6. Concrete

6.1 Scope

This chapter sets forth requirements for the Systematic Rehabilitation of concrete lateral-force-resisting elements within a building. The requirements of this chapter shall apply to existing concrete components of a building system, rehabilitated concrete components of a building system, and new concrete components that are added to an existing building system.

Sections 6.2 and 6.3 specify data collection procedures for obtaining material properties and performing condition assessments. Section 6.4 specifies general analysis and design requirements for concrete components. Sections 6.5, 6.6, 6.7, 6.8, 6.9, and 6.10 provide modeling procedures, component strengths, acceptance criteria, and rehabilitation measures for concrete and precast concrete moment frames, braced frames, and shear walls. Sections 6.11, 6.12, and 6.13 provide modeling procedures, strengths, acceptance criteria, and rehabilitation measures for concrete diaphragm systems.

C6.1 Scope

Techniques for repair of earthquake-damaged concrete components are not included in this standard. The design professional is referred to *FEMA 306*, *FEMA 307,* and *FEMA 308* for information on evaluation and repair of damaged concrete wall components.

6.2 Material Properties Based on Historical Information

Available construction documents and as-built information shall be obtained as specified in Section 2.2. Use of material properties based on historical information as default values shall be as specified in Section 6.3.2.5.

C6.2 Material Properties Based on Historical Information

The form, function, concrete strength, concrete quality, reinforcing steel strength, quality and detailing, forming techniques and concrete placement techniques have constantly evolved and have had a significant impact on the seismic resistance of a concrete building. Innovations such as prestressed and precast concrete, post tensioning, and lift slab construction have created a multivariant inventory of existing concrete structures.

It is important to investigate the local practices relative to seismic design when trying to analyze a concrete building. 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. In the absence of deleterious conditions or materials, concrete gains compressive strength from the time it is originally cast and in-place. Strengths typically exceed specified design values (28-day or similar). Early uses of concrete did not specify any design strength, and low-strength concrete was not uncommon. Also, early use of concrete in buildings often employed reinforcing steel with relatively low strength and ductility, limited continuity, and reduced bond development. Continuity between specific existing components and elements (e.g., beams and columns, diaphragms and shear walls) is also particularly difficult to assess, given the presence of concrete cover and other barriers to inspection.

Properties of welded wire fabric for various periods of construction can be obtained from the Wire Reinforcement Institute.

Table 6-1 Default Lower-Bound Tensile and Yield Properties of Reinforcing Bars for Various Periods[1]

Year	Grade	Structural[2] 33	Intermediate[2] 40	Hard[2] 50	60	70	75
	Minimum Yield (psi)	33,000	40,000	50,000	60,000	70,000	75,000
	Minimum Tensile (psi)	55,000	70,000	80,000	90,000	95,000	100,000
1911-1959		x	x	x			
1959-1966		x	x	x	x		x
1966-1972			x	x	x		
1972-1974			x	x	x		
1974-1987			x	x	x	x	
1987-present			x	x	x	x	x

Notes:

1. An entry of “x” indicates the grade was available in those years.
2. The terms structural, intermediate, and hard became obsolete in 1968.

Table 6-2 Default Lower-Bound Tensile and Yield Properties of Reinforcing Bars for Various ASTM Specifications and Periods[1]

				Structural[2]	**Intermediate**[2]	**Hard**[2]			
			ASTM Grade	**33**	**40**	**50**	**60**	**70**	**75**
			Minimum Yield (psi)	33,000	40,000	50,000	60,000	70,000	75,000
ASTM Designation[5]	Steel Type	Year Range	Minimum Tensile (psi)	55,000	70,000	80,000	90,000	95,000	100,000
A15	Billet	1911-1966		x	x	x			
A16	Rail[3]	1913-1966				x			
A61	Rail[3]	1963-1966					x		
A160	Axle	1936-1964		x	x	x			
A160	Axle	1965-1966		x	x	x	x		
A408	Billet	1957-1966		x	x	x			
A431	Billet	1959-1966							x
A432	Billet	1959-1966					x		
A615	Billet	1968-1972			x		x		x
A615	Billet	1974-1986			x		x		
A615	Billet	1987-1997			x		x		x
A616[4]	Rail[3]	1968-1997				x	x		
A617	Axle	1968-1997			x		x		
A706	Low-Alloy	1974-1997						x	
A955	Stain-less	1996-1997			x		x		x

Notes:

1. An entry of “x” indicates the grade was available in those years.
2. The terms structural, intermediate, and hard became obsolete in 1968.
3. Rail bars are marked with the letter “R.”
4. Bars marked “s!” (ASTM 616) have supplementary requirements for bend tests.
5. ASTM steel is marked with the letter “W.”

Table 6-3 Default Lower-Bound Compressive Strength of Structural Concrete (psi)

Time Frame	Footings	Beams	Slabs	Columns	Walls
1900–1919	1000–2500	2000–3000	1500–3000	1500–3000	1000–2500
1920–1949	1500–3000	2000–3000	2000–3000	2000–4000	2000–3000
1950–1969	2500–3000	3000–4000	3000–4000	3000–6000	2500–4000
1970–Present	3000–4000	3000–5000	3000–5000	3000–10000	3000–5000

Table 6-4 Factors to Translate Lower Bound Material Properties to Expected Strength Material Properties

Material Property	Factor
Concrete Compressive Strength	1.50
Reinforcing Steel Tensile & Yield Strength	1.25
Connector Steel Yield Strength	1.50

6.3 Material Properties and Condition Assessment

6.3.1 General

Mechanical properties for concrete materials and components shall be based on available construction documents and as-built conditions for the particular structure. Where such information fails to provide adequate information to quantify material properties or document the condition of the structure, such information shall be supplemented by materials tests and assessments of existing conditions as required in Section 2.2.6.

Material properties of existing concrete components shall be determined in accordance with Section 6.3.2. A condition assessment shall be conducted in accordance with Section 6.3.3. The extent of materials testing and condition assessment performed shall be used to determine the knowledge factor as specified in Section 6.3.4.

Use of default material properties shall be permitted in accordance with Section 6.3.2.5.

C6.3.1 General

This section identifies properties requiring consideration and provides guidelines for determining the properties of buildings. Also described is the need for a thorough condition assessment and utilization of knowledge gained in analyzing component and system behavior. Personnel involved in material property quantification and condition assessment shall be experienced in the proper implementation of testing practices and the interpretation of results.

Documentation of properties and grades of material used in component/connection construction is invaluable and may be effectively used to reduce the amount of in-place testing required. The design professional is encouraged to research and acquire all available records from original construction.

6.3.2 Properties of In-Place Materials and Components

6.3.2.1 Material Properties

6.3.2.1.1 General

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

1. Concrete compressive strength.

2. Yield and ultimate strength of conventional and prestressing reinforcing steel and metal connection hardware.

When materials testing is required by Section 2.2.6, the test methods to quantify material properties shall comply with the requirements of Section 6.3.2.3. The frequency of sampling, including the minimum number of tests for property determination shall comply with the requirements of Section 6.3.2.4.

C6.3.2.1.1 General

Other material properties that may be of interest for concrete elements and components include:

1. Tensile strength and modulus of elasticity, which can be derived from the compressive strength, do not warrant the damage associated with the extra coring required.
2. Ductility, toughness, and fatigue properties of concrete.
3. Carbon equivalent present in the reinforcing steel.
4. Presence of any degradation such as corrosion, bond with concrete, and chemical composition.

The effort required to determine these properties depends on the availability of accurate updated construction documents and drawings, quality and type of construction (absence of degradation), accessibility, and condition of materials. The method of analysis selected (e.g., Linear Static Procedure, Nonlinear Static Procedure) may also influence the scope of the testing.

The size of the samples and removal practices to be followed are referenced in *FEMA 274*. Generally, mechanical properties for both concrete and reinforcing steel can be established from combined core and specimen sampling at similar locations, followed by laboratory testing. Core drilling should minimize damaging the existing reinforcing steel as much as practicable.

6.3.2.1.2 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 6-4 to translate from lower bound to expected values. Alternative factors shall be permitted where justified by test data.

6.3.2.2 Component Properties

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

1. Cross-sectional dimensions of individual components and overall configuration of the structure.
2. Configuration of component connections, size of anchor bolts, thickness of connector material, anchorage and interconnection of embedments and the presence of bracing or stiffening components.
3. Modifications to components or overall configuration of the structure.
4. Current physical condition of components and connections, and the extent of any deterioration present.
5. Presence of conditions that influence building performance.

C6.3.2.2 Component Properties

Component properties may be needed to characterize building performance properly in the seismic analysis. The starting point for assessing component properties and condition should be retrieval of available construction documents. Preliminary review of these documents shall be performed to identify primary vertical- (gravity-) and lateral load-carrying elements, systems, and their critical components and connections. In the absence of a complete set of building drawings, the design professional must perform a thorough inspection of the building to identify these elements, systems and components as indicated in Section 6.3.3.

6.3.2.3 Test Methods to Quantify Material Properties

6.3.2.3.1 General

Destructive and non-destructive test methods used to obtain in-place mechanical properties of materials identified in Section 6.3.2.1 and component properties identified in Section 6.3.2.2 shall comply with the requirements of this section. Samples of concrete and reinforcing and connector steel shall be examined for physical condition as specified in Section 6.3.3.2.

If the determination of material properties is accomplished through removal and testing of samples for laboratory analysis, sampling shall take place in primary gravity- and lateral-force-resisting components in regions with the least stress.

Expected material properties shall be based on mean test values. Lower bound material properties shall be based on mean test values minus one standard deviation.

6.3.2.3.2 Sampling

For concrete, the sampling program shall consist of the removal of standard cores. Core drilling shall be preceded by nondestructive location of the reinforcing steel. Core holes shall be filled with comparable-strength concrete or grout. If conventional reinforcing and bonded prestressing steel is tested, sampling shall consist of the removal of local bar segments and installation of replacement spliced material to maintain continuity of the rebar for transfer of bar force.

Core strength shall be converted to in situ concrete compressive strength (f_c) by an approved procedure.

If a sample population greater than the minimum specified in Section 6.3.2.4 is used in the testing program and the coefficient of variation in test results is less than 14%, the mean strength derived shall be permitted to be used as the expected strength in the analysis. The expected concrete strength shall not exceed the mean less one standard deviation in situations where the coefficient of variation is greater than 14%.

Removal of core samples and performance of laboratory destructive testing shall be permitted as a method of determining existing concrete strength properties. Removal of core samples shall employ the procedures contained in *ASTM C42/C42M-99*. Testing shall follow the procedures contained in *ASTM C42/C42M-99*, *ASTM C39/C39M-99*, and *ASTM C496-96*. The measured strength from testing shall be correlated to in-place concrete compressive strength.

Removal of bar or tendon length samples and performance of laboratory destructive testing shall be permitted as a method of determining existing reinforcing steel strength properties. The tensile yield strength and ultimate strength for reinforcing and prestressing steels shall be obtained using the procedures contained in *ASTM A370-97a*. Prestressing materials also shall meet the supplemental requirements in *ASTM A416/A416M-99*, *ASTM A421/A421M-98a*, or *ASTM A722/A722M-98*, 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 A370-97a*. Where strengths of embedded connectors are required, in situ testing shall satisfy the provisions of *ASTM E488-96*.

C6.3.2.3 Test Methods to Quantify Material Properties

ACI 318 and *FEMA 274* provide further guidance on correlating core strength to in-place strength and provides references for various test methods that may be used to estimate material properties.

The chemical composition may also be determined from the retrieved samples.

FEMA 274 provides references for these tests.

Usually, the reinforcing steel system used in the construction of a specific building is of a common grade and strength. Occasionally one grade of reinforcement is used for small-diameter bars (e.g., those used for stirrups and hoops) and another grade for large-diameter bars (e.g., those used for longitudinal reinforcement). Furthermore, it is possible that a number of different concrete design strengths (or “classes”) have been employed. Historical research and industry documents also contain insight on material mechanical properties used in different construction eras.

6.3.2.4 Minimum Number of Tests

Materials testing is not required if material properties are available from original construction documents that include material test records or material test reports.

The minimum number of tests necessary to quantify properties by in-place testing for comprehensive data collection shall be as specified in Sections 6.3.2.4.1 through 6.3.2.4.4. The minimum number of tests for usual data collection shall be as specified in Section 6.3.2.4.5. If the existing vertical or lateral-force-resisting system is being replaced in the rehabilitation process, material testing shall be required only to qualify properties of existing materials at new connection points.

