

An ACI Standard

Seismic Evaluation and Retrofit of Existing Concrete Buildings—Code and Commentary

Reported by ACI Committee 369

ACI CODE-369.1-22



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Seismic Evaluation and Retrofit of Existing Concrete Buildings— Code and Commentary

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Seismic Evaluation and Retrofit of Existing Concrete Buildings—Code and Commentary

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This code provides retrofit and rehabilitation criteria for reinforced concrete buildings based on results from the most recent research on the seismic performance of existing concrete buildings. The intent of this code is to provide provisions related to concrete material and concrete members, including material testing criteria, modeling parameters, and acceptance criteria for use within the ASCE 41 framework, similar to how the National Earthquake Hazards Reduction Program (NEHRP) Recommended Seismic Provisions produced by the Federal Emergency Management Agency (FEMA) (FEMA 450) have served as source documents for the International Building Code (IBC) and its predecessor building codes. Starting in the 2023 edition, ASCE 41 references ACI 369.1 directly for concrete-related provisions. Chapter and section numbers of ASCE 41 cited in ACI 369.1 correspond to ASCE 41-17 unless otherwise noted.

This code should be used in conjunction with Chapters 1 through 8 of ASCE 41-17 and applicable sections of the ACI 318-19 Building Code. Chapter 1 of ASCE 41 provides general requirements for evaluation and retrofit, including the selection of performance objectives and retrofit strategies. Chapter 2 of ASCE 41 defines performance objectives and seismic hazards. Chapter 3 of ASCE 41 provides requirements for evaluation and retrofit, including treating as-built information and selecting appropriate screening procedures. Chapter 4 of ASCE 41 summarizes Tier 1 screening procedures, while Chapters 5 and 6 summarize Tier 2 deficiency-based procedures and Tier 3 systematic procedures for evaluation and retrofit, respectively. Chapter 7 of ASCE 41 details analysis procedures referenced in ACI 369.1, including linear and nonlinear analysis procedures, acceptance criteria, and alternative methods for determining modeling parameters and acceptance criteria. Chapter 8 of ASCE 41 provides geotechnical engineering provisions for building foundations and assessment of seismic-geologic site hazards. References to these chapters can be found throughout the standard. The design professional is referred to FEMA 547 for detailed information on seismic rehabilitation measures for concrete buildings. Repair techniques for earthquake-damaged concrete components are not included in ACI 369.1. The licensed design professional is referred to ACI 562, FEMA 306,

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FEMA 307, and FEMA 308 for information on evaluation and repair of damaged concrete wall components.

This code does not provide modeling procedures, acceptance criteria, or rehabilitation measures for concrete-encased steel composite components.

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

INTRODUCTION

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

in future earthquakes but also to extend the life of existing buildings and reduce the need for new construction in the promotion of sustainability objectives.

This code is intended to cover all buildings of the usual types, both large and small. Requirements more stringent than the code provisions may be desirable for unusual construction. It is not possible to codify all problems encountered in the process of performing the seismic evaluation and retrofit of reinforced concrete buildings, nor is the intent of this code to do so. This code provides a basic framework for modeling and evaluation of structures that reflects the latest information available from researchers and practicing engineers enabling seismic evaluation and retrofit to be performed with a consistent set of criteria. Many provisions in this code rely on the use of sound engineering judgment for their implementation. The code and commentary cannot replace sound engineering knowledge, experience, and judgment. The commentary of this code provides references that describe in detail the implementation of methodologies adopted in the standard.

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CHAPTER 1—GENERAL

1.1—Scope

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

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

COMMENTARY

CHAPTER R1—GENERAL

R1.1—Scope

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

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

Concrete-encased steel-composite components behave differently from concrete sections reinforced with steel reinforcement. Concrete-encased steel-composite components frequently behave as over-reinforced sections. This type of component behavior was not represented in the data sets used to develop the force-deformation modeling relationships and acceptance criteria in this code, and is not covered herein.

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

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CHAPTER 2—MATERIAL PROPERTIES AND
CONDITION ASSESSMENT

2.1—General

Mechanical properties of in-place materials shall be obtained from available drawings, specifications, and other documents for the existing building in accordance with the requirements in 3.2 of [ASCE 41](#). When these documents fail to provide adequate information to quantify material properties, such information shall be supplemented by materials testing based on requirements of Chapter 2. Material properties of existing concrete components shall be determined in accordance with 2.2. The use of default material properties based on historical information is permitted in accordance with 2.2.5. A condition assessment of the existing building and site shall be conducted in accordance with 2.3. The extent of materials testing and condition assessment performed shall be used to determine the knowledge factor as specified in 2.4. Mechanical properties of new materials used for retrofit of existing structures shall be obtained in accordance with 2.5.

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CHAPTER R2—MATERIAL PROPERTIES AND
CONDITION ASSESSMENT

R2.1—General

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

When modeling a concrete building, it is important to investigate local practices relative to seismic design. Specific benchmark years can be determined for the implementation of earthquake-resistant design in most locations, but caution should be exercised in assuming optimistic characteristics for any specific building. Particularly with concrete materials, the date of original building construction significantly influences seismic performance. Without deleterious conditions or materials, concrete gains compressive strength from the time it is originally cast. Strengths typically exceed specified design values (28-day or similar). In older construction, concrete strength was often very low (less than 3000 lb/in.²) and it was rarely specified in the drawings. Early adoptions of concrete in buildings often used steel reinforcement with relatively low strength and ductility, limited continuity, and reduced bond development. Continuity between specific existing components and elements, such as beams, columns, diaphragms, and shear walls, can be particularly difficult to assess because of concrete cover and other barriers to inspection.

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

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

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

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

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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 shall be obtained for the as-built structure:

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

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

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

Table 2.2.1.2—Factors to translate lower-bound material properties to expected strength material properties

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

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

- a) Cross-sectional dimensions of individual components and overall configuration of the structure
- b) Configuration of component connections, size, embedment depth, type of anchors, thickness of connector mate-

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R2.2—Properties of in-place materials and components**R2.2.1** *Material properties*

R2.2.1.1 *General*—Other material properties and conditions of interest for concrete components include:

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

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

The sample size and removal practices followed are referenced in **FEMA 274-14**, R6.3.2.3, and R6.3.2.4. **ACI 228.1R** provides guidance on methods to estimate the in-place strength of concrete in existing structures, whereas **ACI 214.4R** provides guidance on coring in existing structures and interpretation of core compressive strength test results. Generally, mechanical properties for both concrete and steel reinforcement can be established from combined core and specimen sampling at similar locations, followed by laboratory testing. Core drilling should minimize damage to the existing steel reinforcement.

R2.2.2 *Component properties*—Component properties are required to properly characterize building performance in seismic analysis. The starting point for assessing component properties and condition is review of available construction documents. A preliminary review should identify primary gravity and seismic-force resisting elements and systems

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rial, anchorage and interconnection of embedments, and the presence of bracing or stiffening components

c) Modifications to components or overall configuration of the structure

d) Most recent physical condition of components and connections, and the extent of any deterioration

e) Deformations beyond those expected because of gravity loads, such as those caused by settlement or past earthquake events

f) Presence of other conditions that influence building performance, such as nonstructural components that can interact with structural components during earthquake excitation

2.2.3 Test methods to quantify material properties

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

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

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

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

Removal of core samples and performance of laboratory destructive testing shall be permitted to determine existing concrete strength properties. Removal of core samples shall use the procedures included in **ASTM C42/C42M**. Testing shall follow the procedures contained in **ASTM C42/C42M**, **ASTM C39/C39M**, and **ASTM C496/C496M**. Core strength shall be converted to in-place concrete compressive strength by an approved procedure. **ACI 214.4R** provides guidance on the conversion procedure. The removal of bar or tendon samples and performance of laboratory destructive testing shall be permitted to determine existing steel reinforcement strength properties. The tensile yield and ultimate

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and their critical components and connections. If there are no drawings of the building, the licensed design professional should perform a thorough investigation of the building to identify these elements, systems, and components as described in 2.3.

R2.2.3 Test methods to quantify material properties

R2.2.3.2 Sampling—**ACI 214.4R** and **FEMA 274** provide further guidance on correlating concrete core strength to in-place strength and provide references for various test methods that can be used to estimate material properties. Chemical composition can be determined from retrieved samples to assess the condition of the concrete. Section C6.3.3.2 of **FEMA 274-97** provides references for these tests.

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

The steel reinforcement system used in the construction of a specific building is usually of uniform grade and similar strength. One grade of reinforcement is occasionally used for small-diameter bars, such as those used for stirrups and hoops, and another grade for large-diameter bars, such as those used for longitudinal reinforcement. In some cases, different concrete design strengths or classes are used. Historical research and industry documents have shown that on material mechanical properties used in different construction

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mate strengths for reinforcing and prestressing steels shall follow the procedures included in **ASTM A370**. Reinforcement samples that are slightly damaged during removal are permitted to be machined to a round bar as long as the tested area is at least 70% of the gross area of the original bar. Prestressing materials shall meet the supplemental requirements in **ASTM A416/A416M**, **ASTM A421/A421M**, or **ASTM A722/A722M**, depending on material type. Properties of connector steels shall be permitted to be determined by wet and dry chemical composition tests, and direct tensile and compressive strength tests as specified by ASTM A370. Where strength, construction quality, or both, of anchors or embedded connectors are required to be determined, in-place testing shall satisfy the provisions of **ASTM E3121/E3121M**.

