

Guide for Seismic Rehabilitation of Existing Concrete Frame Buildings and Commentary

Reported by ACI Committee 369



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Guide for Seismic Rehabilitation of Existing Concrete Frame Buildings and Commentary

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Guide for Seismic Rehabilitation of Existing Concrete Frame Buildings and Commentary

Reported by ACI Committee 369

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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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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 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 determine 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 yd³ (306 m³) of concrete, or 10,000 ft² (930 m²) 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 yd³ (306 m³) of concrete, or 10,000 ft² (930 m²) 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 frame. 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 [†]	Intermediate [†]	Hard [†]	60	65	70	75
		33	40	50				
	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)
Year	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 reinforcement 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 over-strength 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 $0.67A_gf'_c$ and shear span-depth ratios greater than 1.4. Measured effective stiffnesses from the laboratory test data suggest that the effective flexural rigidity for low axial loads could be approximated as $0.2EI_g$; however, considering the scatter in the effective flexural rigidity and to avoid under-estimating the shear demand on columns with low axial loads, $0.3EI_g$ 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 $0.3A_gf'_c$, the effective flexural rigidity can be estimated as $0.5EI_g$, with linear interpolation to the value given in Table 3.1 for axial loads greater than $0.5A_gf'_c$.

Due to low bond stress between concrete and plain reinforcement without deformations, components with plain longitudinal reinforcement and axial loads less than $0.5A_gf'_c$ 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

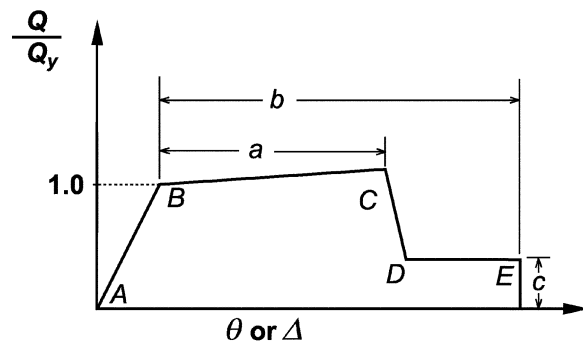


Fig. 3.1—Generalized force-deformation relation for concrete elements or components.

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.

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