C6.3.2.4 Minimum Number of Tests

In order to quantify in-place properties accurately, it is important that a minimum number of tests be conducted on primary components in the lateral-force-resisting system. The minimum number of tests is dictated by available data from original construction, the type of structural system employed, desired accuracy, and the quality and condition of in-place materials. The accessibility of the structural system may also influence the testing program scope. The focus of this testing shall be on primary lateral-force-resisting components and on specific properties needed for analysis. The test quantities provided in this section are minimum numbers; the design professional should determine whether further testing is needed to evaluate as-built conditions.

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

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

6.3.2.4.1 Comprehensive Testing

Unless specified otherwise, a minimum of three tests shall be conducted to determine any property. If the coefficient of variation exceeds 14%, additional tests shall be performed until the coefficient of variation is equal to or less than 14%.

6.3.2.4.2 Concrete Materials

For each concrete element type (such as a shear wall), 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/grades were employed in the construction of the building, a minimum of three samples and tests shall be performed for each class. The modulus of elasticity shall be permitted to be estimated from the data of strength testing. Samples shall be taken from randomly selected components critical to structural behavior of the building. Tests also shall be performed on samples from components that are damaged or degraded, if such damage or degradation is identified to quantify their condition. Test results 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 extent of the condition.

The minimum number of tests to determine compressive and tensile strength shall conform to the following criteria.

1. For concrete elements for which the specified design strength is known and test results are not available, a minimum of three cores/tests shall be conducted for each floor level, 400 cubic yards of concrete, or 10,000 square feet of surface area, whichever requires the most frequent testing.

2. For concrete elements for which the design strength is unknown and test results are not available, a minimum of six cores/tests shall be conducted for each floor level, 400 cubic yards of concrete, or 10,000 square feet of surface area, whichever requires the most frequent testing. Where the results indicate that different classes of concrete were employed, the degree of testing shall be increased to confirm class use.

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

C6.3.2.4.2 Concrete Materials

Ultrasonics and nondestructive test methods should not be substituted for core sampling and laboratory testing since they do not yield accurate strength values directly.

6.3.2.4.3 Conventional Reinforcing and Connector Steels

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

1. If original construction documents defining properties exist, at least three strength coupons shall be randomly removed from each element or component type and tested.

2. If original construction documents defining properties do not exist, but the approximate date of construction is known and a common material grade is confirmed, at least three strength coupons shall be randomly removed from each element or component type for every three floors of the building. If the date of construction is unknown, at least six such samples/tests, for every three floors, shall be performed.

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

6.3.2.4.4 Prestressing Steels

The sampling of prestressing steel tendons for laboratory testing shall be required only for those prestressed components that are a part of the lateral-force-resisting system. Prestressed components in diaphragms shall be permitted to be excluded from testing.

Tendon or prestress removal shall be avoided if possible by sampling of either the tendon grip or extension beyond the anchorage.

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.

6.3.2.4.5 Usual Testing

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

1. 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.

2. If the specified design strength of the concrete is not known, at least one core shall be taken from each type of component, with a minimum of six cores taken for the entire building.

3. If the specified design strength of the reinforcing steel is known, use of nominal or specified material properties shall be permitted without additional testing.

4. If the specified design strength of the reinforcing steel is not known, at least two strength coupons of reinforcing steel shall be removed from a building for testing.

C6.3.2.4.5 Usual Testing

For other material properties, such as hardness and ductility, no minimum number of tests is prescribed. Similarly, standard test procedures may not exist. The design professional should examine the particular need for this type of testing and establish an adequate protocol.

6.3.2.5 Default Properties

Use of default material properties to determine component strengths shall be permitted in conjunction with the linear analysis procedures of Chapter 3.

Default lower-bound concrete compressive strengths shall be taken from Table 6-3. Default expected strength concrete compressive strengths shall be determined by multiplying lower-bound values by an appropriate factor selected from Table 6-4 unless another factor is justified by test data. The appropriate default compressive strength—lower bound or expected strength as specified in Section 2.4.4—shall be used to establish other strength and performance characteristics for the concrete as needed in the structural analysis.

Default lower-bound values for reinforcing steel shall be taken from Table 6-1 or 6-2. Default expected strength values for reinforcing steel shall be determined by multiplying lower-bound values by an appropriate factor selected from Table 6-4 unless another factor is justified by test data. For Rehabilitation Objectives in which default values are assumed for existing reinforcing steel, welding or mechanical coupling of new reinforcement to the existing reinforcing steel shall not be used.

The default lower-bound yield strength for steel connector material shall be taken as 27,000 psi. The default expected yield strength for steel connector material shall be determined by multiplying lower-bound values by an appropriate factor selected from Table 6-4 unless another value is justified by test data.

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

C6.3.2.5 Default Properties

Default values provided in this standard are generally conservative. While strength of reinforcing steel may be fairly consistent throughout a building, the strength of concrete in a building could be highly variable, given variability in concrete mix designs and sensitivity to water-cement ratio and curing practices.

Until about 1920, a variety of proprietary reinforcing steels was used. Yield strengths are likely to be in the range of 33,000 to 55,000 psi, but higher values are possible and actual yield and tensile strengths may exceed minimum values. Once commonly used to designate reinforcing steel grade, the terms *structural*, *intermediate* and *hard* became obsolete in 1968. Plain and twisted square bars were sometimes used between 1900 and 1949.

Factors to convert default reinforcing steel strength to expected strength include consideration of material overstrength and strain hardening.

6.3.3 Condition Assessment

6.3.3.1 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:

1. The physical condition of primary and secondary components shall be examined and the presence of any degradation shall be noted.

2. The presence and configuration of components and their connections, and the continuity of load paths between components, elements, and systems shall be verified or established.

3. Other conditions including neighboring party walls and buildings, presence of nonstructural components, prior remodeling, and limitations for rehabilitation that may influence building performance shall be reviewed and documented.

4. A basis for selecting a knowledge factor in accordance with Section 6.3.4 shall be formulated.

5. Component orientation, plumbness, and physical dimensions shall be confirmed.

6.3.3.2 Scope and Procedures

The scope of the condition assessment shall include all accessible structural elements and components involved in lateral load resistance.

C6.3.3.2 Scope and Procedures

The degree to which the condition assessment is performed will affect the knowledge (κ) factor as specified in Section 6.3.4.

6.3.3.2.1 Visual Condition Assessment

Direct visual inspection of accessible and representative primary components and connections shall be performed to identify any configurational issues, determine whether degradation is present, establish continuity of load paths, establish the need for other test methods to quantify the presence and degree of degradation, and measure dimensions of existing construction to compare with available design information and reveal any permanent deformations.

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

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

6.3.3.2.2 Comprehensive Condition Assessment

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

1. If detailed design drawings exist, exposure of at least three different primary connections shall occur, with the connection sample including different types of connections. If no deviations from the drawings exist, it shall be permitted to consider the sample as being representative of installed conditions. If deviations are noted, then at least 25% of the specific connection type shall be inspected to identify the extent of deviation.

2. 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 as 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.

6.3.3.2.3 Additional Testing

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

C6.3.3.2.3 Additional Testing

The physical condition of components and connectors will affect their performance. The need to accurately identify the physical condition may also dictate the need for certain additional destructive and nondestructive test methods. Such methods may be used to determine the degree of damage or presence of deterioration, and to improve understanding of the internal condition and quality of the concrete. Further guidelines and procedures for destructive and nondestructive tests that may be used in the condition assessment are provided in *FEMA 274* and *FEMA 306*. The following paragraphs identify those nondestructive examination (NDE) methods having the greatest use and applicability to condition assessment.

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

- Volumetric NDE methods, including radiography and ultrasonics, may be used to identify the presence of internal discontinuities, as well as to identify loss of section. Impact-echo ultrasonics is particularly useful because of ease of implementation and proven capability in concrete.

- Structural condition and performance may be assessed through on-line monitoring using acoustic emissions and strain gauges, and in-place static or dynamic load tests. Monitoring is used to determine if active degradation or deformations are occurring, while nondestructive load testing provides direct insight on load-carrying capacity.

- Locating, sizing, and initial assessment of the reinforcing steel may be completed using electromagnetic methods (such as a pacometer) or radiography. Further assessment of suspected corrosion activity should use electrical half-cell potential and resistivity measurements.

- Where it is absolutely essential, the level of prestress remaining in an unbonded prestressed system may be measured using lift-off testing (assuming original design and installation data are available), or another nondestructive method such as “coring stress relief” specified in *ASCE 11*.

6.3.3.3 Basis for the Mathematical Building Model

The results of the condition assessment shall be used to quantify the following items needed to create the mathematical building model:

1. Component section properties and dimensions.

2. Component configuration and the presence of any eccentricities or permanent deformation.

3. Connection configuration and the presence of any eccentricities.

4. Presence and effect of alterations to the structural system since original construction.

5. Interaction of nonstructural components and their involvement in lateral load resistance.

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

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

6.3.4 Knowledge Factor

A knowledge factor (κ) for computation of concrete component capacities and permissible deformations shall be selected in accordance with Section 2.2.6.4 with the following 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:

1. Components are found damaged or deteriorated during assessment, and further testing is not performed to quantify their condition or justify the use of κ=1.0.

2. Component mechanical properties have a coefficient of variation exceeding 25%.

3. Components contain archaic or proprietary material and the condition is uncertain.

6.4 General Assumptions and Requirements

6.4.1 Modeling and Design

6.4.1.1 General Approach

Seismic rehabilitation of concrete structural elements of existing buildings shall comply with the requirements of *ACI 318*, except as otherwise indicated in this standard. Seismic evaluation shall identify brittle or low-ductility failure modes of force-controlled actions as defined in Section 2.4.4.

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

C6.4.1.1 General Approach

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. It is recommended that the stresses, forces, and moments acting to cause these failure modes be determined from a limit-state analysis considering probable resistances at locations of nonlinear action.

6.4.1.2 Stiffness

Component stiffnesses shall be calculated considering shear, flexure, axial behavior and reinforcement slip deformations. Consideration shall be given to the state of stress on the component due to volumetric changes from temperature and shrinkage, and to deformation levels to which the component will be subjected under gravity and earthquake loading.

6.4.1.2.1 Linear Procedures

Where design actions are determined using the linear procedures of Chapter 3, component effective stiffnesses shall correspond to the secant value to the yield point of the component. The use of higher stiffnesses shall be permitted where it is demonstrated by analysis to be appropriate for the design loading.

Table 6-5 Effective Stiffness Values

Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams—nonprestressed	$0.5E_cI_g$	$0.4E_cA_w$	—
Beams—prestressed	E_cI_g	$0.4E_cA_w$	—
Columns with compression due to design gravity loads $\geq 0.5\ A_g f'_c$	$0.7E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns with compression due to design gravity loads $\leq 0.3\ A_g f'_c$ or with tension	$0.5E_cI_g$	$0.4E_cA_w$	E_sA_s
Walls—uncracked (on inspection)	$0.8E_cI_g$	$0.4E_cA_w$	E_cA_g
Walls—cracked	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
Flat Slabs—nonprestressed	See Section 6.5.4.2	$0.4E_cA_g$	—
Flat Slabs—prestressed	See Section 6.5.4.2	$0.4E_cA_g$	—

Note: It shall be permitted to take I_g for T-beams as twice the value of I_g of the web alone. Otherwise, I_g shall be based on the effective width as defined in Section 6.4.1.3. For columns with axial compression falling between the limits provided, linear interpolation shall be permitted. Alternatively, the more conservative effective stiffnesses shall be used.

Alternatively, the use of effective stiffness values in Table 6-5 shall be permitted.

6.4.1.2.2 Nonlinear Procedures

Where design actions are determined using the nonlinear procedures of Chapter 3, component load-deformation response shall be represented by nonlinear load-deformation relations. Linear relations shall be permitted where nonlinear response will not occur in the component. The nonlinear load-deformation relation shall be based on experimental evidence or taken from quantities specified in Sections 6.5 through 6.13. For the Nonlinear Static Procedure (NSP), use of the generalized load-deformation relation shown in Figure 6-1 or other curves defining behavior under monotonically increasing deformation shall be permitted. For the Nonlinear Dynamic Procedure (NDP), load-deformation relations shall define behavior under monotonically increasing lateral deformation and under multiple reversed deformation cycles as specified in Section 6.4.2.1.