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

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

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

- a) If the specified design strength of the concrete is known, at least one core shall be taken from samples of each different concrete strength used in the construction of the building, with a minimum of three cores taken for the entire building
- b) If the specified design strength of the concrete is not known, at least one core shall be taken from each type of seismic-force-resisting component, with a minimum of six cores taken for the entire building
- c) If the specified design strength of the steel reinforcement is known, nominal or specified material properties shall be permitted without additional testing
- d) If the specified design strength of the steel reinforcement is not known, at least two strength test coupons of steel reinforcement shall be removed from the building for testing

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tion eras (2.2.5). This information can be used with laboratory and field test data to gain confidence in in-place strength properties. Undamaged steel reinforcement can be reduced to a smooth bar if the samples meet the requirements of ASTM A370, excluding the limitations of Annex 9. This type of reinforcement would occur in a situation where only a limited length of bar can be removed for testing.

R2.2.4 Minimum number of tests—To quantify in-place properties accurately, it is essential that a minimum number of tests be conducted on primary components of the seismic-force-resisting system. The minimum number of tests is dictated by the availability of original construction data, structural system type used, desired accuracy, quality and condition of in-place materials, level of seismicity, and target performance level. Accessibility to the structural system can influence the testing program scope. The focus of testing should be on primary seismic-force-resisting components and specific properties for analysis. Test quantities provided in this section are minimal; the licensed design professional should determine whether further testing is needed to evaluate as-built conditions.

Testing is generally not required on components other than those of the seismic-force-resisting system. The licensed design professional and subcontracted testing agency should carefully examine test results to verify that suitable sampling and testing procedures were followed and appropriate values for the analysis were selected from the data.

R2.2.4.1 Usual data collection—Sets of anchors should be based on similar types, manufacturer, and installation orientation (such as overhead, horizontal, or downward) and should represent similar areas of concrete substrate within the building. The licensed design professional responsible for the test program, in coordination with the licensed design professional, should designate the anchor sets.

A minimum of six tests of a set is necessary for a statistical evaluation of the test results at a 90 percent confidence level. For usual data collection, the number of tests can be decreased to four for each set with a resulting decrease in the confidence level that all of the anchors will achieve the expected strength. Four tests to failure is the recommended minimum number of tests needed to establish a characteristic capacity using the procedure in **ACI 355.2-19**, Appendix A2.

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e) Existing cast-in-place or post-installed anchors shall be classified in sets based on common diameter, type of anchor, orientation of installation, embedment, similar concrete substrate, and structural use. In sets of anchors used for out-of-plane wall anchorage and in sets of anchors whose failure in tension or shear would cause the structure not to meet the selected performance objective, a minimum of four anchors of each anchor set not exceeding 100 anchors and a minimum of six anchors of each anchor set including 100 anchors or more shall be tested to a proof load to establish an available strength in accordance with 3.6.3

2.2.4.2 Comprehensive data collection

2.2.4.2.1 Coefficient of variation—Unless specified otherwise, a minimum of three tests shall be conducted to determine any property. If the coefficient of variation exceeds 20%, additional tests shall be performed until the coefficient of variation is equal to or less than 20%. If additional testing does not reduce the coefficient of variation below 20%, a knowledge factor reduction per 2.4 shall be used. In determining coefficient of variation, cores shall be grouped by grades of concrete and element type. The number of tests in a single component shall be limited to prevent compromising the integrity of the component.

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

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

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

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

R2.2.4.2 Comprehensive data collection

R2.2.4.2.2 Concrete materials—ACI 214.4R provides guidance on coring in existing structures and interpretation of core compressive strength test results. If a structure was constructed in phases or if construction documents for different parts of the structure were issued at separate times, the licensed design professional, to determine sampling size, should consider the concrete in each construction phase or in each set of construction documents a different type. Section 6.4.3 of ACI 562-19 provides a method to calculate an equivalent specified concrete strength f'_c based on statistical analysis of compression strength test results from core samples. ASTM E178 provides guidance on consideration of outliers in a set of core samples. Equation (6.4.3.1) of ACI 562-19 defines the equivalent specified compressive strength of concrete as a function of the number of tests, the coefficient of variation of the samples, and a factor to account for the number of samples. Section 6.4.3 of ACI 562-19 permits the licensed design professional to select the number of samples used to evaluate concrete compressive strength; however, the strength varies based on the number of samples to account for the uncertainty associated with the number of samples.

Equation (6.4.3.1) of ACI 562-19 was derived with the objective of calculating the 13% fractile of the in-place concrete compressive strength, which some studies have shown to be approximately equal to the specified compressive strength of concrete f'_c (Bartlett and MacGregor 1996). The first term in Eq. (6.4.3) of ACI 562-19 represents the effect of sample size on the uncertainty of the mean concrete strength, where the coefficient k is obtained from a student's

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

c) Alternately, for concrete elements for which the design strength is known or unknown, and test results are not available, it is permitted to determine the lower-bound compressive strength based on core sample testing and applying the provisions in 6.4.3 of ACI 562-19. If the lower-bound compressive strength is determined in this manner, the expected compressive strength shall be determined as the lower-bound compressive strength value obtained from ACI 562-19, Eq. (6.4.3.1), plus one standard deviation of the strength of the core samples. When following the provisions in 6.4.3 of ACI 562-19, the minimum number of samples per element type shall be four. The sample locations shall be:

- a) Distributed to quantify element material properties throughout the height of the building
- b) Distributed to quantify element material properties in locations critical to the structural system being investigated

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

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t distribution with $n - 1$ degrees of freedom and a 90% confidence level. The second term in Eq. (6.4.3.1) of ACI 562-19 represents the uncertainty attributable to correction factors relating cylinder strength to specified compressive strength, which were assumed to have a normal distribution, also estimated with a 90% confidence level. The study by Bartlett and MacGregor (1996) showed that the specified compressive strength f'_c corresponds approximately to the 13% fractile of the 28-day in-place strength in walls and columns, and approximately the 23% fractile of the 28-day in-place compressive strength in beams and slabs. The former was considered to be a more appropriate measure of specified compressive strength f'_c than the latter because the nominal strength of columns is more sensitive to concrete compressive strength than the strength of beams and slabs (ACI 214.4R).

In 2.2.1.2, it is stated that nominal material properties or properties specified in construction documents shall be taken as lower-bound material properties unless otherwise specified. The method to estimate the specified concrete compressive strength f'_c in 6.4.3 of ACI 562-19 was adopted in this code to obtain the lower-bound compressive strength consistent with the provisions in 2.2.1.2.

The minimum of four samples was adopted based on the recommendations in ACI 214.4R. The following equation is provided in ACI 214.4R.

$$n_{\text{samples}} = \left(\frac{2 \text{COV}_{\text{population}}}{e_{\text{population}}} \right)^2 \quad (\text{R2.2.4.2.2})$$

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

Users of the document are cautioned that, for coefficients of variation between 13 and 20%, the minimum number of samples needed to limit the error below one standard deviation according to the recommendations in ACI 214.4R is higher than four. For example, for a coefficient of variation of 20%, a minimum of seven samples is recommended to limit the error to one standard deviation. If the maximum error is reduced to 10%, the minimum number of samples recommended is significantly higher. For a coefficient of variation of 15.87% (one standard deviation from the mean) and a maximum error of 10%, the minimum number

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

a) If original construction documents defining properties exist, then at least three strength coupons shall be removed from random locations from each element or component type and tested

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

c) If the construction date is unknown, at least six strength coupons for every three floors shall be performed

Refer to 2.2.3.2 for replacement of sampled material.

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

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

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

2.2.4.2.5 Cast-in-place or post-installed anchors—Existing cast-in-place or post-installed anchors shall be classified in sets in accordance with 2.2.4.1. In sets of anchors used for out-of-plane wall anchorage and in sets of anchors

of samples recommended is 11, and for a coefficient of variation of 20% and a maximum error of 10%, the minimum number of samples recommended is 16. If the coefficient of variation exceeds 20%, the requirements in 2.2.4.2.1 are satisfied.

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

R2.2.4.2.5 Cast-in-place or post-installed anchors—To estimate ultimate strength of the anchors in accordance with **3.6**, the frequency of the test should be increased to at least 25% of the anchors and the test load should be at least the

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whose failure in tension or shear causes the structure not to meet the selected performance objective, 10% of the anchors with a minimum of six anchors of each anchor set shall be tested to a proof load in accordance with 3.6.3. Where other anchors or anchorage assemblies are critical parts of the load paths and are nonredundant, all anchors in the anchorage assembly shall be individually proof-load tested in accordance with 3.6.3.

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

Default lower-bound concrete compressive strengths are specified in Table 2.2.5a. Default expected concrete compressive strengths shall be determined by multiplying lower-bound values by an appropriate factor selected from Table 2.2.1.2, unless another factor is justified by test data. The appropriate default compressive strength, lower-bound strength, or expected strength as specified in ASCE 41, 7.5.1.3, shall be used to establish other strength and performance characteristics for the concrete as needed in the structural analysis.

Default lower-bound values for steel reinforcement are specified for various ASTM specifications and periods in Tables 2.2.5b and 2.2.5c. Default expected strength values for steel reinforcement shall be determined by multiplying

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nominal design strength in accordance with Chapter 17 of ACI 318. In-place anchor testing performed in accordance with 2.2.4.2.5 provides the minimum available tensile strength of a single anchor, which is likely governed by pullout or bond strength in tension. Other failure modes and parameters that affect the strength of the anchors, such as proximity to edges, group effect, presence of cracks, or eccentricity of applied loads, should be considered in accordance with Chapter 17 of ACI 318.

R2.2.5 Default properties—Default values provided in this code are generally conservative. Whereas the strength of steel reinforcement can be fairly consistent throughout a building, the strength of concrete in a building could be highly variable, given variability in concrete mixtures and sensitivity to water-cement ratio (w/c) and curing practices. A conservative assumption based on the field observation of the concrete compressive strength in the given range is recommended unless a higher strength is substantiated by construction documents, test reports, or material testing. For the capacity of an element in question, the lower value within the range can be conservative. It can be appropriate to use the maximum value in a given range where determining the force-controlled actions on other components.

