the Seismic Rehabilitation of Existing Buildings

Structural Engineering Institute/American Society of Civil Engineers

11 Guideline for Structural Condition Assessment of Existing Buildings

These publications may be obtained from these organizations:

American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
www.concrete.org

American Institute of Steel Construction
One East Wacker Drive Suite 700
Chicago, IL 60601-1802
www.aisc.org

American National Standards Institute
1899 L Street, NW, 11th Floor
Washington, DC 20036
www.ansi.org

American Society of Civil Engineers
1801 Alexander Bell Drive
Reston, VA 20191
www.asce.org

ASTM International
100 Barr Harbor Drive
West Conshohocken, PA 19428
www.astm.org

Concrete Reinforcing Steel Institute
933 N. Plum Grove Road
Schaumburg, IL 60173
www.crsi.org

Federal Emergency Management Agency
P.O. Box 10055
Hyattsville, MD 20782-7055
www.fema.gov/index.shtm

Structural Engineering Institute/American Society of Civil Engineers
1801 Alexander Bell Drive
Reston, VA 20191
http://content.seinstitute.org

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