The generalized load-deformation relation shown in Figure 6-1 shall be described by linear response from A (unloaded component) to an effective yield B, then a linear response at reduced stiffness from point B to C, then sudden reduction in lateral load resistance to point D, then response at reduced resistance to E, and final loss of resistance thereafter. The slope from point A to B shall be determined according to Section 6.4.1.2.1. The slope from point B to C, ignoring effects of gravity loads acting through lateral displacements, shall be taken between zero and 10% of the initial slope unless an alternate slope is justified by experiment or analysis. Point C shall have an ordinate equal to the strength of the component and an abscissa equal to the deformation at which significant strength degradation begins. Representation of the load-deformation relation by points A, B, and C only (rather than all points A–E), shall be permitted if the calculated response does not exceed point C. Numerical values for the points identified in Figure 6-1 shall be as specified in Sections 6.5 through 6.13. Other load-deformation relations shall be permitted if justified by experimental evidence or analysis.

C6.4.1.2.2 Nonlinear Procedures

Typically, the responses shown in Figure 6-1 are associated with flexural response or tension response. In this case, the resistance at $Q/Q_y = 1.0$ is the yield value, and subsequent strain hardening accommodates strain hardening in the load-deformation relation as the member is deformed toward the expected strength. When the response shown in Figure 6-1 is associated with compression, the resistance at $Q/Q_y = 1.0$ typically is the value at which concrete begins to spall, and strain hardening in well-confined sections may be associated with strain hardening of the longitudinal reinforcement and the confined concrete. When the response shown in Figure 6-1 is associated with shear, the resistance at $Q/Q_y = 1.0$ typically is the value at which the design shear strength is reached, and no strain hardening follows.

The deformations used for the load-deformation relation of Figure 6-1 shall 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* and *b* shall refer to those portions of the deformation that occur after yield; that is, the plastic deformation. The parameter *c* is the reduced resistance after the sudden reduction from *C* to *D*. Parameters *a*, *b*, and *c* are defined numerically in various tables in this chapter. Alternatively, it shall be permitted to determine the parameters *a*, *b*, and *c* 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* and *e* refer to total deformations measured from the origin. Parameters *c*, *d*, and *e* are defined numerically in various tables in this chapter. Alternatively, it shall be permitted to determine the parameters *c*, *d*, and *e* directly by analytical procedures justified by experimental evidence.

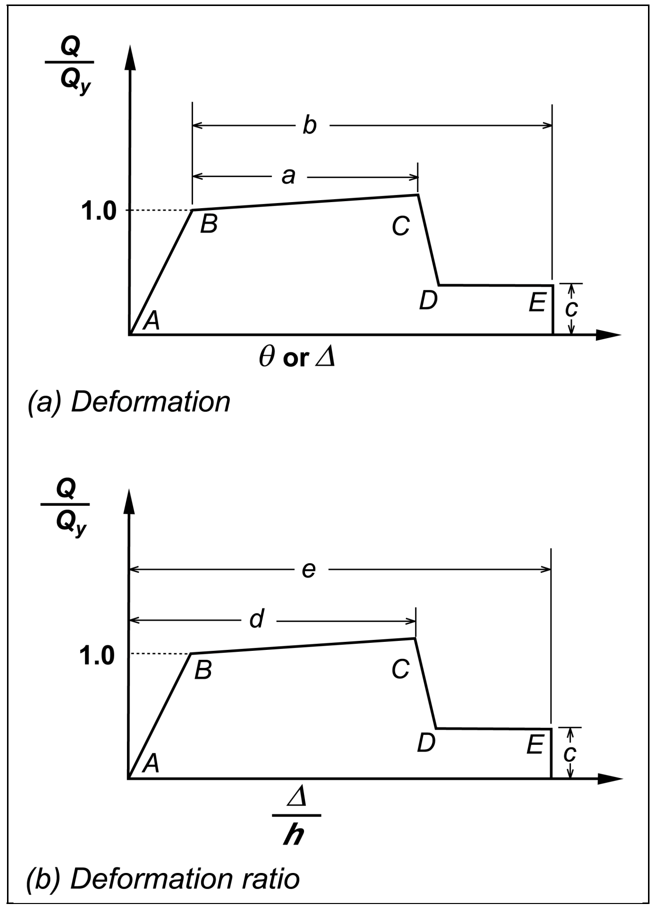

Figure 6-1 Generalized Force-Deformation Relations for Concrete Elements or Components

6.4.1.3 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 smaller of: (1) the provided flange width, (2) eight times the flange thickness, (3) half the distance to the next web, or (4) one-fifth of the span for beams. When the flange is in compression, both the concrete and reinforcement within the effective width shall be considered effective in resisting flexure and axial load. When the flange is in tension, longitudinal reinforcement within the effective width and that is 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 shall be in accordance with Chapter 21 of *ACI 318*.

6.4.2 Strength and Deformability

6.4.2.1 General

Actions in a structure shall be classified as being either deformation-controlled or force-controlled, as defined in Section 2.4.4. Design strengths for deformation-controlled and force-controlled actions shall be calculated in accordance with Sections 6.4.2.2 and 6.4.2.3, respectively.

Components shall be classified as having low, moderate, or high ductility demands according to Section 6.4.2.4.

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 requirements specified in Section 2.8.1.

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

C6.4.2.1 General

Strengths and deformation capacities given in this chapter are for earthquake loadings involving three fully reversed deformation cycles to the design deformation levels, in addition to similar cycles to lesser deformation levels. In some cases—including some short-period buildings and buildings subjected to a long-duration design earthquake—a building may be expected to be subjected to additional cycles to the design deformation levels. The increased number of cycles may lead to reductions in resistance and deformation capacity. The effects on strength and deformation capacity of additional deformation cycles should be considered in design. Large earthquakes will cause additional cycles.

6.4.2.2 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. Expected strength is defined as the mean maximum resistance expected over the range of deformations to which the concrete component is likely to be subjected. When calculations are used to define expected strength, expected material properties shall be used. Unless other procedures are specified in this standard, procedures specified in *ACI 318* to calculate design 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 Sections 6.5 to Section 6.13. For components constructed of lightweight concrete, Q_{CE} shall be modified in accordance with *ACI 318* procedures for lightweight concrete.

C6.4.2.2 Deformation-Controlled Actions

Expected yield strength of reinforcing steel, as specified in this standard, includes consideration of material overstrength and strain hardening.

6.4.2.3 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 minus one standard deviation of resistance expected over the range of deformations and loading cycles to which the concrete component is likely to be subjected. When calculations are used to define lower-bound strengths, lower bound estimates of material properties shall be used. Unless other procedures are specified in this standard, procedures specified in *ACI 318* to calculate design 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.

6.4.2.4 Component Ductility Demand Classification

Where procedures in this chapter require classification of component ductility demand, components shall be classified as having low, moderate, or high ductility demands, based on the maximum value of the demand capacity ratio (DCR) defined in Section 2.4.1 for linear procedures, or the calculated displacement ductility for nonlinear procedures in accordance with Table 6-6.

Table 6-6 Component Ductility Demand Classification

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

6.4.3 Flexure and Axial Loads

Flexural strength and deformation capacity of members with and without axial loads shall be calculated according to the procedures of *ACI 318* or by other approved methods. 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 Section 6.4.1.3.

Strength and deformation capacities shall be determined considering available development of longitudinal reinforcement. Where longitudinal reinforcement has embedment or development length that is insufficient for development of reinforcement strength, flexural strength shall be calculated based on limiting stress capacity of the embedded bar as defined in Section 6.4.5.

Where flexural deformation capacities are calculated from basic principles of mechanics, reductions in deformation capacity due to applied shear shall be taken into consideration. When using analytical models for flexural deformability that do not directly consider effect of shear, and where design shear equals or exceeds $6\sqrt{f_c'}A_w$, where f'_c is in psi and A_w is gross area of web in square inches, the design value shall not exceed eighty percent of the value calculated using the analytical model.

For concrete columns under combined axial load and biaxial bending, the combined strength shall be evaluated considering biaxial bending. When using linear procedures, the design axial load, P_{UF}, shall be calculated as a force-controlled action in accordance with Section 3.4. The design moments, M_{UD}, shall be calculated about each principal axis in accordance with Section 3.4. Acceptance shall be based on the following equation:

$$\left(\frac{M_{UDx}}{m_x \kappa M_{CEx}}\right)^2 + \left(\frac{M_{UDy}}{m_y \kappa M_{CEy}}\right)^2 \le 1 \qquad (6\text{-}1)$$

where:

M_{UDx} = design bending moment about x axis for axial load P_{UF}, kip-in.

M_{UDy} = design bending moment about y axis for axial load P_{UF}, kip-in.

M_{CEx} = expected bending moment strength about x axis, kip-in.

M_{CEy} = expected bending moment strength about y axis, kip-in.

m_x = *m*-factor for column for bending about x axis

m_y = *m*-factor for column for bending about y axis

Alternative approaches based on principles of mechanics shall be permitted.

C6.4.3 Flexure and Axial Loads

Laboratory tests indicate that flexural deformability may be reduced as co-existing shear forces increase. As flexural ductility demands increase, shear capacity decreases, which may result in a shear failure before theoretical flexural deformation capacities are reached. Caution should be exercised when flexural deformation capacities are determined by calculation. *FEMA 306* is a resource for guidance regarding the interaction between shear and flexure.

6.4.3.1 Usable Strain Limits

Without confining transverse reinforcement, the maximum usable strain at the extreme concrete compression fiber shall not exceed 0.002 for components in nearly pure compression and 0.005 for other components unless larger strains are substantiated by experimental evidence and approved. Maximum usable compressive strains for confined concrete shall be based on experimental evidence and shall consider limitations posed by fracture of transverse reinforcement, buckling of longitudinal reinforcement, and degradation of component resistance at large deformation levels. Maximum compressive strains in longitudinal reinforcement shall not exceed 0.02, and maximum tensile strains in longitudinal reinforcement shall not exceed 0.05.

6.4.4 Shear and Torsion

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

Within yielding regions of components with moderate or high ductility demands, shear and torsional strength shall be calculated according to procedures for ductile components, such as the provisions in Chapter 21 of *ACI 318*. Within yielding regions of components with low ductility demands and outside yielding regions for all ductility demands, calculation of design shear strength using procedures for effective elastic response such as the provisions in Chapter 11 of *ACI 318* shall be permitted.

Where the longitudinal spacing of transverse reinforcement exceeds half the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed not more than 50% effective in resisting shear or torsion. Where the longitudinal spacing of transverse reinforcement exceeds the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed ineffective in resisting shear or torsion. For beams and columns in which perimeter hoops are either lap-spliced or have hooks that are not adequately anchored in the concrete core, transverse reinforcement shall be assumed not more than 50% effective in regions of moderate ductility demand and shall be assumed ineffective in regions of high ductility demand.

Shear friction strength shall be calculated according to *ACI 318*, taking into consideration the expected axial load due to gravity and earthquake effects. Where rehabilitation involves the addition of concrete requiring overhead work with dry-pack, the shear friction coefficient μ shall be taken as equal to 70% of the value specified by *ACI 318*.

6.4.5 Development and Splices of Reinforcement

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

1. Within yielding regions of components with moderate or high ductility demands as classified in Section 6.4.2.4, deformed straight bars, hooked bars, and lap-spliced bars shall meet the development requirements of Chapter 21 of *ACI 318*. Within yielding regions of components with low ductility demands, and outside yielding regions for all ductility demands, bars shall meet the development requirements of Chapter 12 of *ACI 318*, except requirements for lap splices shall be the same as those for straight development of bars in tension without consideration of lap splice classifications.

2. Where existing deformed straight bars, hooked bars, and lap-spliced bars do not meet the development requirements of (1) above, the capacity of existing reinforcement shall be calculated using Equation (6-2):

$$f_s = \frac{l_b}{l_d} f_y \tag{6-2}$$

   where f_s = maximum stress that can be developed in the bar for the straight development, hook, or lap splice length l_b provided; f_y = yield strength of reinforcement; and l_d = length required by Chapter 12 or Chapter 21 (as appropriate) of *ACI 318* for straight development, hook development, or lap splice length, except required splice lengths may be taken as straight bar development lengths in tension. Where transverse reinforcement is distributed along the development length with spacing not exceeding one-third of the effective depth of the component, it shall be permitted to assume the reinforcement retains the calculated maximum stress to high ductility demands. For larger spacings of transverse reinforcement, the developed stress shall be assumed to degrade from f_s to $0.2f_s$ at a ductility demand or DCR equal to 2.0.