Until approximately 1920, a variety of proprietary steel reinforcements was used. Yield strengths are likely to be in the range of 33,000 to 55,000 lb/in.², but higher values are possible and actual yield and tensile strengths can exceed

Table 2.2.5a—Default lower-bound compressive strength of structural concrete, lb/in.²

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

Table 2.2.5b—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, lb/in. ²	33,000	40,000	50,000	60,000	65,000	70,000	75,000
	Minimum tensile, lb/in. ²	55,000	70,000	80,000	90,000	75,000	80,000	100,000
1911 to 1959		x	x	x		x		
1959 to 1966		x	x	x	x	x	x	x
1966 to 1972			x	x	x	x	x	
1972 to 1974			x	x	x	x	x	
1974 to 1987			x	x	x	x	x	
1987 to present			x	x	x	x	x	x

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

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

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Table 2.2.5c—Default lower-bound tensile and yield properties of reinforcing steel for various ASTM specifications and periods

ASTM designation*	Steel type	Year range	ASTM grade	Structural†	Intermediate‡	Hard†				
				33	40	50	60	65	70	75
			Minimum yield, lb/in. ²	33,000	40,000	50,000	60,000	65,000	70,000	75,000
			Minimum tensile, lb/in. ²	55,000	70,000	80,000	90,000	75,000	80,000	100,000
A15 (withdrawn)	Billet	1911 to 1966		x	x	x				
A16 (withdrawn)	Rail‡	1913 to 1966				x				
A61 (withdrawn)	Rail‡	1963 to 1966					x			
A160 (withdrawn)	Axle	1936 to 1964		x	x	x				
A160 (withdrawn)	Axle	1965 to 1966		x	x	x	x			
A185	WWR	1936 to present						x		
A408 (withdrawn)	Billet	1957 to 1966		x	x	x				
A431	Billet	1959 to 1966								x
A432 (withdrawn)	Billet	1959 to 1966					x			
A497	WWR	1964 to present							x	
A615/A615M	Billet	1968 to 1972			x		x			x
A615/A615M	Billet	1974 to 1986			x		x			
A615/A615M	Billet	1987 to present			x		x			x
A616 [§] (withdrawn)	Rail‡	1968 to 1999				x	x			
A617 (withdrawn)	Axle	1968 to 1999			x		x			
A996	Rail, Axle	2000 to present			x	x	x			
A706/A706M [#]	Low-alloy	1974 to present					x			
A955	Stainless	1996 to present			x		x			x

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

†ASTM steel is marked with the letter W.

‡Rail bars are marked with the letter R.

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

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

Note: An entry of x indicates that the grade was available in those years. The use of default values for prestressing steel in prestressed concrete construction shall not be permitted.

lower-bound values by an appropriate factor selected from Table 2.2.1.2, unless another factor is justified by test data. Where default values are assumed for existing steel reinforcement, welding or mechanical coupling of new reinforcement to the existing steel reinforcement shall not be permitted.

The default lower-bound yield strength for steel connector material shall be taken as 27,000 lb/in.². The default expected yield strength for steel connector material shall be determined by multiplying lower-bound values by an appropriate factor selected from Table 2.2.1.2, unless another value is justified by test data.

The default lower-bound yield strength for cast-in-place or post-installed anchor material shall be taken as 27,000 lb/in.² unless another value is justified by test data. Component actions on the connections shall be considered as force-

minimum values. Once commonly used to designate steel reinforcement grade, the terms structural, intermediate, and hard became obsolete in 1968. Plain and twisted square bars were occasionally used between 1900 and 1949. Factors to convert default steel reinforcement strength to expected strength include consideration of material overstrength and strain hardening.

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controlled actions and default expected yield strength shall not be used.

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

2.3—Condition assessment

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

The condition assessment shall include the following:

- a) Examination of the physical condition of primary and secondary components, and the presence of any degradation shall be noted
- b) Verification of the presence and configuration of components and their connections, and the continuity of load paths between components, elements, and systems
- c) A review and documentation of other conditions, including neighboring party walls and buildings, presence of nonstructural components and mass, and prior remodeling
- d) Collection of information needed to select a knowledge factor in accordance with 2.4
- e) Confirmation of component orientation, plumbness, and physical dimensions

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

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

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

A visual building inspection shall include visible portions of foundations, seismic-force-resisting members, diaphragms (slabs), and connections. As a minimum, a representative sampling of at least 20% of the components and connections shall be visually inspected at each floor level. If significant damage or degradation is found, the assessment sample of all similar-type critical components in the building shall be increased to 40% 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 shall be permitted to be performed using drilled holes and a fiberscope.

2.3.2.2 Comprehensive condition assessment—Reinforcement details shall be observed and documented. All removed

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R2.3—Condition assessment

R2.3.1 General—The condition assessment also affords an opportunity to review other conditions that can influence concrete elements and systems and overall building performance. Of particular importance is the identification of other elements and components that can contribute to or impair the performance of the concrete system in question, including infills, neighboring buildings, and equipment attachments. Limitations posed by existing coverings, wall and ceiling space, infills, and other conditions shall also be defined such that prudent retrofit measures can be planned.

R2.3.2 Scope and procedures

R2.3.2.1 Visual condition assessment—Further guidance can be found in [ACI 201.1R](#) and [ACI 364.1R](#), which provides a system for reporting the condition of concrete in service.

R2.3.2.2 Comprehensive condition assessment—Where exposure of reinforcement is necessary, the least practical

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concrete cover shall be replaced after the observation. The following criteria shall be used for assessing primary connections in the building for comprehensive data collection:

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

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

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

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amount of concrete cover and other materials should be removed.

R2.3.2.3 Additional testing—The physical condition of components and connectors affects their performance. The need to accurately identify the physical condition can dictate the need for certain additional destructive and nondestructive test methods. Such methods can be used to determine the degree of damage or presence of deterioration and to improve understanding of the internal condition and concrete quality. Further guidelines and procedures for destructive and nondestructive tests that can be used in the condition assessment are provided in [ACI 228.1R](#), [ACI 228.2R](#), [FEMA 274](#), and [FEMA 306](#).

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

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

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

c) Online monitoring using acoustic emissions, strain gauges, in-place static or dynamic load tests, and ambient vibration tests can be used to assess structural condition and performance. Monitoring is used to determine if active degradation or deformations are occurring, whereas nondestructive load testing provides direct insight on load-carrying capacity.

d) Electromagnetic methods using a pachometer or radiography can be used to locate, size, or perform an initial assessment of steel reinforcement. Further assessment of

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

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

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

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

2.4—Knowledge factor

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

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

2.5—Properties of new materials

2.5.1 General—Properties of new materials shall be specified in accordance with this code and in accordance with the applicable material codes and standards in 2.5.1.1 through 2.5.1.3.

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suspected corrosion activity should use electrical half-cell potential and resistivity measurements.

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

R2.5—Properties of new materials



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2.5.1.1 Cast-in-place reinforced concrete and shotcrete shall comply with provisions in **ACI 318** and **ACI 562**.

2.5.1.2 Structural steel shall comply with provisions in Chapter 9 of **ASCE 41**.

2.5.1.3 Externally bonded FRP systems shall comply with provisions in 7.8 and 10.3 of **ACI 562**.

2.5.2 *Lower-bound material properties*—For new materials, lower-bound material properties shall be defined by material properties specified in the construction documents.

2.5.3 *Expected material properties*

2.5.3.1 *Concrete and steel reinforcement*—Expected material properties of new concrete and steel reinforcement shall be determined in accordance with Section A.9 of **ACI 318**. Alternative factors to convert from specified to expected strength shall be permitted where justified by test data and as approved by the agency having jurisdiction.

2.5.3.2 *Structural steel*—Expected material properties of new structural steel elements shall be determined in accordance with **ASCE 41**, Chapter 9.

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R2.5.1.1 Further recommendations for shotcrete construction are provided in **ACI 506.2**.

R2.5.1.3 Further recommendations for FRP construction are provided in **ACI 440.2R**.

R2.5.2 *Lower-bound material properties*—Specified strength and ultimate strain of FRP materials used for design in accordance with Chapter 13 of **ACI 440.2R** represent values determined using mean minus three standard deviations (**ACI 440.8**). These properties should be taken as lower-bound material properties in this code.

R2.5.3 *Expected material properties*—Design of FRP materials is based on specified strength and ultimate strain values such as those defined in accordance with **ACI 440.2R**; therefore, expected material properties are not provided.

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CHAPTER 3—GENERAL ASSUMPTIONS AND REQUIREMENTS

3.1—Modeling and design

3.1.1 General—New components connected to the existing structure shall comply with ACI 318, except as otherwise indicated in this code.

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

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

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

3.1.2.1 Linear procedures—Where design actions are determined using the linear procedures of ASCE 41, Chapter 7, effective stiffness values in Table 3.1.2.1 shall be permitted.

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CHAPTER R3—GENERAL ASSUMPTIONS AND REQUIREMENTS

R3.1—Modeling and design

R3.1.1 General—Seismic retrofit of a concrete building involves the design of new components connected to the existing structure, seismic upgrading of existing components, or both. 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.

R3.1.2 Stiffness—For columns with low axial loads (below approximately $0.1A_gf'_c$), deformations caused by bar slip can account for as much as 50% of the total deformations at yield. Further guidance regarding calculation of the effective stiffness of reinforced concrete columns that include the effects of flexure, shear, and bar slip can be found in Elwood and Eberhard (2009). Flexure-controlled wall stiffness can vary from approximately $0.15E_cE_g$ to $0.5E_cE_g$, depending on wall reinforcement and axial load. A method for calculating wall stiffness, which provides compatibility with fiber section analysis, is offered in R7.2.2.