3. Strength of deformed straight, discontinuous bars embedded in concrete sections or beam-column joints, with clear cover over the embedded bar not less than $3d_b$, shall be calculated according to Equation (6-3):

$$f_s = \frac{2500}{d_b} l_e \le f_y \tag{6-3}$$

   where f_s = maximum stress (in psi) that can be developed in an embedded bar having embedment length l_e (in inches), d_b = diameter of embedded bar (in inches), and f_y = bar yield stress (in psi). When f_s is less than f_y, and the calculated stress in the bar due to design loads equals or exceeds f_s, the maximum developed stress shall be assumed to degrade from f_s to $0.2f_s$ at a ductility demand or DCR equal to 2.0. In beams with short bottom bar embedments into beam-column joints, flexural strength shall be calculated considering the stress limitation of Equation (6-3).

4. For plain straight bars, hooked bars, 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 or calculations considering only the chemical bond between the bar and the concrete.

5. Doweled bars added in seismic rehabilitation shall be assumed to develop yield stress when all the following conditions are satisfied:

   5.1. Drilled holes for dowel bars are cleaned with a stiff brush that extends the length of the hole.

   5.2. Embedment length l_e is not less than $10d_b$

   5.3. Minimum spacing of dowel bars is not less than $4l_e$ and minimum edge distance is not less than $2l_e$. Design values for dowel bars not satisfying these conditions shall be verified by test data. Field samples shall be obtained to ensure design strengths are developed in accordance with Section 6.4.6.3.

C6.4.5 Development and Splices of Reinforcement

Chapter 21 requirements for development of reinforcement in ACI 318 are intended for use in yielding components of reinforced concrete moment frame elements, and not shear wall components. When checking development of bars in shear wall components, Chapter 12 requirements should be used.

For buildings constructed prior to 1950, the bond strength developed between reinforcing steel and concrete may be less than present-day strength. Current equations for development and splices of reinforcement account for mechanical bond due to deformations present in deformed bars in addition to chemical bond. The length required to develop plain bars will be much greater than that required for deformed bars, and will be more sensitive to cracking in the concrete. Procedures for testing and assessment of tensile lap splices and development length of plain reinforcing steel may be found in *CRSI*.

6.4.5.1 Square Reinforcing Bars

Square reinforcing bars in a building shall be classified as either twisted or straight. The developed strength of twisted square bars shall be as specified for deformed bars in Section 6.4.5, using an effective diameter calculated based on the gross area of the square bar. Straight square bars shall be considered as plain bars, and the developed strength shall be as specified for plain bars in Section 6.4.5.

6.4.6 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.

6.4.6.1 Cast-In-Place Systems

Component actions on cast-in-place connection systems, including shear forces, tension forces, bending moments, and prying actions, shall be considered force-controlled. Lower-bound strength of connections shall be ultimate values as specified in an approved building code with $\phi = 1.0$.

The capacity of anchors placed in areas where cracking is expected shall be reduced by a factor of 0.5.

6.4.6.2 Post-Installed Systems

Component actions on post-installed connection systems shall be considered force-controlled. The lower-bound capacity of post-installed anchors shall be mean minus one standard deviation of ultimate values published in approved test reports.

6.4.6.3 Quality Assurance

Connections between existing concrete components and new components added to rehabilitate the structure shall be subject to the quality assurance provisions specified in Section 2.7. The design professional shall specify the required inspection and testing of cast-in-place and post-installed anchors as part of the Quality Assurance Plan.

6.4.7 Rehabilitation—General Requirements

Upon determining that concrete elements in an existing building are deficient for the selected Rehabilitation Objective, these elements shall be rehabilitated or replaced or the structure shall be otherwise rehabilitated so that the element is no longer deficient for the selected rehabilitation objective. If replacement of the element is selected, the new element shall be designed in accordance with an approved building code.

Rehabilitation measures shall be evaluated in accordance with the principles and requirements of this standard, to assure that the completed rehabilitation achieves the selected Rehabilitation Objective. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in an analytical model of the rehabilitated structure. The compatibility of new and existing components shall be checked at displacements consistent with the selected performance level.

Connections required between existing and new elements shall satisfy the requirements of Section 6.4.6 and other requirements of this standard.

6.5 Concrete Moment Frames

6.5.1 Types of Concrete Moment Frames

Concrete moment frames shall be defined as elements composed primarily of horizontal framing components (beams and/or slabs), vertical framing components (columns) and joints connecting horizontal and vertical framing components. These elements resist lateral loads acting alone, or in conjunction with shear walls, braced frames, or other elements in a dual system.

Frames that are cast monolithically, including monolithic concrete frames created by the addition of new material, shall meet the provisions of this section. Frames covered under this section include reinforced concrete beam-column moment frames, prestressed concrete beam-column moment frames, and slab-column moment frames. Precast concrete frames, concrete frames with infills, and concrete braced frames shall meet the provisions of Sections 6.6, 6.7, and 6.10, respectively.

6.5.1.1 Reinforced Concrete Beam-Column Moment Frames

Reinforced concrete beam-column moment frames shall satisfy the following conditions:

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

Special Moment Frames, Intermediate Moment Frames, and Ordinary Moment Frames as defined in *ASCE 7*, shall be deemed to satisfy the above conditions. This classification shall include existing construction, new construction, and existing construction that has been rehabilitated.

6.5.1.2 Post-Tensioned Concrete Beam-Column Moment Frames

Post-tensioned concrete beam-column moment frames shall satisfy the following conditions:

1. Framing components shall be beams (with or without slabs), columns, and their connections.
2. Frames shall be of monolithic construction that provides for moment transfer between beams and columns.
3. Primary reinforcement in beams contributing to lateral load resistance shall include post-tensioned reinforcement with or without mild reinforcement.

This classification shall include existing construction, new construction, and existing construction that has been rehabilitated.

6.5.1.3 Slab-Column Moment Frames

Slab-column moment frames shall satisfy the following conditions:

1. Framing components shall be slabs (with or without beams in the transverse direction), columns, and their connections.
2. Frames shall be of monolithic construction that provides for moment transfer between slabs and columns.
3. Primary reinforcement in slabs contributing to lateral load resistance shall include nonprestressed reinforcement, prestressed reinforcement, or both.

This classification shall include frames intended as part of the lateral-force-resisting system and frames not intended as part of the lateral-force-resisting system in the original design, including existing construction, new construction, and existing construction that has been rehabilitated.

6.5.2 Reinforced Concrete Beam-Column Moment Frames

6.5.2.1 General Considerations

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

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

The beam-column joint in monolithic construction shall be represented as a stiff or rigid zone having horizontal dimensions equal to the column cross-sectional dimensions and vertical dimension equal to the beam depth, except that a wider joint shall be permitted where the beam is wider than the column and where justified by experimental evidence. The model of the connection between the columns and foundation shall be selected based on the details of the column-foundation connection and rigidity of the foundation-soil system.

Action of the slab as a diaphragm interconnecting vertical elements shall be represented. Action of the slab as a composite beam flange shall be considered in developing stiffness, strength, and deformation capacities of the beam component model, according to Section 6.4.1.3.

Inelastic action shall be restricted to those components and actions listed in Tables 6-7 through 6-9, except where it is demonstrated by experimental evidence and analysis that other inelastic action is acceptable for the selected performance level. Acceptance criteria shall be as specified in Section 6.5.2.4.

6.5.2.2 Stiffness for Analysis

6.5.2.2.1 Linear Static and Dynamic Procedures

Beams shall be modeled considering flexural and shear stiffnesses, including the effect of the slab acting as a flange in monolithic construction. Columns shall be modeled considering flexural, shear, and axial stiffnesses. Joints shall be modeled as either stiff or rigid components. Effective stiffnesses shall be according to Section 6.4.1.2.

6.5.2.2.2 Nonlinear Static Procedure

Nonlinear load-deformation relations shall follow the requirements of Section 6.4.1.2.

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

Monotonic load-deformation relations shall be according to the generalized load-deformation relation shown in Figure 6-1, except that different relations shall be permitted where verified by experiments. The overall load-deformation relation shall be established so that the maximum resistance is consistent with the design strength specifications of Sections 6.4.2 and 6.5.2.3.

For beams and columns, the generalized deformation in Figure 6-1 shall be either the chord rotation or the plastic hinge rotation. For beam-column joints, the generalized deformation shall be shear strain. Values of the generalized deformation at points *B*, *C*, and *D* shall be derived from experiments or rational analyses, and shall take into account the interactions between flexure, axial load, and shear.

Table 6-7 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

Conditions			Modeling Parameters[3]			Acceptance Criteria[3]				
						Plastic Rotation Angle, radians				
						Performance Level				
			Plastic Rotation Angle, radians		Residual Strength Ratio		Component Type			
							Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.010	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.0020	0.0030	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.0020	0.0030	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. “C” and “NC” are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table shall be permitted.

Table 6-8 ***Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Columns***

			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
Conditions			**a**	**b**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008
ii. Columns controlled by shear[1, 3]										
All cases [5]			—	—	—	—	—	—	.0030	.0040
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]										
Hoop spacing ≤ d/2			0.01	0.02	0.4	0.005	0.005	0.01	0.01	0.02
Hoop spacing > d/2			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iv. Columns with axial loads exceeding $0.70P_o$[1, 3]										
Conforming hoops over the entire length			0.015	0.025	0.02	0.0	0.005	0.01	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, columns must have transverse reinforcement consisting of hoops. Otherwise, actions shall be treated as force-controlled.
4. Linear interpolation between values listed in the table shall be permitted.
5. For columns controlled by shear, see Section 6.5.2.4.2 for acceptance criteria.

Table 6-9 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beam-Column Joints

Conditions			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Shear Angle, radians		Residual Strength Ratio		Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Interior joints[2, 3]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.	$\frac{V}{V_n}$ [3]								
≤ 0.1	C	≤ 1.2	0.015	0.03	0.2	0.0	0.0	0.0	0.02	0.03
≤ 0.1	C	≥ 1.5	0.015	0.03	0.2	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≤ 1.2	0.015	0.025	0.2	0.0	0.0	0.0	0.015	0.025
≥ 0.4	C	≥ 1.5	0.015	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≤ 1.2	0.005	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≤ 1.2	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
ii. Other joints[2, 3]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[1]	$\frac{V}{V_n}$								
≤ 0.1	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≤ 0.1	NC	≤ 1.2	0.005	0.01	0.2	0.0	0.0	0.0	0.0075	0.01
≤ 0.1	NC	≥ 1.5	0.005	0.01	0.2	0.0	0.0	0.0	0.0075	0.01
≥ 0.4	NC	≤ 1.2	0.0	0.0	–	0.0	0.0	0.0	0.005	0.0075
≥ 0.4	NC	≥ 1.5	0.0	0.0	–	0.0	0.0	0.0	0.005	0.0075

1. “C” and “NC” are abbreviations for conforming and nonconforming transverse reinforcement. A joint is conforming if hoops are spaced at ≤ $h_c/3$ within the joint. Otherwise, the component is considered nonconforming.
2. P is the design axial force on the column above the joint and A_g is the gross cross-sectional area of the joint.
3. V is the design shear force and V_n is the shear strength for the joint. The design shear force and shear strength shall be calculated according to Section 6.5.2.3.
4. Linear interpolation between values listed in the table shall be permitted.

Table 6-10 Values of γ for Joint Strength Calculation

ρ"	Value of γ: Interior joint with transverse beams	Interior joint without transverse beams	Exterior joint with transverse beams	Exterior joint without transverse beams	Knee joint
<0.003	12	10	8	6	4
≥0.003	20	15	15	12	8

ρ'' = volumetric ratio of horizontal confinement reinforcement in the joint; knee joint = self-descriptive (with transverse beams or not).

6.5.2.2.3 Nonlinear Dynamic Procedure

For the NDP, the complete hysteretic behavior of each component shall be modeled using properties verified by experimental evidence. The use of the generalized load-deformation relation described by Figure 6-1 to represent the envelope relation for the analysis shall be permitted. Unloading and reloading properties shall represent significant stiffness and strength degradation characteristics.

6.5.2.3 Strength

Component strengths shall be computed according to the general requirements of Sections 6.4.2 as modified in this section.

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

6.5.2.3.1 Columns

For columns, the contribution of concrete to shear strength, V_c, calculated according to Equation (6-4) shall be permitted.

$$V_c = \lambda k\left(\frac{6\sqrt{f'_c}}{M/Vd}\sqrt{1+\frac{N_u}{6\sqrt{f'_c}A_g}}\right)(0.8b_w h) \qquad (6\text{-}4)$$

in which k = 1.0 in regions of low ductility demand, 0.7 in regions of high ductility demand, and varies linearly between these extremes in regions of moderate ductility demand; λ = 0.75 for lightweight aggregate concrete and 1.0 for normal weight aggregate concrete; N_u = axial compression force in pounds (= 0 for tension force); M/V is the largest ratio of moment to shear under design loadings for the column but shall not be taken greater than 3 or less than 2; d is the effective depth; and A_g is the gross cross-sectional area of the column. It shall be permitted to assume d=0.8h, where h is the dimension of the column in the direction of shear. Where axial force is calculated from the linear procedures of Chapter 3, the maximum compressive axial load for use in Equation (6-4) shall be taken as equal to the value calculated using Equation (3-4) considering design gravity load only, and the minimum compression axial load shall be calculated according to Equation (3-18). Alternatively, limit analysis as specified in Section 3.4.2.1.2 shall be permitted to be used to determine design axial loads for use with the linear analysis procedures of Chapter 3. Alternative formulations for column strength that consider effects of reversed cyclic, inelastic deformations and that are verified by experimental evidence shall be permitted.

For columns satisfying the detailing and proportioning requirements of Chapter 21 of *ACI 318*, the shear strength equations of *ACI 318* shall be permitted to be used.

For beam-column joints, the nominal cross-sectional area, A_j, shall be defined by a joint depth equal to the column dimension in the direction of framing and a joint width equal to the smallest of (1) the column width, (2) the beam width plus the joint depth, and (3) twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side. Design forces shall be calculated based on development of flexural plastic hinges in adjacent framing members, including effective slab width, but need not exceed values calculated from design gravity and earthquake-load combinations. Nominal joint shear strength V_n shall be calculated according to the general procedures of *ACI 318*, as modified by Equation (6-5):

$$Q_{CL} = V_n = \lambda\gamma\sqrt{f'_c}A_j,\ psi \qquad (6\text{-}5)$$

in which $\lambda = 0.75$ for lightweight aggregate concrete and 1.0 for normal weight aggregate concrete, A_j is the effective horizontal joint area with dimensions as defined above, and γ is as defined in Table 6-10.

6.5.2.4 Acceptance Criteria

6.5.2.4.1 Linear Static and Dynamic Procedures

All actions shall be classified as being either deformation-controlled or force-controlled, as defined in Section 2.4.4. In primary components, deformation-controlled actions shall be restricted to flexure in beams (with or without slab) and columns. In secondary components, deformation-controlled actions shall be restricted to flexure in beams (with or without slab), plus restricted actions in shear and reinforcement development, as identified in Tables 6-11 through 6-13. All other actions shall be defined as being force-controlled actions.

Design actions on components shall be determined as prescribed in Chapter 3. Where the calculated DCR values exceed unity, the following design actions shall be determined using limit analysis principles as prescribed in Chapter 3: (1) moments, shears, torsions, and development and splice actions corresponding to development of component strength in beams and columns; (2) joint shears corresponding to development of strength in adjacent beams and columns; and (3) axial load in columns and joints, considering likely plastic action in components above the level in question.

Design actions shall be compared with design strengths in accordance with Section 3.4.2.2. *m*-factors shall be selected from Tables 6-11 through 6-13. Those components that satisfy Equations (3-20) and (3-21) shall satisfy the performance criteria.

Where the average DCR of columns at a level exceeds the average value of beams at the same level, and exceeds the greater of 1.0 and $m/2$ for all columns, the level shall be defined as a weak column element. For weak column elements, one of the following shall be satisfied.

1. The check of average DCR values at the level shall be repeated, considering all primary and secondary components at the level with a weak column element. If the average of the DCR values for vertical components exceeds the average value for horizontal components at the level, and exceeds 2.0, the structure shall be reanalyzed using a nonlinear procedure, or the structure shall be rehabilitated to remove this deficiency.

2. The structure shall be reanalyzed using either the NSP or the NDP of Chapter 3.

3. The structure shall be rehabilitated to remove the weak story.

6.5.2.4.2 Nonlinear Static and Dynamic Procedures

Calculated component actions shall satisfy the requirements of Section 3.4.3.2. Where the generalized deformation is taken as rotation in the flexural plastic hinge zone in beams and columns, the plastic hinge rotation capacities shall be as defined by Tables 6-7 and 6-8. Where the generalized deformation is shear distortion of the beam-column joint, shear angle capacities shall be as defined by Table 6-9. For columns designated as primary components and for which calculated design shears exceed design shear strength as defined by Equation (6-4), the permissible deformation for the Collapse Prevention Performance Level shall not exceed the deformation at which shear strength is calculated to be reached; the permissible deformation for the Life Safety Performance Level shall not exceed three quarters of that value. Where inelastic action is indicated for a component or action not listed in these tables, the performance shall be deemed unacceptable. Alternative approaches or values shall be permitted where justified by experimental evidence and analysis.

Table 6-11 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beams

Conditions			*m*-factors[3]				
			Performance Level				
				Component Type			
				Primary		Secondary	
			IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]							
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$					
≤ 0.0	C	≤ 3	3	6	7	6	10
≤ 0.0	C	≥ 6	2	3	4	3	5
≥ 0.5	C	≤ 3	2	3	4	3	5
≥ 0.5	C	≥ 6	2	2	3	2	4
≤ 0.0	NC	≤ 3	2	3	4	3	5
≤ 0.0	NC	≥ 6	1.25	2	3	2	4
≥ 0.5	NC	≤ 3	2	3	3	3	4
≥ 0.5	NC	≥ 6	1.25	2	2	2	3
ii. Beams controlled by shear[1]							
Stirrup spacing ≤ d/2			1.25	1.5	1.75	3	4
Stirrup spacing > d/2			1.25	1.5	1.75	2	3
iii. Beams controlled by inadequate development or splicing along the span[1]							
Stirrup spacing ≤ d/2			1.25	1.5	1.75	3	4
Stirrup spacing > d/2			1.25	1.5	1.75	2	3
iv. Beams controlled by inadequate embedment into beam-column joint[1]							
			2	2	3	3	4

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table shall be permitted.

Table 6-12 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Columns

Conditions			*m*-factors[4]				
			Performance Level				
				Component Type			
				Primary		Secondary	
			IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]							
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$					
≤ 0.1	C	≤ 3	2	3	4	4	5
≤ 0.1	C	≥ 6	2	2.4	3.2	3.2	4
≥ 0.4	C	≤ 3	1.25	2	3	3	4
≥ 0.4	C	≥ 6	1.25	1.6	2.4	2.4	3.2
≤ 0.1	NC	≤ 3	2	2	3	2	3
≤ 0.1	NC	≥ 6	2	1.6	2.4	1.6	2.4
≥ 0.4	NC	≤ 3	1.25	1.5	2	1.5	2
≥ 0.4	NC	≥ 6	1.25	1.5	1.75	1	1.6
ii. Columns controlled by shear[1,3]							
Hoop spacing ≤ d/2, or $\frac{P}{A_g f'_c} \le 0.1$			–	–	–	2	3
Other cases			–	–	–	1.5	2
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]							
Hoop spacing ≤ d/2			1.25	1.5	1.75	3	4
Hoop spacing > d/2			–	–	–	2	3
iv. Columns with axial loads exceeding $0.70P_o$[1,3]							
Conforming hoops over the entire length			1	1	2	2	2
All other cases			–	–	–	1	1

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. “C” and “NC” are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, columns must have transverse reinforcement consisting of hoops. Otherwise, actions shall be treated as force-controlled.
4. Linear interpolation between values listed in the table shall be permitted.

Table 6-13 Numerical Acceptance Criteria for Linear Procedures— Reinforced Concrete Beam-Column Joints

Conditions			m-factors[4]				
			Performance Level				
				Component Type			
				Primary[5]		Secondary	
			IO	LS	CP	LS	CP
i. Interior joints[2, 3]							
$\frac{P}{A_g f'_c}$	Trans. Reinf.	$\frac{V}{V_n}$					
≤ 0.1	C	≤ 1.2	–	–	–	3	4
≤ 0.1	C	≥ 1.5	–	–	–	2	3
≥ 0.4	C	≤ 1.2	–	–	–	3	4
≥ 0.4	C	≥ 1.5	–	–	–	2	3
≤ 0.1	NC	≤ 1.2	–	–	–	2	3
≤ 0.1	NC	≥ 1.5	–	–	–	2	3
≥ 0.4	NC	≤ 1.2	–	–	–	2	3
≥ 0.4	NC	≥ 1.5	–	–	–	2	3
ii. Other joints[2, 3]							
$\frac{P}{A_g f'_c}$	Trans. Reinf.	$\frac{V}{V_n}$					
≤ 0.1	C	≤ 1.2	–	–	–	3	4
≤ 0.1	C	≥ 1.5	–	–	–	2	3
≥ 0.4	C	≤ 1.2	–	–	–	3	4
≥ 0.4	C	≥ 1.5	–	–	–	2	3
≤ 0.1	NC	≤ 1.2	–	–	–	2	3
≤ 0.1	NC	≥ 1.5	–	–	–	2	3
≥ 0.4	NC	≤ 1.2	–	–	–	1.5	2.0
≥ 0.4	NC	≥ 1.5	–	–	–	1.5	2.0

1. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcements. A joint is conforming if hoops are spaced at ≤ $h_c/3$ within the joint. Otherwise, the component is considered nonconforming.
2. P is the ratio of the design axial force on the column above the joint and A_g is the gross cross-sectional area of the joint.
3. V is the design shear force and V_n is the shear strength for the joint. The design shear force and shear strength shall be calculated according to Section 6.5.2.3.
4. Linear interpolation between values listed in the table shall be permitted.
5. For linear procedures, all primary joints shall be force-controlled; *m*-factors shall not apply.

6.5.2.5 Rehabilitation Measures

Concrete beam-column moment frame components that do not meet the acceptance criteria for the selected rehabilitation objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 6.4.7 and other provisions of this standard.

C6.5.2.5 Rehabilitation Measures

The following rehabilitation measures may be effective in rehabilitating reinforced concrete beam-column moment frames:

1. **Jacketing existing beams, columns, or joints with new reinforced concrete, steel, or fiber wrap overlays.** The new materials should be designed and constructed to act compositely with the existing concrete. Where reinforced concrete jackets are used, the design should provide detailing to enhance ductility. Component strength should be taken to not exceed any limiting strength of connections with adjacent components. Jackets should be designed to provide increased connection strength and improved continuity between adjacent components.
2. **Post-tensioning existing beams, columns, or joints using external post-tensioned reinforcement.** Post-tensioned reinforcement should be unbonded within a distance equal to twice the effective depth from sections where inelastic action is expected. Anchorages should be located away from regions where inelastic action is anticipated, and should be designed considering possible force variations due to earthquake loading.
3. **Modification of the element by selective material removal from the existing element.** Examples include: (1) where nonstructural elements or components interfere with the frame, removing or separating the nonstructural elements or components to eliminate the interference; (2) weakening, due to removal of concrete or severing of longitudinal reinforcement, to change response mode from a nonductile mode to a more ductile mode (e.g., weakening of beams to promote formation of a strong-column, weak-beam system); and (3) segmenting walls to change stiffness and strength.
4. **Improvement of deficient existing reinforcement details.** Removal of cover concrete for modification of existing reinforcement details should avoid damage to core concrete and the bond between existing reinforcement and core concrete. New cover concrete should be designed and constructed to achieve fully composite action with the existing materials.
5. **Changing the building system to reduce the demands on the existing element.** Examples include addition of supplementary lateral-force-resisting elements such as walls or buttresses, seismic isolation, and mass reduction.
6. **Changing the frame element to a shear wall, infilled frame, or braced frame element by addition of new material.** Connections between new and existing materials should be designed to transfer the forces anticipated for the design load combinations. Where the existing concrete frame columns and beams act as boundary elements and collectors for the new shear wall or braced frame, these should be checked for adequacy, considering strength, reinforcement development, and deformability. Diaphragms, including drag struts and collectors, should be evaluated and, if necessary, rehabilitated to ensure a complete load path to the new shear wall or braced frame element.

6.5.3 Post-Tensioned Concrete Beam-Column Moment Frames

6.5.3.1 General Considerations

The analytical model for a post-tensioned concrete beam-column frame element shall be established following the criteria specified in Section 6.5.2.1 for reinforced concrete beam-column moment frames. In addition to potential failure modes described in Section 6.5.2.1, the analysis model shall consider potential failure of tendon anchorages.