R3.1.2.1 Linear procedures—The effective flexural rigidity values in Table 3.1.2.1 for beams and columns account for the additional flexibility from reinforcement slip within the beam-column joint or foundation before yielding. The values specified for columns were determined based on a database of 221 rectangular reinforced concrete column tests with axial loads less than $0.67A_gf'_c$ and shear span-depth ratios greater than 1.4. Measured effective stiffnesses from the laboratory test data suggest that the effective flexural rigidity for low axial loads could be approximated as $0.2EI_g$; however, considering the scatter in the effective flexural rigidity and to avoid underestimating the shear demand on columns with low axial loads, $0.3EI_g$ is recommended in Table 3.1.2.1 (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 caused by flexure, slip, and shear (Elwood and Eberhard 2009).

The modeling recommendations for beam-column joints (6.2.2.1) do not include the influence of reinforcement slip. When the effective stiffness values for columns from Table 3.1.2.1 are used in combination with the modeling

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Table 3.1.2.1—Effective stiffness values for linear analysis

Component	Axial rigidity	Flexural rigidity	Shear rigidity
Beams—nonprestressed*	$1.0E_{cE}A_g$	$0.3E_{cE}I_g$	$0.4E_{cE}A_w$
Beams—prestressed*	$1.0E_{cE}A_g$	$E_{cE}I_g$	$0.4E_{cE}A_w$
Columns with compression caused by design gravity loads $\geq 0.5A_gf'_{cE}$ [†]	$1.0E_{cE}A_g$	$0.7E_{cE}I_g$	$0.4E_{cE}A_w$
Columns with compression caused by design gravity loads $\leq 0.1A_gf'_{cE}$ or with tension [†]	$1.0E_{cE}A_g$ (compression) $1.0E_sA_s$ (tension)	$0.3E_{cE}I_g$	$0.4E_{cE}A_w$
Beam-column joints	$1.0E_{cE}A_g$	Refer to 4.2.2.1	
Flat slabs—nonprestressed	—	Refer to 4.4.2	$0.4E_{cE}A_g$
Flat slabs—prestressed	—	Refer to 4.4.2	$0.4E_{cE}A_g$
Diaphragms (in-plane) —nonprestressed [§]	$0.25E_{cE}A_g$	$0.25E_{cE}I_g$	$0.25E_{cE}A_g$
Diaphragms (in-plane) —prestressed [§]	$0.5E_{cE}A_g$	$0.5E_{cE}I_g$	$0.4E_{cE}A_g$
Walls—uncracked with compression caused by design gravity loads $\geq 0.3A_gf'_{cE}$ ^{†,‡}	$1.0E_{cE}A_g$	$1.0E_{cE}I_g$	$0.4E_{cE}A_w$
Walls—uncracked with compression caused by design gravity loads $\leq 0.05A_gf'_{cE}$ or with tension ^{†,‡}	$1.0E_{cE}A_g$	$0.5E_{cE}I_g$	$0.4E_{cE}A_w$
Walls—cracked ^{†,§}	$1.0E_{cE}A_g$ (compression) $1.0E_sA_s$ (tension)	$0.25E_{cE}I_g$	$0.15E_{cE}A_w$
Coupling beams with longitudinal or diagonal reinforcement	$1.0E_{cE}A_g$	$0.05(I_w/h)E_{cE}I_g$ $\leq 0.20 E_{cE}I_g$	$0.2E_{cE}A_w$

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

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

[‡]Walls are permitted to be considered cracked due to earthquake demands in flexural actions where flexural demands exceed M_{crE} , in shear actions where shear stress demands exceed $2\sqrt{f'_{cE}}$, or both. It shall be permitted to assume all walls to be cracked.

[§]In-plane diaphragm effective stiffness values apply where diaphragm flexibility is considered in accordance with 10.2.2.

[#]Alternative stiffness values dependent on axial load and boundary longitudinal reinforcement ratio shall be permitted in accordance with 7.3.

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 $0.5E_{cE}I_g$, with linear interpolation to the value given in Table 3.1.2.1 for axial loads greater than $0.5A_gf'_c$.

Because of low bond stress between concrete and plain reinforcement without deformations, components with plain longitudinal reinforcement and axial loads less than $0.5A_gf'_c$ can have lower effective flexural rigidity values than in Table 3.1.2.1.

3.1.2.2 Nonlinear procedures

R3.1.2.2 Nonlinear procedures—The generalized load-deformation relations shown in Fig. 3.1.2.2.3 are described by linear response from Point A (unloaded component) to an effective yield Point B, then a linear response at reduced stiffness from Point B to C, then reduction in lateral strength to Point D, then response at residual strength P_{res} from Point D, and loss of the residual strength or gravity load-carrying capacity

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thereafter. Typically, the response shown in Fig. 3.1.2.2.3 is associated with flexural response or tension response. In this case, the resistance at $Q/Q_{yE} = 1.0$ is the yield value, and subsequent strain hardening is accommodated by hardening in the load-deformation relation as the member is deformed toward the expected strength. Where the response shown in Fig. 3.1.2.2.3 is associated with compression, the resistance at $Q/Q_{yE} = 1.0$ typically is the value where concrete begins to spall, and strain hardening in well-confined sections can be associated with strain hardening of the longitudinal reinforcement and an increase in strength from the confinement of concrete. Where the response shown in Fig. 3.1.2.2.3 is associated with shear, the resistance at $Q/Q_{yE} = 1.0$ typically is the value at which the design shear strength is reached and, typically, no strain hardening follows.

The deformations used for the load-deformation relation of Fig. 3.1.2.2.3 should be defined in one of two ways, as follows:

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

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

Provisions for determining alternative modeling parameters and acceptance criteria based on experimental evidence are given in [ASCE 41](#), 7.6.

Displacement demands determined from nonlinear dynamic analysis are sensitive to the rate of strength degradation included in the structural model. However, the available experimental results for establishing the strength degradation relationship are limited and exhibit wide ranges of behavior. 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 Points C to D in Fig. 3.1.2.2.3 can result in overestimation of the drift demands for a structural system and individual components. A more realistic model for many concrete components has a linear degradation in resistance from Points C to E, as shown in Fig. 3.1.2.2.3 with a dashed line.

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 3.1.2.2.3 does not distinguish between these types of strength degradation and may not accurately predict

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the displacement demands if the two forms of strength degradation are not properly considered.

Common shapes of the hysteresis under force or deformation reversals shown in Fig. R3.1.2.2a illustrate stiffness degradations and energy dissipation under cyclic loading. Type A represents the behavior of the components with low pinching such as columns subjected to low axial loads adequately detailed for toughness. Type B represents the behavior of the components with moderate pinching such as a column under moderate axial load. Type C represents the hysteretic behavior of the components with significant pinching such as a poorly detailed column with high axial load or the components sustaining sliding shear or splice failures.

Some commercially available analysis software has features that enable calculating the fraction of the total ground motion input energy dissipated through different sources, such as inelastic hysteretic behavior of deformation-controlled actions and structural damping. Where this information is available, it should be compared with characteristic values reported in the literature to evaluate the adequacy of the model.

Nonlinear fiber elements formulated solely on the basis of stress-strain relationships directly obtained from tensile and compressive tests of axially loaded reinforcing steel

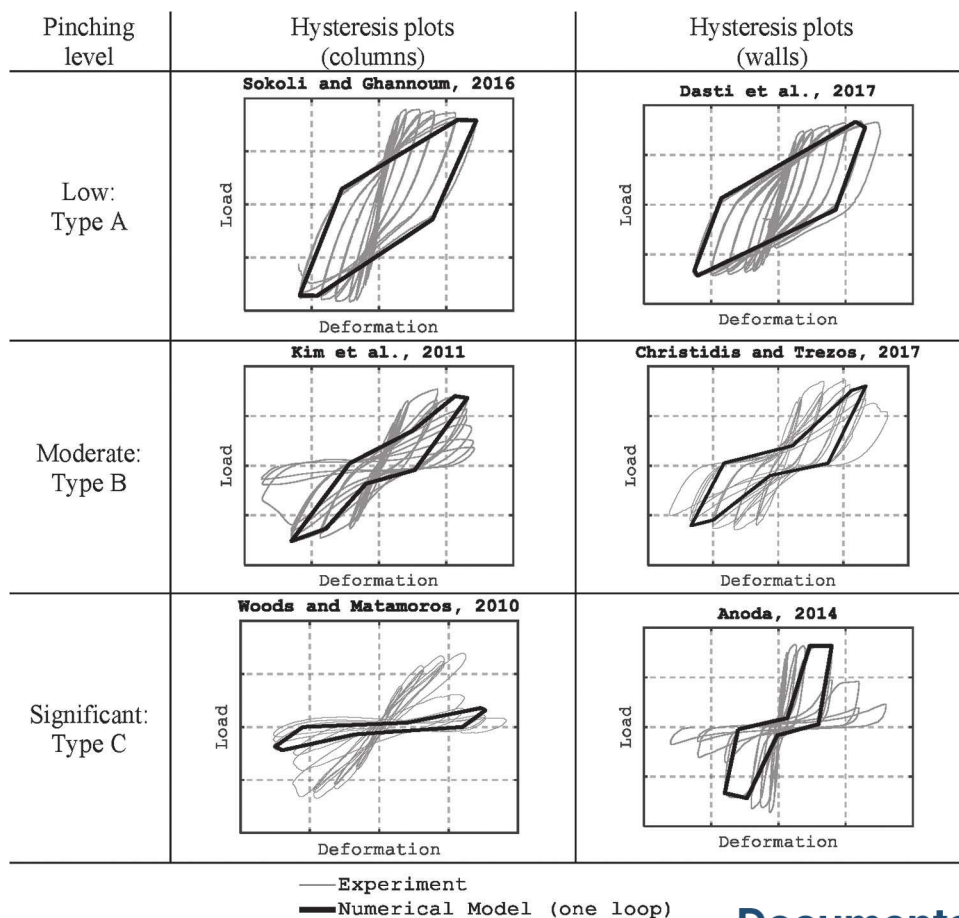


Fig. R3.1.2.2a—General hysteresis types.