The analysis procedures described in Chapter 3 shall apply to frames with post-tensioned beams satisfying the following conditions:

1. The average prestress, f_{pc}, calculated for an area equal to the product of the shortest cross-sectional dimension and the perpendicular cross-sectional dimension of the beam, does not exceed the greater of 750 psi or $f'_c/12$ at locations of nonlinear action.

2. Prestressing tendons do not provide more than one-quarter of the strength for both positive moments and negative moments at the joint face.

3. Anchorages for tendons are demonstrated to have performed satisfactorily for seismic loadings in compliance with the requirements of *ACI 318*. These anchorages occur outside hinging areas or joints, except in existing components where experimental evidence demonstrates that the connection will meet the performance objectives under design loadings.

Alternative procedures shall be provided where these conditions are not satisfied.

6.5.3.2 Stiffness

6.5.3.2.1 Linear Static and Dynamic Procedures

Beams shall be modeled considering flexural and shear stiffnesses, including the effect of the slab acting as a flange in monolithic and composite construction. Columns shall be modeled considering flexural, shear, and axial stiffnesses. Joints shall be modeled as either stiff or rigid components. Effective stiffnesses shall be according to Section 6.4.1.2.

6.5.3.2.2 Nonlinear Static Procedure

Nonlinear load-deformation relations shall comply with the requirements of Section 6.4.1.2 and the reinforced concrete frame requirements of Section 6.5.2.2.2.

Values of the generalized deformation at points *B*, *C*, and *D* in Figure 6-1 shall be either derived from experiments or approved rational analyses, and shall take into account the interactions between flexure, axial load, and shear. Alternatively, where the generalized deformation is taken as rotation in the flexural plastic hinge zone, and where the three conditions of Section 6.5.3.1 are satisfied, beam plastic hinge rotation capacities shall be as defined by Table 6-7. Columns and joints shall be modeled as described in Section 6.5.2.2.

6.5.3.2.3 Nonlinear Dynamic Procedure

For the NDP, the complete hysteretic behavior of each component shall be modeled using properties verified by experimental evidence. The relation of Figure 6-1 shall be taken to represent the envelope relation for the analysis. Unloading and reloading properties shall represent significant stiffness and strength degradation characteristics as influenced by prestressing.

6.5.3.3 Strength

Component strengths shall be computed according to the general requirements of Sections 6.4.2 and the additional requirements of Section 6.5.2.3. Effects of prestressing on strength shall be considered.

For deformation-controlled actions, prestress shall be assumed to be effective for the purpose of determining the maximum actions that may be developed associated with nonlinear response of the frame. For force-controlled actions, the effects on strength of prestress loss shall also be considered as a design condition, where these losses are possible under design load combinations including inelastic deformation reversals.

6.5.3.4 Acceptance Criteria

Acceptance criteria for post-tensioned concrete beam-column moment frames shall follow the criteria for reinforced concrete beam-column frames specified in Section 6.5.2.4.

Modeling parameters and acceptance criteria shall be based on Tables 6-7 through 6-9 and 6-11 through 6-13.

6.5.3.5 Rehabilitation Measures

Post-tensioned concrete beam-column moment frame components that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 6.4.7 and other provisions of this standard.

C6.5.3.5 Rehabilitation Measures

The rehabilitation measures described in C6.5.2.5 for reinforced concrete beam-column moment frames may also be effective in rehabilitating post-tensioned concrete beam-column moment frames.

6.5.4 Slab-Column Moment Frames

6.5.4.1 General Considerations

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

The analytical model that represents the slab-column frame, using either line elements with properties concentrated at component centerlines or a combination of line elements (to represent columns) and plate-bending elements (to represent the slab), based on any of the following approaches, shall be permitted.

1. An effective beam width model, in which the columns and slabs are represented by line elements that are rigidly interconnected at the slab-column joint.

2. An equivalent frame model in which the columns and slabs are represented by line elements that are interconnected by connection springs.

3. A finite element model in which the columns are represented by line elements and the slab is represented by plate-bending elements.

In any model, the effects of changes in cross section, including slab openings, shall be considered.

The connection between the columns and foundation shall be modeled based on the details of the column-foundation connection and rigidity of the foundation-soil system.

Action of the slab as a diaphragm interconnecting vertical elements shall be represented.

In the design model, inelastic deformations in primary components shall be restricted to flexure in slabs and columns, plus nonlinear response in slab-column connections. Other inelastic deformations shall be permitted as part of the design in secondary components. Acceptance criteria shall be as specified in Section 6.5.4.4.

6.5.4.2 Stiffness

6.5.4.2.1 Linear Static and Dynamic Procedures

Slabs shall be modeled considering flexural, shear, and torsional (in the slab adjacent to the column) stiffnesses. Columns shall be modeled considering flexural, shear, and axial stiffnesses. Joints shall be modeled as either stiff or rigid components. The effective stiffnesses of components shall be determined according to the general principles of Section 6.4.1.2, but adjustments on the basis of experimental evidence shall be permitted.

6.5.4.2.2 Nonlinear Static Procedure

Nonlinear load-deformation relations shall comply with the requirements of Section 6.4.1.2.

Slabs and columns shall be modeled using concentrated plastic hinge models, distributed plastic hinge models, or other models whose behavior has been demonstrated to adequately represent behavior of reinforced concrete slab and column components subjected to lateral loading. The model shall be capable of representing inelastic response along the component length, except where it is shown by equilibrium that yielding is restricted to the component ends. Slab- column connections shall be modeled separately from the slab and column components in order to identify potential failure in shear and moment transfer; alternatively, the potential for connection failure shall be otherwise checked as part of the analysis. Where nonlinear response is expected in a mode other than flexure, the model shall be established to represent these effects.

Monotonic load-deformation relations shall be according to the generalized relation shown in Figure 6-1, with definitions according to Section 6.5.2.2.2. The overall load-deformation relation shall be established so that the maximum resistance is consistent with the design strength specifications of Sections 6.4.2 and 6.5.4.3. Where the generalized deformation shown in Figure 6-1 is taken as the flexural plastic hinge rotation for the column, the plastic hinge rotation capacities shall be as defined by Table 6-8. Where the generalized deformation shown in Figure 6-1 is taken as the rotation of the slab-column connection, the plastic rotation capacities shall be as defined by Table 6-14.

6.5.4.2.3 Nonlinear Dynamic Procedure

The requirements of Sections 6.4.2 6.5.2.2.3 for reinforced concrete beam-column moment frames shall apply to slab-column moment frames.

6.5.4.3 Strength

Component strengths shall be computed according to the general requirements of Sections 6.4.2, as modified in this section.

The maximum component strength shall be determined considering potential failure in flexure, axial load, shear, torsion, development, and other actions at all points along the length of the component under the actions of design gravity and earthquake load combinations. The strength of slab-column connections also shall be determined and incorporated in the analytical model.

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

For columns, the evaluation of shear strength according to Section 6.5.2.3 shall be permitted.

Shear and moment transfer strength of the slab-column connection shall be calculated considering the combined action of flexure, shear, and torsion acting in the slab at the connection with the column. The procedures described below shall be permitted to satisfy this requirement.

For interior connections without transverse beams, and for exterior connections with moment about an axis perpendicular to the slab edge, the shear and moment transfer strength calculated as the minimum of the following strengths shall be permitted:

1. The strength calculated considering eccentricity of shear on a slab critical section due to combined shear and moment, as prescribed in *ACI 318*.

2. The moment transfer strength equal to $\Sigma M_n/\gamma_f$, where ΣM_n = the sum of positive and negative flexural strengths of a section of slab between lines that are two and one-half slab or drop panel thicknesses ($2.5h$) outside opposite faces of the column or capital; γ_f = the fraction of the moment resisted by flexure per *ACI 318*; and h = slab thickness.

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

6.5.4.4 Acceptance Criteria

6.5.4.4.1 Linear Static and Dynamic Procedures

All component actions shall be classified as being either deformation-controlled or force-controlled, as defined in Section 2.4.4. In primary components, deformation-controlled actions shall be restricted to flexure in slabs and columns, and shear and moment transfer in slab-column connections. In secondary components, deformation-controlled actions shall also be permitted in shear and reinforcement development, as identified in Table 6-15. All other actions shall be defined as being force-controlled actions.

Design actions on components shall be determined as prescribed in Chapter 3. Where the calculated DCR values exceed unity, the following design actions shall be determined using limit analysis principles as prescribed in Chapter 3: (1) moments, shears, torsions, and development and splice actions corresponding to development of component strength in slabs and columns; and (2) axial load in columns, considering likely plastic action in components above the level in question.

Design actions shall be compared with design strengths in accordance with Section 3.4.2.2, Tables 6-12 and 6-15. Those components that satisfy Equations (3-20) and (3-21) shall satisfy the performance criteria. Components that reach their design strengths shall be further evaluated according to this section to determine performance acceptability.

Where the average of the DCRs of columns at a level exceeds the average value of slabs at the same level, and exceeds the greater of 1.0 and $m/2$, the element shall be defined as a weak story element and shall be evaluated by the procedure for weak story elements described in Section 6.5.2.4.1.

6.5.4.4.2 Nonlinear Static and Dynamic Procedures

In the design model, inelastic response shall be restricted to those components and actions listed in Tables 6-8 and 6-14, except where it is demonstrated by experimental evidence and analysis that other inelastic action is acceptable for the selected performance levels.

Calculated component actions shall satisfy the requirements of Chapter 3. Maximum permissible inelastic deformations shall be as listed in Tables 6-8 and 6-14. Where inelastic action is indicated for a component or action not listed in these tables, the performance shall be deemed unacceptable. Alternative approaches or values shall be permitted where justified by experimental evidence and analysis.

6.5.4.5 Rehabilitation Measures

Reinforced concrete slab-column, beam-column moment frame components that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 6.4.7 and other provisions of this standard.

C6.5.4.5 Rehabilitation Measures

The rehabilitation measures described in C6.5.2.5 for reinforced concrete beam-column moment frames may also be effective in rehabilitating reinforced concrete slab-column moment frames.

Table 6-14 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Two-way Slabs and Slab-Column Connections

		Modeling Parameters[4]			Acceptance Criteria[4]				
					Plastic Rotation Angle, radians				
					Performance Level				
						Component Type			
		Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
Conditions		a	b	c	IO	LS	CP	LS	CP
i. Slabs controlled by flexure, and slab-column connections[1]									
$\frac{V_g}{V_o}$ [2]	Continuity Reinforcement[3]								
≤ 0.2	Yes	0.02	0.05	0.2	0.01	0.015	0.02	0.03	0.05
≥ 0.4	Yes	0.0	0.04	0.2	0.0	0.0	0.0	0.03	0.04
≤ 0.2	No	0.02	0.02	–	0.01	0.015	0.02	0.015	0.02
≥ 0.4	No	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0
ii. Slabs controlled by inadequate development or splicing along the span[1]									
		0.0	0.02	0.0	0.0	0.0	0.0	0.01	0.02
iii. Slabs controlled by inadequate embedment into slab-column joint[1]									
		0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions i, ii, and iii occurs for a given component, use the minimum appropriate numerical value from the table.
2. V_g = the gravity shear acting on the slab critical section as defined by *ACI 318;* V_o = the direct punching shear strength as defined by *ACI 318.*
3. Under the heading "Continuity Reinforcement," use "Yes" where at least one of the main bottom bars in each direction is effectively continuous through the column cage. Where the slab is post-tensioned, use "Yes" where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, use "No."
4. Linear interpolation between values shown in the table shall be permitted.

Table 6-15 Numerical Acceptance Criteria for Linear Procedures— Two-way Slabs and Slab-Column Connections

Conditions		*m*-factors				
		Performance Level				
		IO	Component Type			
			Primary		Secondary	
			LS	CP	LS	CP
i. Slabs controlled by flexure, and slab-column connections[1]						
$\frac{V_g}{V_o}$ [2]	Continuity Reinforcement[3]					
≤ 0.2	Yes	2	2	3	3	4
≥ 0.4	Yes	1	1	1	2	3
≤ 0.2	No	2	2	3	2	3
≥ 0.4	No	1	1	1	1	1
ii. Slabs controlled by inadequate development or splicing along the span[1]						
		–	–	–	3	4
iii. Slabs controlled by inadequate embedment into slab-column joint[1]						
		2	2	3	3	4

1. When more than one of the conditions i, ii, and iii occurs for a given component, use the minimum appropriate numerical value from the table.
2. V_g = the gravity shear acting on the slab critical section as defined by *ACI 318;* V_o = the direct punching shear strength as defined by *ACI 318.*
3. Under the heading "Continuity Reinforcement," use "Yes" where at least one of the main bottom bars in each direction is effectively continuous through the column cage. Where the slab is post-tensioned, use "Yes" where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, use "No."