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coupons or concrete cylinders may not completely represent the nonlinear behavior of reinforced concrete components because uniaxial material tests do not include phenomena such as bar slip, bar buckling, cover spalling, and cyclic response. Because the objective is to accurately reproduce member response, stress-strain relations and mesh configuration of fiber elements should be adjusted to verify that the calculated response obtained with the numerical model produces action-deformation envelope curves similar to those obtained with the modeling parameters provided in this code or obtained from experimental data. [Pugh et al. \(2017\)](#), [Marafi et al. \(2019\)](#), [Panagiotou et al. \(2012\)](#), and [Lowes et al. \(2016\)](#) provide guidance on regularization of concrete and steel material models to minimize mesh sensitivity in models that comprise solid elements, layered and fiber shell elements, and beam-column elements with fiber-type sections.

Fiber-type section models are often used to simulate the nonlinear cyclic response of concrete members controlled by flexure. Fiber-section models provide explicit simulation of the impact of axial load variation on component response and are appropriate for use for columns and walls for which variation in axial load during earthquake loading may alter strength, stiffness, and deformation capacity. When constructing these models, the cross section of a structural component is discretized using fibers with constitutive behavior representative of unconfined and confined concrete, as well as reinforcing steel bars. Fiber-section models should reproduce the action-deformation response of components to produce numerical models that accurately simulate the response of the system. To that effect, even though fiber-section models are calibrated to accurately reproduce component behavior, local responses such as curvatures or material strains may not be adequately reproduced.

At hinge locations, axial deformation and curvature of each fiber section along the length of the hinge are multiplied by an integration length, which is often an assumed plastic hinge length, to compute an axial load-axial deformation or moment-rotation response history, respectively. To obtain a computed response that is in substantial agreement with the results of physical tests, the fiber discretization of a section should be sufficient to represent the stress and strain field at the section level, and concrete and reinforcement material models should represent characteristic features of the cyclic material stress-strain histories, where material response includes strain softening. The softening portion of the stress-strain response history should be defined accounting for the energy dissipated during softening and the integration length associated with the fiber section.

Fiber-type section models may be embedded in zero-length hinge elements, in finite-length hinge elements, and in beam-column elements with integration schemes that assume distributed or lumped plasticity. For components for which deformations due to response modes other than flexural and axial response modes are significant, it is necessary to combine the fiber-type models with models that simulate

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the other significant deformation modes such as shear and reinforcing bar slippage.

A limitation of typical fiber-type section models, which may significantly affect simulation results, is the assumption that plane sections remain plane and perpendicular to the neutral axis. Using this assumption, it is not possible to directly simulate flexure-shear interaction or deformation associated with slip of anchored longitudinal reinforcement.

Nonlinear axial deformations of fiber sections may not be compatible with those in lumped plastic hinges with axial force and bending moment interaction. It is not recommended for the design professional to embed a frame element with lumped plastic hinges in a fiber-type model for the simulation of an embedded column or pilaster in a wall unless force-deformation behavior of the wall-column assembly model is consistent with modeling requirements in [Chapter 7](#) for walls or experimental data.

Within this code, rotation is the primary measure used to define component response and acceptance criteria. When the nonlinear response of a structure is simulated using a modeling approach other than a concentrated hinge, defined by a moment-rotation or story shear-deflection history, it is necessary to convert between the rotations specified in this code and the deformation measures employed within the model. To make these conversions, it is recommended that the component rotations specified within this code be compared with a rotation computed using: a) curvature at the critical section and, as needed, curvature computed at additional sections; and b) a hinge length (l_p) equivalent to that specified within this code or justified by experimental evidence for the hinge rotation being considered. Figure R3.1.2.2b shows an example of this conversion for a wall modeled using multiple approaches.

[NIST GCR-17-917-45](#) and [GCR17-917-46](#) also provide state-of-the-art recommendations for nonlinear modeling of structural elements where nonlinear response is highly dependent on the loading protocol, especially for large inelastic deformation demands, for which adaptive hyster-

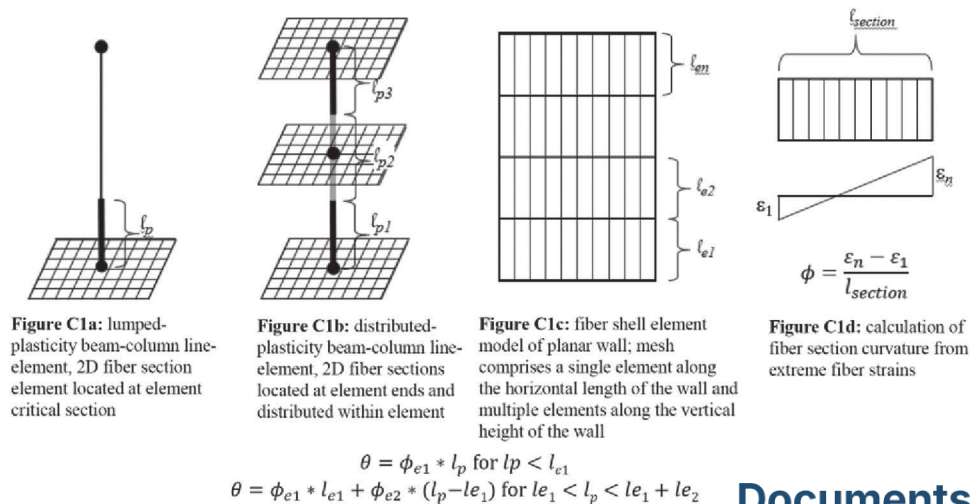


Fig. R3.1.2.2b—Nonlinear wall modeling options.



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3.1.2.2.1 Where design actions are determined using the nonlinear procedures of **ASCE 41** Chapter 7, component load-deformation response shall be modeled using nonlinear load-deformation relations for each deformation-controlled action. These relations shall include the effective stiffness, expected strength, deformation capacity, and hysteretic response under force or deformation reversals.

3.1.2.2.2 Linear relations shall be permitted if the expected yield strength of the component is not exceeded in each ground motion analysis.

3.1.2.2.3 The nonlinear load-deformation relation shall be based on experimental evidence defined by quantities specified in **Chapters 4** through **12**. For the nonlinear static procedure (NSP), the generalized load-deformation relation shown in Fig. 3.1.2.2.3 shall be used. For the nonlinear dynamic procedure (NDP), the generalized load-deformation relations shown in Fig. 3.1.2.2.3 shall be combined with the general hysteresis shapes specified in 3.1.2.2.8.

Alternatively, it shall be permitted to base the nonlinear load-deformation relation derived from laboratory test data for components or subassemblages: a) subjected to gravity load effects and lateral load or deformation histories similar to those expected for building components; and b) exhibiting response modes similar to those expected for building components. Where experimental data sets are used to define nonlinear action-deformation relations, simulated analytical deformation demands shall not exceed the maximum tested deformation imposed on the component or subassemblages used for model calibration.

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etic models are recommended, provided that implementation and use of an adaptive model are practical and feasible in analysis software. Modeling parameters in this code are based on the experimental data of components subjected to cyclic loading and may underestimate energy dissipation and deformation capacities of deformation-controlled members subjected to limited load or deformation reversals.

For columns or walls under the combined effects of axial load and bidirectional lateral loads, the ratio of deformation demand to deformation capacity in the acceptance criterion of the principal directions should be combined using an appropriate combination rule such as square root of the sum of the squares. Criteria to account for the interaction of shear forces acting along orthogonal axes is provided in 22.5.1 of **ACI 318**.

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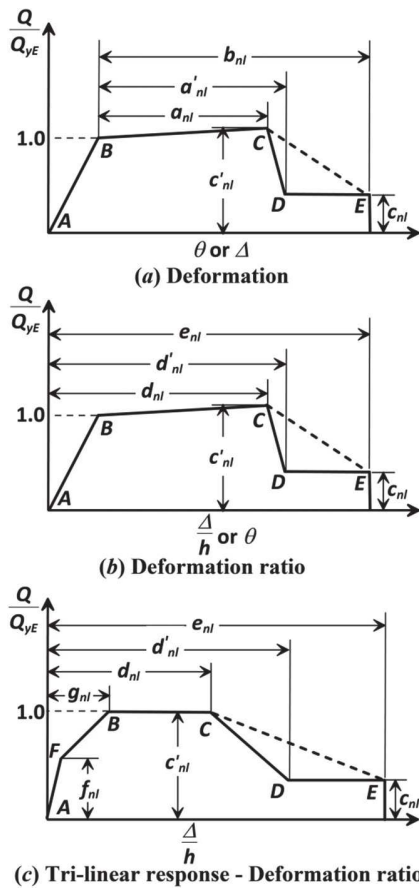


Fig. 3.1.2.2.3—Generalized force-deformation relation for concrete elements or components.

3.1.2.2.4 The slope from Point A to B in Fig. 3.1.2.2.3(a) and (b) or Point A to F in Fig. 3.1.2.2.3(c) shall be determined according to Table 3.1.2.2.4 or determined using a secant-to-yield stiffness based on experimental data. Point B shall have an ordinate equal to the expected yield strength of the component.

R3.1.2.2.4 The effective stiffness coefficient for beams in Table 3.1.2.2.4 was inferred from test data and was calibrated to provide the mean secant stiffness to yield of beam subassemblies subjected to cyclic loading. It is intended for use with the nonlinear analysis procedures (NSP and NDP). The effective stiffness coefficient for beams in Table 3.1.2.2.4 was not calibrated to produce estimates of drift demand of linear systems similar to those calculated using nonlinear dynamic analyses (Sozen 2003).

Govindan (2018) performed a regression analysis of data from 58 beam tests and found that the effective stiffness coefficient of elements with axial load ratios $P/f'_c A_g$ between 0 and 0.125 had a mean of 0.22 with a coefficient of variation of 35%. In the analysis by Govindan, an effective stiffness coefficient of 0.20 corresponded to the 45th percentile of the test data, a coefficient of 0.25 corresponded to the 64th percentile, and 81% of beams had an effective stiffness coefficient lower than 0.3. Effective stiffness in the regression analysis performed by Govindan was defined on the basis of the secant to the yield point of experimental data, defined by extrapolating a secant to the deformation at 75% of yield to the yield moment.

Owing to the manner in which it was derived, the proposed effective stiffness coefficient for beams includes flexibilities related to flexure, shear, and reinforcement slip.

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Table 3.1.2.2.4—Effective stiffness values for nonlinear analysis*

Component	Axial rigidity	Flexural rigidity	Shear rigidity
Beams—nonprestressed [†]	$1.0E_{cE}A_g$	$0.2E_{cE}I_g$	$0.4E_{cE}A_w$
Beams—prestressed [†]	$1.0E_{cE}A_g$	$1.0E_{cE}I_g$	$0.4E_{cE}A_w$
Columns with compression caused by design gravity loads $\geq 0.5A_gf_c^{* \dagger}$	$1.0E_{cE}A_g$	$0.7E_{cE}I_g$	$0.4E_{cE}A_w$
Columns with compression caused by design gravity loads $\leq 0.1A_gf_c'$ or with tension [‡]	$1.0E_{cE}A_g$ (compression) $1.0E_{cE}A_s$ (tension)	$0.2E_{cE}I_g$	$0.4E_{cE}A_w$
Diaphragms (in-plane)—nonprestressed [§]	$0.25E_{cE}A_g$	$0.25E_{cE}I_g$	$0.25E_{cE}A_g$
Diaphragms (in-plane)—prestressed [§]	$0.5E_{cE}A_g$	$0.5E_{cE}I_g$	$0.4E_{cE}A_g$
Walls—uncracked with compression caused by design gravity loads $\geq 0.3A_gf_{cE}^{* \dagger, \#}$	$1.0E_{cE}A_g$	$1.0E_{cE}I_g$	$0.4E_{cE}A_w$
Walls—uncracked with compression caused by design gravity loads $\leq 0.05A_gf_{cE}'$ or with tension ^{‡, \#}	$1.0E_{cE}A_g$	$0.5E_{cE}I_g$	$0.4E_{cE}A_w$
Walls—cracked ^{#, \parallel}	$1.0E_{cE}A_g$ (compression) $1.0E_{cE}A_s$ (tension)	$0.25E_{cE}I_g$	$0.15E_{cE}A_w$
Coupling beams with longitudinal or diagonal reinforcement	$1.0E_{cE}A_g$	$0.05(I_n/h)E_{cE}I_g$ $\leq 0.20E_{cE}I_g$	$0.2E_{cE}A_w$

*Tabulated values for axial, flexural, and shear shall be applied jointly in defining effective stiffness of an element, unless alternative combinations are justified. For other elements not covered in this table, it shall be permitted to use values in Table 3.1.2.1.

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

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

[§]In-plane diaphragm effective stiffness values apply where diaphragm flexibility is considered in accordance with 10.2.2.

[#]Walls are permitted to be considered cracked due to earthquake demands in flexural actions where flexural demands exceed M_{crE} , in shear actions where shear stress demands exceed $2\sqrt{f_{cE}'}$, or both. It shall be permitted to assume all walls to be cracked.

^{\parallel}Alternative stiffness values dependent on axial load and boundary longitudinal reinforcement ratio shall be permitted in accordance with 7.4.

Calculated deformation related to shear at yield for the beam data set compiled by Govindan (2018) based on an elastic shear modulus equal to $0.4E_{cE}$ were very small compared with the total deformation at yield, ranging between 0.4 and 2.1% of the chord rotation at yield, with a mean of approximately 1.2%.

3.1.2.2.5 Point C in Fig. 3.1.2.2.3 shall have an ordinate equal to the expected strength of the component, including the effect of strain-hardening, and an abscissa equal to the generalized deformation at which significant strength degradation begins.

3.1.2.2.6 Point D in Fig. 3.1.2.2.3 shall have an ordinate equal to the residual strength of the component and an abscissa equal to the generalized deformation at which the residual strength is reached.

Point E in Fig. 3.1.2.2.3 shall have an ordinate equal to the residual strength of the component. For actions in which failure results in structural collapse, the abscissa shall be equal to the generalized deformation at which the component loses the ability to sustain gravity loads. For other actions, the abscissa shall be equal to the generalized deformation at which the component loses its residual strength. If Point D is unknown, connecting Points C and E by a straight line shall be permitted, as illustrated in Fig. 3.1.2.2.3 with a dashed line.

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3.1.2.2.7 General deformation values for the points identified in Fig. 3.1.2.2.3 shall be as specified in 4.2.2.2 and 4.2.2.3 for beams, columns, and joints; 4.3.2.2 and 4.3.2.3 for post-tensioned beams; 4.4.2.2 and 4.4.2.3 for slab-column connections; 7.4.1 for structural walls and wall segments; and 7.6.1 for coupling beams. Other load-deformation relations shall be permitted if justified by experimental evidence.

For analyses using the nonlinear dynamic procedure (NDP), the general hysteretic shape for all components shall be defined in 3.1.2.2.8.

3.1.2.2.8 The following general hysteresis shapes shall be used for analyses with the nonlinear dynamic procedure (NDP):

- a) Type A: Hysteresis shape representing the behavior of the components with low pinching
- b) Type B: Hysteresis shape representing the behavior of the components with moderate pinching
- c) Type C: Hysteresis shape representing the behavior of the components with significant pinching

3.1.2.2.9 If structural components are modeled using lumped plasticity or distributed plasticity models, the force-deformation or stress-strain relationships shall be adjusted to achieve component-expected behavior based on the assumed plastic hinge length or integration length used in the analysis.

3.1.2.2.10 Nonlinear fiber-type section models shall be shown to result in the calculation of load-deformation response that is in substantial agreement with the results of physical tests of reinforced concrete components or subassemblages exhibiting response mechanisms consistent with those expected in the components or subassemblages being modeled.

Where simulation results generated using fiber-type section models cannot be validated using experimental data, the stress-strain relationships or meshing of the fiber elements that compose the fiber-type section models shall be modified such that the predicted response is in substantial agreement with the generalized load-deformation values in 3.1.2.2.7.

3.1.2.2.11 For components under combined axial load and bidirectional lateral load, the effect of the combined loading on the strength and deformation capacity of the component shall be 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 shall be calculated considering a width of effective flange on each side of the web equal to the smallest of:

- a) The provided flange width
- b) Eight times the flange thickness
- c) Half the distance to the next web

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d) One-fifth the beam span length

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

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

3.2—Strength and deformability

3.2.1 General—Actions in a structure shall be classified as being either deformation-controlled or force-controlled. Deformation-controlled actions are defined by the designation of linear and nonlinear acceptance criteria in Table 4.2.2.2.2a, Table 4.2.2.2.2b, Table 4.2.2.2.2c, Table 4.2.3.2.2, Table 4.2.4.1.2b, Table 4.2.4.1.2c, Table 4.2.4.1.2d, Table 4.4.2.2, Table 4.4.4.1, Table 6.2.2.2, Table 6.2.4.1, Table 7.3.2b, Table 7.3.2c, Table 7.3.2d, Table 7.3.2e, Table 7.4.1.1.1a, Table 7.4.1.1.1b, Table 7.4.1.1.2, Table 7.5.2a, Table 7.5.2b, Table 7.6.2a, and Table 7.6.2b. Where linear and nonlinear acceptance criteria are not specified in the tables, actions shall be taken as force-controlled unless component testing is performed in accordance with [ASCE 41](#), 7.6. Strengths for deformation-controlled and force-controlled actions shall be calculated in accordance with 3.2.2 and 3.2.3, respectively.

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

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

3.2.2 Deformation-controlled actions—Strengths used for deformation-controlled actions shall be taken as equal to expected strengths Q_{CE} obtained experimentally or calculated using accepted principles of mechanics. Unless specified in this code, other procedures specified in [ACI 318](#) to calculate strengths shall be permitted, except that the strength reduction factor ϕ shall be taken equal to unity. Deformation capacities for acceptance of deformation-controlled actions calculated by nonlinear procedures shall be as specified in [Chapters 4 through 12](#) of this code. For components constructed of lightweight concrete, Q_{CE} shall be modified in accordance with [ACI 318](#) procedures for lightweight concrete.

3.2.3 Force-controlled actions—Strengths used for force-controlled actions shall be taken as lower-bound strengths Q_{CL} , obtained experimentally or calculated using established principles of mechanics. Lower-bound strength is defined as the mean less one standard deviation of resistance expected

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R3.2—Strength and deformability

R3.2.1 General—In this code, actions are classified as either deformation-controlled or force-controlled. Actions are considered deformation-controlled where the component behavior is well documented by test results. Where linear or nonlinear acceptance criteria are tabulated in this code, the committee has judged the action to be deformation-controlled and expected material properties should be used. Where such acceptance criteria are not specified, the action should be assumed force-controlled, thereby requiring the use of lower-bound material properties, or the licensed design professional can opt to perform testing to validate the classification of deformation-controlled. [ASCE 41](#), 7.6, provides guidance on procedures to be followed during testing, and [ASCE 41](#), 7.5.1.2, provides a methodology based on the test data to distinguish force-controlled from deformation-controlled actions. Further guidance on the testing of moment-frame components can be found in [ACI 374.1](#).