6.6 Precast Concrete Frames

6.6.1 Types of Precast Concrete Frames

Precast concrete frames shall be defined as those elements that are constructed from individually made beams and columns that are assembled to create gravity-load-carrying systems. These systems shall include those that are considered in design to resist design lateral loads, and those that are considered in design as secondary elements that do not resist design lateral loads but must resist the effects of deformations resulting from design lateral loads.

The provisions of this section shall apply to precast concrete frames that emulate cast-in-place moment frames defined in Section 6.6.1.1, precast concrete beam-column moment frames constructed with dry joints defined in Section 6.6.1.2, and precast concrete frames not expected to directly resist lateral loads defined in Section 6.6.1.3.

6.6.1.1 Precast Concrete Frames that Emulate Cast-in-Place Moment Frames

Precast concrete frames that emulate cast-in-place moment frames shall be defined as those precast beam-column systems that are interconnected using reinforcement and wet concrete to create a system that resists lateral loads in a manner similar to cast-in-place concrete systems.

C6.6.1.1 Precast Concrete Frames that Emulate Cast-in-Place Moment Frames

These systems are recognized and accepted by *FEMA 222A*, and are based on *ACI 318*, which requires safety and serviceability levels expected from monolithic construction. There is insufficient research and testing data at this time to qualify systems assembled using dry joints as emulated moment frames.

6.6.1.2 Precast Concrete Moment Frames Constructed with Dry Joints

Frames of this classification shall be assembled using dry joints (connections are made by bolting, welding, post-tensioning, or other similar means) in a way that results in significant lateral force resistance in the framing element. Frames of this classification resist lateral loads either acting alone, or acting in conjunction with shear walls, braced frames, or other lateral load resisting elements in a dual system.

C6.6.1.2 Precast Concrete Moment Frames Constructed with Dry Joints

The appendix to Chapter 6 of *FEMA 222A* contains a trial version of code provisions for new construction of this classification.

6.6.1.3 Precast Concrete Frames Not Expected to Resist Lateral Load Directly

Frames of this classification shall be assembled using dry joints in a way that does not result in significant lateral force resistance in the framing element. Shear walls, braced frames, or moment frames provide the entire lateral load resistance, with the precast concrete frame system deforming in a manner that is compatible with the structure as a whole.

6.6.2 Precast Concrete Frames that Emulate Cast-in-Place Moment Frames

6.6.2.1 General Considerations

The analytical model for a frame element of this classification shall represent strength, stiffness, and deformation capacity of beams, columns, beam-column joints, and other components of the frame. Potential failure in flexure, shear, and reinforcement development at any section along the component length shall be considered. Interaction with other elements, including nonstructural elements and components, shall be included. All other considerations of Section 6.5.2.1 shall be taken into account. In addition, the effects of shortening due to creep, and other effects of prestressing and post-tensioning on member behavior, shall be evaluated.

6.6.2.2 Stiffness

Stiffness for analysis shall be as defined in Section 6.5.2.2. The effects of prestressing shall be considered when computing the effective stiffness values using Table 6-5.

6.6.2.3 Strength

Component strength shall be computed according to the requirements of Section 6.5.2.3, with the additional requirement that the following effects be included in the analysis:

1. Effects of prestressing that are present, including, but not limited to, reduction in rotation capacity, secondary stresses induced, and amount of effective prestress force remaining;
2. Effects of construction sequence, including the possibility of construction of the moment connections occurring after portions of the structure are subjected to dead loads;
3. Effects of restraint due to interaction with interconnected wall or brace components.

6.6.2.4 Acceptance Criteria

Acceptance criteria for precast concrete frames that emulate cast-in-place moment frames shall be as specified in Section 6.5.2.4, except that the factors defined in Section 6.5.2.3 shall also be considered.

6.6.2.5 Rehabilitation Measures

Precast concrete frame components that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 6.4.7 and other provisions of this standard.

C6.6.2.5 Rehabilitation Measures

The rehabilitation measures described in C6.5.2.5 for reinforced concrete beam-column moment frames may also be effective in rehabilitating precast concrete frames that emulate cast-in-place moment frames. When installing new elements, components, or materials to the existing system, existing prestressing strands should be protected.

6.6.3 Precast Concrete Moment Frames Constructed with Dry Joints

6.6.3.1 General Considerations

The analysis model for precast concrete moment frames constructed with dry joints shall be as specified in Section 6.5.2.1 for reinforced concrete beam-column moment frames, with additional consideration of the special nature of the dry joints used in assembling the precast system. Where connections yield under design lateral loads, the analysis model shall take this into account.

C6.6.3.1 General Considerations

The requirements given in the appendix to Chapter 6 of *FEMA 222A* for this type of structural system should be adhered to where possible, and the philosophy and approach should be employed when designing new connections for existing components.

6.6.3.2 Stiffness

Stiffness for analysis shall be as defined in Section 6.6.2.2. Flexibilities associated with connections shall be included in the analytical model.

6.6.3.3 Strength

Component strength shall be computed according to the requirements of Section 6.6.2.3, with the additional requirement that connection strength shall comply with the requirements of Section 6.4.6.

C6.6.3.3 Strength

In addition to complying with the provisions of Section 6.6.3.3, the connections should comply with the requirements given in the appendix to Chapter 6 of *FEMA 222A*.

6.6.3.4 Acceptance Criteria

Acceptance criteria for precast concrete moment frames constructed with dry joints shall be as specified in Section 6.6.2.4, with the additional requirement that the connections comply with the requirements of Section 6.4.6.

C6.6.3.4 Acceptance Criteria

In addition to complying with the provisions of Section 6.6.3.4, the acceptance criteria should comply with the requirements given in Section 6A.4 of the appendix to Chapter 6 of *FEMA 222A*.

6.6.3.5 Rehabilitation Measures

Precast concrete moment frame components that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 6.4.7 and other provisions of this standard.

C6.6.3.5 Rehabilitation Measures

The rehabilitation measures described in C6.5.2.5 for reinforced concrete beam-column moment frames may also be effective in rehabilitating precast concrete frames constructed with dry joints. When installing new elements, components, or materials to the existing system, existing prestressing strands should be protected.

6.6.4 Precast Concrete Frames Not Expected to Resist Lateral Loads Directly

6.6.4.1 General Considerations

The analytical model for precast concrete frames that are not expected to resist lateral loads directly shall comply with the requirements of Section 6.6.3.1 and shall include the effects of deformations that are calculated to occur under the design earthquake loadings.

6.6.4.2 Stiffness

The analytical model shall either include realistic lateral stiffness of these frames to evaluate the effects of deformations under lateral loads or, if the lateral stiffness is ignored in the analytical model, the effects of calculated building drift on these frames shall be evaluated separately. The analytical model shall consider the negative effects of connection stiffness on component response where that stiffness results in actions that may cause component failure.

C6.6.4.2 Stiffness

The stiffness used in the analysis should consider possible resistance that may develop under lateral deformation. In some cases it may be appropriate to assume zero lateral stiffness. However, the Northridge earthquake graphically demonstrated that there are few instances where the precast column can be considered to be completely pinned top and bottom, and as a consequence, not resist any shear from building drift. Several parking structures collapsed as a result of this defect. Conservative assumptions should be made.

6.6.4.3 Strength

Component strength shall be computed according to the requirements of Section 6.6.3.3. All components shall have sufficient strength and ductility to transmit induced forces from one member to another and to the designated lateral-force-resisting system.

6.6.4.4 Acceptance Criteria

Acceptance criteria for components in precast concrete frames not expected to directly resist lateral loads shall be as specified in Section 6.6.3.4. All moments, shear forces, and axial loads induced through the deformation of the structural system shall be checked for acceptability by appropriate criteria in the referenced section.

6.6.4.5 Rehabilitation Measures

Precast concrete frame components that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 6.4.7 and other provisions of this standard.

C6.6.4.5 Rehabilitation Measures

The rehabilitation measures described in C6.5.2.5 for reinforced concrete beam-column moment frames may also be effective in rehabilitating precast concrete frames not expected to resist lateral loads directly. When installing new elements, components, or materials to the existing system, existing prestressing strands should be protected.

6.7 Concrete Frames with Infills

6.7.1 Types of Concrete Frames with Infills

Concrete frames with infills are elements with complete gravity-load-carrying concrete frames infilled with masonry or concrete, constructed in such a way that the infill and the concrete frame interact when subjected to vertical and lateral loads.

Isolated infills are infills isolated from the surrounding frame complying with the minimum gap requirements specified in Section 7.5.1. If all infills in a frame are isolated infills, the frame shall be analyzed as an isolated frame according to provisions given elsewhere in this chapter, and the isolated infill panels shall be analyzed according to the requirements of Chapter 7.

The provisions of Section 6.7 shall apply to concrete frames with existing infills, frames that are rehabilitated by addition or removal of material, and concrete frames that are rehabilitated by the addition of new infills.

6.7.1.1 Types of Frames

The provisions of Section 6.7 shall apply to concrete frames as defined in Sections 6.5, 6.6, and 6.10, where those frames interact with infills.

6.7.1.2 Masonry Infills

The provisions of Section 6.7 shall apply to masonry infills as defined in Chapter 7, where those infills interact with concrete frames.

6.7.1.3 Concrete Infills

The provisions of Section 6.7 shall apply to concrete infills that interact with concrete frames, where the infills were constructed to fill the space within the bay of a complete gravity frame without special provision for continuity from story to story. The concrete of the infill shall be evaluated separately from the concrete of the frame.

C6.7.1.3 Concrete Infills

The construction of concrete-infilled frames is very similar to that of masonry-infilled frames, except that the infill is of concrete instead of masonry units. In older existing buildings, the concrete infill commonly contains nominal reinforcement, which is unlikely to extend into the surrounding frame. The concrete is likely to be of lower quality than that used in the frame, and should be investigated separately from investigations of the frame concrete.

6.7.2 Concrete Frames with Masonry Infills

6.7.2.1 General Considerations

The analytical model for a concrete frame with masonry infills shall represent strength, stiffness, and deformation capacity of beams, slabs, columns, beam-column joints, masonry infills, and all connections and components of the element. Potential failure in flexure, shear, anchorage, reinforcement development, or crushing at any section shall be considered. Interaction with other nonstructural elements and components shall be included.

For a concrete frame with masonry infill resisting lateral forces within its plane, modeling of the response using a linear elastic model shall be permitted provided that the infill will not crack when subjected to design lateral forces. If the infill will not crack when subjected to design lateral forces, modeling the assemblage of frame and infill as a homogeneous medium shall be permitted.

For a concrete frame with masonry infills that will crack when subjected to design lateral forces, modeling of the response using a diagonally braced frame model, in which the columns act as vertical chords, the beams act as horizontal ties, and the infill acts as an equivalent compression strut, shall be permitted. Requirements for the equivalent compression strut analogy shall be as specified in Chapter 7.

Frame components shall be evaluated for forces imparted to them through interaction of the frame with the infill, as specified in Chapter 7. In frames with full-height masonry infills, the evaluation shall include the effect of strut compression forces applied to the column and beam, eccentric from the beam-column joint. In frames with partial-height masonry infills, the evaluation shall include the reduced effective length of the columns above the infilled portion of the bay.

C6.7.2.1 General Considerations

The design professional is referred to *FEMA 274* and *FEMA 306* for additional information regarding the behavior of masonry infills.

6.7.2.2 Stiffness

6.7.2.2.1 Linear Static and Dynamic Procedures

In frames having infills in some bays and no infill in other bays, the restraint of the infill shall be represented as described in Section 6.7.2.1, and the noninfilled bays shall be modeled as frames as specified in appropriate portions of Sections 6.5, 6.6, and 6.10. Where infills create a discontinuous wall, the effects of the discontinuity on overall building performance shall be evaluated. Effective stiffnesses shall be in accordance with Section 6.4.1.2.

6.7.2.2.2 Nonlinear Static Procedure

Nonlinear load-deformation relations for use in analysis by Nonlinear Static Procedure (NSP) shall follow the requirements of Section 6.4.1.2.2.