R3.2.2 Deformation-controlled actions—Expected yield strength of steel reinforcement, as specified in [2.2.1.2](#), includes material overstrength considerations.

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over the range of deformations and loading cycles to which the concrete component is likely to be subjected. Where calculations are used to define lower-bound strengths, lower-bound estimates of material properties shall be used. Unless other procedures are specified in this code, procedures specified in **ACI 318** to calculate strengths shall be permitted, except that the strength reduction factor ϕ shall be taken equal to unity. For components constructed of lightweight concrete, Q_{CL} shall be modified in accordance with ACI 318 procedures for lightweight concrete.

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

Table 3.2.4—Component ductility demand classification

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

3.3—Flexure and axial loads

3.3.1 Flexural strength—Flexural strength of members with and without axial loads shall be calculated according to ACI 318 or by other demonstrated rational methods, such as sectional analysis using appropriate concrete and steel constitutive models.

3.3.2 Deformation capacity—Deformation capacity of members with and without axial loads shall be calculated considering shear, flexure, and reinforcement slip deformations, or based on acceptance criteria given in this code.

3.3.3 Components with monolithic flanges—Strengths and deformation capacities of components with monolithic flanges shall be calculated considering concrete and developed longitudinal reinforcement within the effective flange width, as defined in 3.1.3.

3.3.4 Components with inadequate development length—Strength and deformation capacities shall be determined based on the available development of longitudinal reinforcement. Where longitudinal reinforcement has embedment or development length that is insufficient for reinforcement strength development, flexural strength shall be calculated based on limiting stress capacity of the embedded bar as defined in 3.5.

3.3.5 Effect of shear on component strength and deformation capacity—Where flexural deformation capacities

R3.3—Flexure and axial loads

Where flexural strength of an axially loaded member is calculated in the linear procedure, compressive load should be considered in this case as a force-controlled action due to its nonductile nature, whereas tensile load should be considered in this case as a deformation-controlled action because the tensile actions are controlled by reinforcement yielding in tension. The m -factor for the flexural behavior can be conservatively used to estimate the deformation-controlled action due to the tension.

R3.3.5 Effect of shear on component strength and deformation capacity—Laboratory tests indicate that flexural

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

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

3.3.7 Usable strain limits

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

R3.3.6 Flexure-axial load interaction—The combined strength under uniaxial or biaxial bending with axial load is difficult to generalize in a closed-form solution, given the range of column section geometries encountered. For a particular class of rectangular column sections, closed-form solutions based on section capacities about the principal axes have been developed that provide excellent agreement when compared to a more generalized analysis ([Hsu 1988](#); [Furlong et al. 2004](#)). A circular envelope provides a poor prediction of the strength for all but circular columns. For general sections, the strength envelope should be developed based on principles of mechanics.

R3.3.7 Usable strain limits—Early research on the stress-strain behavior of unconfined concrete ([Hognestad 1952](#)) has shown that the stress-strain behavior of concrete is different in members subjected to flexure than in members subjected to nearly pure compression. Concrete subjected to concentric compression exhibits crushing shortly after the maximum stress is reached at strains of approximately 0.0015 to 0.0020 ([Hognestad 1952](#)), whereas crushing in the extreme compression fiber of members subjected to flexure and axial load is observed at higher strains, ranging between 0.003 and 0.005 ([Hognestad 1952](#)). The maximum usable strain limits established in this section are intended to caution licensed design professionals when using stress-strain relationships for concrete to calculate moment and axial strengths. In members subjected to nearly pure compression, redistribution of stresses within the compression zone after the strain in the concrete exceeds the strain corresponding to peak stress (0.0015 to 0.0020 for unconfined concrete) ([Hognestad 1952](#)) is not possible because most of the concrete in the cross section will be on the descending branch of the stress-strain curve for concrete.

Usable strain limits specified in this section do not preclude licensed design professionals from using the provisions in 22.2 of [ACI 318](#). Section 22.2.2.1 of [ACI 318](#) stipulates that to calculate the moment and axial strength of reinforced concrete members, the maximum usable strain in the extreme compression fiber of reinforced concrete shall be assumed to be 0.003. This usable strain is within the limit of 0.005 specified in 3.3.7.1 herein. In the case of members subjected to nearly pure compression, provisions in 22.4.2 of [ACI 318](#) stipulate that the design axial strength

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of columns with unconfined concrete shall not exceed 80% of the nominal axial strength. According to the commentary of 22.4.2.1 of **ACI 318**, the reduced nominal axial strength corresponds to a minimum eccentricity of 5% of the column depth. The usable strain limit of 0.002 specified in 3.3.7.1 herein is intended to prevent overestimating the flexural strength of columns with very small eccentricities, so the provisions in 22.4.2.1 of ACI 318 can be used in place of calculating the axial and moment strength based on stress-strain models for concrete.

While provisions in 21.2.2 of ACI 318 establish that, for tension-controlled members, the strain in the reinforcement at failure shall be at least 0.003 greater than the yield strain, there is no upper limit in the Code for the usable strain in the reinforcement of beams and columns. Although an upper limit in the strain at failure of beams and columns is implied in the provisions for minimum reinforcement in 9.6 and 10.6 of ACI 318, those limits are not intended for members that will be subjected to deformation cycles in the nonlinear range of response. The reinforcement tensile strain limit in 3.3.7.3 herein is based on consideration of the effects of material properties and low-cycle fatigue. Low-cycle fatigue is influenced by spacing and size of transverse reinforcement and strain history. Using extrapolated monotonic test results to develop tensile strains greater than those previously specified is not recommended. **Caltrans (2006)** recommends an ultimate tensile strain of 0.09 for No. 10 bars and smaller, and 0.06 for No. 11 bars and larger, for **ASTM A706/A706M** 60 kip/in.² reinforcing bars. A lower bound is selected herein considering the variability in materials and details typically found in existing structures.

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

3.3.7.1 Usable strain limits for unconfined concrete—

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

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

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

3.3.7.2 Usable strain limits for confined concrete—

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

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concrete, it shall be permitted to adopt usable strain limits for unconfined concrete.

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

3.4—Shear and torsion

3.4.1 Strength—Strengths in shear and torsion shall be calculated according to ACI 318, except as modified in this code.

3.4.2 Strength of components with widely spaced reinforcement—Unless otherwise noted, where the longitudinal spacing of transverse reinforcement exceeds half the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed to have reduced effectiveness in resisting shear or torsion by a factor of $2(1 - s/d)$. Where the longitudinal spacing of transverse reinforcement exceeds the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed ineffective in resisting shear or torsion.

3.4.3 Components with lapped spliced transverse reinforcement—For beams and columns, lap-spliced transverse reinforcement shall be assumed not more than 50% effective in regions of moderate ductility demand and ineffective in regions of high ductility demand, and applies in addition to the effectiveness factor due to spacing.

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

3.5—Development and splices of reinforcement

Development of straight bars, hooked bars, and lap-spliced bars shall be calculated according to the provisions of ACI 318, with the modifications in 3.5.1 to 3.5.8.

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

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R3.4—Shear and torsion

Strength estimates in shear and torsion for seismic assessment are intended to provide a best estimate of component capacity. Shear strength equations for plastic hinge regions of beams in Chapter 18 of ACI 318 provide a conservative assessment of shear strength as is appropriate for new design. For assessment intended to simulate expected response, it is more appropriate to use the shear strength models of ACI 318, Chapter 22, even in plastic hinge regions, to avoid skewing of analytical results and provide the most accurate representation of force distribution. It is noted that there is a potential for concrete beam shear strength to degrade with increased plastic rotation demands, especially in beam hinge regions in the presence of large flexural cracks and axial elongation. The degradation in strength is indirectly captured in nonlinear analysis based on the defined α -parameter (Priestley et al. 1994; Sezen and Moehle 2004).

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

R3.5—Development and splices of reinforcement

Development requirements in accordance with Chapter 25 of ACI 318 are applicable to development of bars in all components. Chapter 18 of ACI 318 provides development requirements that are intended only for use in yielding components of reinforced concrete moment frames that comply with the cover and confinement provisions of Chapter 18 of ACI 318. Chapter 25 of ACI 318 permits reductions in lengths if minimum cover and confinement are present in an existing component. For additional information



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

b) For columns where deformed straight and lap-spliced bars pass through regions where inelastic deformations and damage are expected, the bar length within those regions shall be considered effective for anchorage only until inelastic deformations occur. In such cases, the development length obtained using **ACI 318** procedures shall be compared with a degraded available development length ℓ_{b-deg} , as defined in 3.5.3.

3.5.2 Anchorage and bond strength of deformed bars with inadequate development or splice length—Where existing deformed straight bars, hooked bars, and lap-spliced bars do not meet the development requirements of (3.5.1), the capacity of existing reinforcement shall be calculated as follows

$$f_s = 1.25 \left(\frac{\ell_b}{\ell_d} \right)^{2/3} f_{yL/E} \leq f_{yL/E} \quad (3.5.2)$$

If the maximum applied bar stress is larger than f_s given in Eq. (3.5.2), members shall be deemed controlled by inadequate development or splicing.

3.5.3 Anchorage and bond strength of deformed bars in plastic hinge regions—For columns where deformed straight and lap-spliced longitudinal bars pass through regions where inelastic deformations and damage are expected, the bar length within those regions shall be considered effective for anchorage only until inelastic deformations occur. In such cases, if $f_s = f_{yL/E}$ from Eq. (3.5.2), the degraded reinforcement capacity f_{s-deg} accounting for the loss of anchorage in the damaged region shall be evaluated using a degraded available development length (ℓ_{b-deg}). ℓ_{b-deg} shall be evaluated by subtracting from ℓ_b a distance of $2/3d$ from the point of maximum flexural demand in any direction damage is anticipated within the column.

$$f_{s-deg} = 1.25 \left(\frac{\ell_{b-deg}}{\ell_d} \right)^{2/3} f_{yL/E} \leq f_{yL/E} \quad (3.5.3)$$

In cases where $f_s = f_{yL/E}$ from Eq. (3.5.2) but the maximum applied longitudinal bar stress is larger than f_{s-deg} given in Eq. (3.5.3), columns shall be deemed controlled by inadequate development or splicing and the capacity of the existing reinforcement shall be taken as $f_{yL/E}$.

3.5.4 Strength and deformation capacity of components that have deformed bars with inadequate development length for nonlinear procedures—For inadequate development or splicing of straight bars in beams and columns: for nonlinear procedures, it shall be permitted to assume that the

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on development and lap splices, refer to **ACI 408R**, and for hooked anchorage, refer to **Sperry et al. (2005)**.

Equation (3.5.2), which is a modified version of the model presented by **Cho and Pincheira (2006)**, reflects the intent of ACI 318 development and splice equations to develop 1.25 times the nominal bar strength, referred to in this code as the expected yield strength. The nonlinear relation between developed stress and development length reflects the effect of increasing slip and, hence, reduced unit bond strength, for longer development lengths. Refer to **Elwood et al. (2007)** for more details.

Bond strength can be significantly curtailed in damaged regions within plastic hinges (**Sokoli and Ghannoum 2016; Ichinose 1995**). The length where bond capacity is curtailed during inelastic deformations is recommended to be two-thirds of the section effective depth d (Sokoli and Ghannoum 2016). If f_s , evaluated using Eq. (3.5.2), equals $f_{yL/E}$, then bond failure is not expected prior to inelastic hinging and the bar under consideration can be expected to resist the full yield stress $f_{yL/E}$. However, f_s should be re-evaluated using a degraded effective anchorage length ℓ_{b-deg} using Eq. (3.5.3), which is reduced by the bar length within the region expected to be damaged. If f_{s-deg} remains equal to $f_{yL/E}$, even after the anchorage length is reduced, then no anchorage failure is expected even during inelastic deformations. If, however, f_{s-deg} becomes smaller than $f_{yL/E}$ when the available anchorage length is reduced, then anchorage failure is expected, but only after inelastic deformations occur. In such cases, the limiting stress in longitudinal bars will be $f_{yL/E}$ but the modeling parameters in Tables 4.2.2.2.2b and 4.2.2.2.2c for columns with inadequate development or splicing should be used.

For buildings constructed before 1950, the bond strength developed between steel reinforcement and concrete can be less than present-day strength (**Clark 1949; Howell and Higgins 2007; Feldman and Cairns 2017**). 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 is much greater than for deformed bars and more sensitive to cracking in concrete. Testing and assessment procedures for tensile lap splices and development length for plain steel reinforcement are found in **Concrete Reinforcing Steel Institute (1981)**.

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reinforcement retains the calculated maximum stress evaluated using Eq. (3.5.2) up to the deformation levels defined by a_{nl} in Tables 4.2.2.2.2a, 4.2.2.2.2b, and 4.2.2.2.2c.

3.5.5 Strength and deformation capacity of components that have deformed bars with inadequate development length for linear procedures—For linear procedures, the calculated maximum stress evaluated using Eq. (3.5.2) shall be used for strength calculations. For members other than beams and columns controlled by inadequate development or splicing and hooked anchorage, the developed stress shall be assumed to degrade from $1.0f_s$, at a ductility demand or DCR equal to 1.0, to $0.2f_s$ at a ductility demand or DCR equal to 2.0.

The strength of deformed straight, discontinuous bars embedded in concrete sections or beam-column joints, with clear cover over the embedded bar not less than $3d_b$, shall be calculated according to Eq. (3.5.5)

$$f_s = \frac{2500}{d_b} \ell_e \leq f_{yL/E} \quad (\text{psi}) \quad (3.5.5)$$

where f_s is less than $f_{yL/E}$ and the calculated stress in the bar caused by design loads equals or exceeds f_s , the maximum developed stress shall be assumed to degrade from $1.0f_s$, at a ductility demand or DCR equal to 1.0, to $0.2f_s$ at a ductility demand or DCR equal to 2.0. In beams with bottom bar embedment length into beam-column joints less than the requirements of ACI 318, flexural strength shall be calculated considering the stress limitation of Eq. (3.5.5).

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

3.5.7 Anchorage and bond strength of doweled bars—Doweled bars added in seismic retrofit shall be assumed to develop yield stress where all the following conditions are satisfied:

- a) Drilled holes for dowel bars are cleaned
- 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 shall be verified by test data. Field samples shall be obtained to ensure that design strengths are developed in accordance with Chapter 3.

3.5.8 Anchorage and bond strength of square bars—Square reinforcing bars in a building should be classified as either twisted or straight. The developed strength of twisted square bars shall be as specified for deformed bars in this section, using an effective diameter calculated based on the

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area of the square bar. Straight square bars shall be considered plain bars, and the developed strength shall be as specified in 3.5.6.

3.6—Connections to existing concrete

Connections used to connect two or more components shall be classified according to their anchoring systems as cast-in-place or as post-installed and shall be evaluated and designed according to Chapter 17 of **ACI 318** as modified in this section. The properties of the existing anchors and connection systems shall be obtained in accordance with 3.6.3. These provisions do not apply to connections in plastic hinge zones.

The strength of new cast-in-place or post-installed anchors shall be computed using the procedures of Chapter 17 of ACI 318 using a strength reduction factor, ϕ , equal to 1.0. Special inspection of the installation of the anchors shall be in accordance with Chapter 17 of the **2018 IBC**. For post-installed anchors, requirements from the anchor manufacturer shall also apply.

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R3.6—Connections to existing concrete

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

Capacities of existing anchors should be evaluated based on the properties obtained in accordance with 3.6.3 and Chapter 17 of ACI 318. If the anchors are not tested to failure but to a load based on the force-controlled action determined by the licensed design professional for the seismic hazard under consideration, the procedure in Chapter 17 of ACI 318 can be used to calculate available strength based on the test results and the geometry of anchors measured or assumed by the licensed design professional.

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

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

Not all manufacturers of post-installed anchors publish information on the mean and the standard deviation of the ultimate anchor capacity. Older testing for existing post-installed anchors is often reported at allowable stress design levels and may not comply with the requirements of Chapter 17 of **ACI 318** for simulated seismic tests. Care and

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

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

3.6.3 Proof load testing—Anchor testing shall be in general conformance with **ASTM E3121/E3121M**, adapted as necessary for on-site testing. Testing of anchors to failure is not required. Proof loading of mechanical anchors and cast-in-place anchors shall be conducted as an unconfined test as prescribed by ASTM E3121/E3121M. For adhesive anchors, the proof loading is permitted to be conducted as a confined test according to ASTM E3121/E3121M.

Where application of a proof load is not feasible without removal of the structural framing to which the anchors are attached, torque testing of mechanical and cast-in-place anchors shall be permitted. The relationship of torque to bolt tension shall be established by the evaluating licensed design professional and approved by the licensed design professional based on testing of a representative anchor to which the applied torque and tension are measured.

3.6.3.1 The proof load in tension shall be specified by the licensed design professional but shall not exceed the lesser of the following:

- a) 1.33 times the maximum tension value of Q_{UF} for an anchor in a common anchorage connection or 1.33 times the maximum calculated tensile capacity for a single anchor of the set
- b) $2/3N_{cb}$

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judgment should be used in determining pullout strength for anchors, particularly those that are critical to satisfying the target performance level.

R3.6.1 Cast-in-place anchors and connection systems—The strength reduction factor ϕ in ACI 318 is taken equal to unity for the lower-bound connection strength of structural components, but the requirements in 17.2.3 of ACI 318 need to be satisfied, including the reduction of the strength due to cracked concrete and cyclic loading. The component actions on the anchors and connection systems for structural components are considered as force-controlled actions according to 7.5.2 and 7.5.3 of **ASCE 41**, so further amplification of the seismic demand is not necessary.

R3.6.2 Post-installed anchors—The strength reduction factor ϕ in ACI 318 is taken equal to unity for the lower-bound connection strength of structural components, but the requirements in 17.2.3 of ACI 318 need to be satisfied, including the reduction of the strength due to cracked concrete and cyclic loading. The component actions on post-installed anchors for structural components are considered force-controlled actions according to 7.5.2 and 7.5.3 of **ASCE 41**, so further amplification of the seismic demand is not necessary.

R3.6.3 Proof load testing—Testing of existing anchors is intended to determine a mean characteristic tensile strength for the anchors. Where feasible, tension tests should be set up as unconfined tension tests. The proof load testing should not damage the anchor because the proof load magnitude is intended only to verify that the anchors are capable of resisting the required design tension force.

It is recognized that some existing installations do not easily allow for an unconfined test setup. For adhesive anchors, a bond failure often governs the tensile failure mode so a confined test can be used to determine a characteristic tensile strength. Torque testing of the anchors can provide tensile strength values if a relationship can be established between torque and bolt tension. The anchor manufacturer should be consulted, if possible, to establish the relationship between torque and tension in the anchor. The evaluating engineer and licensed design professional should consider the condition of the anchors that are torque-tested in establishing the relationship between torque and applied tension.

R3.6.3.1 The maximum anchor demand, Q_{UF} , needs to be increased by a factor of 1.33 to determine the demand for the test load. This ratio assumes that the anchors do not fail the proof load testing.

The proof load of $2/3N_{cb}$ is based on the value recommended by **Silva and Mattis (2011)**. Under the allowable stress design format, proof loads are established as twice the anchor allowable load. Because allowable loads