Modeling beams and columns using nonlinear truss elements shall be permitted in infilled portions of the frame. Beams and columns in noninfilled portions of the frame shall be modeled using the relevant specifications of Sections 6.5, 6.6, and 6.10. The model shall be capable of representing inelastic response along the component lengths.

Monotonic load-deformation relations shall be according to the generalized relation shown in Figure 6-1, except different relations shall be permitted where verified by tests. Numerical quantities in Figure 6-1 shall be derived from tests or by analyses procedures as specified in Chapter 2, and shall take into account the interactions between frame and infill components. Alternatively, the following procedure shall be permitted for monolithic reinforced concrete frames.

1. For beams and columns in noninfilled portions of frames, where the generalized deformation is taken as rotation in the flexural plastic hinge zone, the plastic hinge rotation capacities shall be as defined by Table 6-18.

2. For masonry infills, the generalized deformations and control points shall be as defined in Chapter 7.

3. For beams and columns in infilled portions of frames, where the generalized deformation is taken as elongation or compression displacement of the beams or columns, the tension and compression strain capacities shall be as specified in Table 6-16.

6.7.2.2.3 Nonlinear Dynamic Procedure

Nonlinear load-deformation relations for use in analysis by Nonlinear Dynamic Procedure (NDP) shall model the complete hysteretic behavior of each component using properties verified by tests. Unloading and reloading properties shall represent stiffness and strength degradation characteristics.

6.7.2.3 Strength

Strengths of reinforced concrete components shall be calculated according to the general requirements of Sections 6.4.2, as modified by other specifications of this chapter. Strengths of masonry infills shall be calculated according to the requirements of Chapter 7. Strength calculations shall consider:

1. Limitations imposed by beams, columns, and joints in nonfilled portions of frames.

2. Tensile and compressive capacity of columns acting as boundary elements of infilled frames.

3. Local forces applied from the infill to the frame.

4. Strength of the infill.

5. Connections with adjacent elements.

6.7.2.4 Acceptance Criteria

6.7.2.4.1 Linear Static and Dynamic Procedures

All component actions shall be classified as either deformation-controlled or force-controlled, as defined in Section 2.4.4. In primary components, deformation-controlled actions shall be restricted to flexure and axial actions in beams, slabs, and columns, and lateral deformations in masonry infill panels. In secondary components, deformation-controlled actions shall be restricted to those actions identified for the isolated frame in Sections 6.5, 6.6, and 6.10, as appropriate, and for the masonry infill in Section 7.5.

Design actions shall be determined as prescribed in Chapter 3. Where calculated DCR values exceed unity, the following design actions shall be determined using limit analysis principles as prescribed in Chapter 3: (1) moments, shears, torsions, and development and splice actions corresponding to development of component strength in beams, columns, or masonry infills; and (2) column axial load corresponding to development of the flexural capacity of the infilled frame acting as a cantilever wall.

Design actions shall be compared with design strengths in accordance with Section 3.4.2.2.

Values of *m*-factors shall be as specified in Section 7.5.2.3 for masonry infills; applicable portions of Sections 6.5, 6.6, and 6.10 for concrete frames; and Table 6-17 for columns modeled as tension and compression chords. Those components that have design actions less than design strengths shall be assumed to satisfy the performance criteria for those components.

6.7.2.4.2 Nonlinear Static and Dynamic Procedures

In the design model, inelastic response shall be restricted to those components and actions that are permitted for isolated frames as specified in Sections 6.5, 6.6, and 6.10, as well as for masonry infills as specified in Section 7.5.

Table 6-16 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Infilled Frames

Conditions	Modeling Parameters[4]			Acceptance Criteria				
				Total Strain				
				Performance Level				
					Component Type			
	Total Strain		Residual Strength Ratio		Primary		Secondary	
	d	e	c	IO	LS	CP	LS	CP
i. Columns modeled as compression chords[3]								
Columns confined along entire length[2]	0.02	0.04	0.4	0.003	0.015	0.020	0.03	0.04
All other cases	0.003	0.01	0.2	0.002	0.002	0.003	0.01	0.01
ii. Columns modeled as tension chords[3]								
Columns with well-confined splices, or no splices	0.05	0.05	0.0	0.01	0.03	0.04	0.04	0.05
All other cases	See note 1	0.03	0.2	See note 1			0.02	0.03

1. Potential for splice failure shall be evaluated directly to determine the modeling and acceptability criteria. For these cases, refer to the generalized procedure of Sections 6.4.2. For primary actions, Collapse Prevention Performance Level shall be defined as the deformation at which strength degradation begins. Life Safety Performance Level shall be taken as three-quarters of that value.
2. A column shall be permitted to be considered to be confined along its entire length when the quantity of hoops along the entire story height including the joint is equal to three-quarters of that required by *ACI 318* for boundary elements of concrete shear walls. The maximum longitudinal spacing of sets of hoops shall not exceed either $h/3$ or $8d_b$.
3. If load reversals will result in both conditions i and ii applying to a single column, both conditions shall be checked.
4. Interpolation shall not be permitted.

Calculated component actions shall satisfy the requirements of Section 3.4.3.2, and shall not exceed the numerical values listed in Table 6-16, the relevant tables for isolated frames given in Sections 6.5, 6.6, and 6.10, and the relevant tables for masonry infills given in Chapter 7. Component actions not listed in Tables 6-7 through 6-9 shall be treated as force-controlled. Alternative approaches or values shall be permitted where justified by experimental evidence and analysis.

6.7.2.5 Rehabilitation Measures

Concrete frames with masonry infill that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 6.4.7 and other provisions of this standard.

Table 6-17 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Infilled Frames

Conditions	*m*-factors[3]				
	Performance Level				
		Component Type			
		Primary		Secondary	
	IO	LS	CP	LS	CP
i. Columns modeled as compression chords[2]					
Columns confined along entire length[1]	1	3	4	4	5
All other cases	1	1	1	1	1
ii. Columns modeled as tension chords[2]					
Columns with well-confined splices, or no splices	3	4	5	5	6
All other cases	1	2	2	3	4

1. A column may be considered to be confined along its entire length when the quantity of hoops along the entire story height including the joint is equal to three-quarters of that required by *ACI 318* for boundary elements of concrete shear walls. The maximum longitudinal spacing of sets of hoops shall not exceed either $h/3$ or $8d_b$.
2. If load reversals will result in both conditions i and ii applying to a single column, both conditions shall be checked.
3. Interpolation shall not be permitted.

C6.7.2.5 Rehabilitation Measures

The rehabilitation measures described in relevant commentary of Sections 6.5, 6.6, and 6.10 for isolated frames, and rehabilitation measures described in relevant commentary or Section 7.5 for masonry infills, may also be effective in rehabilitating concrete frames with masonry infills. The design professional is referred to *FEMA 308* for further information in this regard. In addition, the following rehabilitation measures may be effective in rehabilitating concrete frames with infills:

1. **Post-tensioning existing beams, columns, or joints using external post-tensioned reinforcement.** Vertical post-tensioning may be effective in increasing tensile capacity of columns acting as boundary zones. Anchorages should be located away from regions where inelastic action is anticipated, and should be designed considering possible force variations due to earthquake loading.
2. **Modification of the element by selective material removal from the existing element.** Either the infill should be completely removed from the frame, or gaps should be provided between the frame and the infill. In the latter case, the gap requirements of Chapter 7 should be satisfied.
3. **Changing the building system to reduce the demands on the existing element.** Examples include the addition of supplementary lateral-force-resisting elements such as walls, steel braces, or buttresses; seismic isolation; and mass reduction.

6.7.3 Concrete Frames with Concrete Infills

6.7.3.1 General Considerations

The analytical model for a concrete frame with concrete infills shall represent the strength, stiffness, and deformation capacity of beams, slabs, columns, beam-column joints, concrete infills, and all connections and components of the elements. Potential failure in flexure, shear, anchorage, reinforcement development, or crushing at any section shall be considered. Interaction with other nonstructural elements and components shall be included.

The analytical model shall be established considering the relative stiffness and strength of the frame and the infill, as well as the level of deformations and associated damage. For low deformation levels, and for cases where the frame is relatively flexible, the infilled frame shall be permitted to be modeled as a shear wall, with openings modeled where they occur. In other cases, the frame-infill system shall be permitted to be modeled using a braced-frame analogy such as that described for concrete frames with masonry infills in Section 6.7.2.

Frame components shall be evaluated for forces imparted to them through interaction of the frame with the infill as specified in Chapter 7. In frames with full-height infills, the evaluation shall include the effect of strut compression forces applied to the column and beam eccentric from the beam-column joint. In frames with partial-height infills, the evaluation shall include the reduced effective length of the columns above the infilled portion of the bay.

In frames having infills in some bays and no infills in other bays, the restraint of the infill shall be represented as described in this section, and the noninfilled bays shall be modeled as frames as specified in appropriate portions of Sections 6.5, 6.6, and 6.10. Where infills create a discontinuous wall, the effects of the discontinuity on overall building performance shall be evaluated.

6.7.3.2 Stiffness

6.7.3.2.1 Linear Static and Dynamic Procedures

Effective stiffnesses shall be calculated according to the principles of Section 6.4.1.2 and the procedure of Section 6.7.2.2.1.

6.7.3.2.2 Nonlinear Static Procedure

Nonlinear load-deformation relations for use in analysis by Nonlinear Static Procedure (NSP) shall follow the requirements of Section 6.4.1.2.2.

Monotonic load-deformation relations shall be according to the generalized relation shown in Figure 6-1, except different relations shall be permitted where verified by tests. Numerical quantities in Figure 6-1 shall be derived from tests or by analysis procedures specified in Section 2.8, and shall take into account the interactions between frame and infill components. Alternatively, the procedure of Section 6.7.2.2.2 shall be permitted for the development of nonlinear modeling parameters for concrete frames with concrete infills.

6.7.3.2.3 Nonlinear Dynamic Procedure

Nonlinear load-deformation relations for use in analysis by Nonlinear Dynamic Procedure (NDP) shall model the complete hysteretic behavior of each component using properties verified by tests. Unloading and reloading properties shall represent stiffness and strength degradation characteristics.

6.7.3.3 Strength

Strengths of reinforced concrete components shall be calculated according to the general requirements of Sections 6.4.2, as modified by other specifications of this chapter. Strength calculations shall consider:

1. Limitations imposed by beams, columns, and joints in unfilled portions of frames.
2. Tensile and compressive capacity of columns acting as boundary elements of infilled frames.
3. Local forces applied from the infill to the frame.
4. Strength of the infill.
5. Connections with adjacent elements.

Strengths of existing concrete infills shall be determined considering shear strength of the infill panel. For this calculation, procedures specified in Section 6.8.2.3 shall be used for calculation of the shear strength of a wall segment.

Where the frame and concrete infill are assumed to act as a monolithic wall, flexural strength shall be based on continuity of vertical reinforcement in both (1) the columns acting as boundary elements, and (2) the infill wall, including anchorage of the infill reinforcement in the boundary frame.

6.7.3.4 Acceptance Criteria

The acceptance criteria for concrete frames with concrete infills shall comply with relevant acceptance criteria of Sections 6.7.2.4, 6.8, and 6.9.

6.7.3.5 Rehabilitation Measures

Concrete frames with concrete infills that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 6.4.7 and other provisions of this standard.

C6.7.3.5 Rehabilitation Measures

Rehabilitation measures described in C6.7.2.5 for concrete frames with masonry infills may also be effective in rehabilitating concrete frames with concrete infills. In addition, application of shotcrete to the face of an existing wall to increase the thickness and shear strength may be effective. For this purpose, the face of the existing wall should be roughened, a mat of reinforcing steel should be doweled into the existing structure, and shotcrete should be applied to the desired thickness. The design professional is referred to *FEMA 308* for further information regarding rehabilitation of concrete frames with concrete infill.

6.8 Concrete Shear Walls

6.8.1 Types of Concrete Shear Walls and Associated Components

The provisions of Section 6.8 shall apply to all shear walls in all types of structural systems that incorporate shear walls. This includes isolated shear walls, shear walls used in dual (wall-frame) systems, coupled shear walls, and discontinuous shear walls. Shear walls shall be permitted to be considered as solid walls if they have openings that do not significantly influence the strength or inelastic behavior of the wall. Perforated shear walls shall be defined as walls having a regular pattern of openings in both horizontal and vertical directions that creates a series of pier and deep beam elements referred to as wall segments.

Coupling beams, and columns that support discontinuous shear walls, shall comply with provisions of Section 6.8.2. These special frame components associated with shear walls shall be exempted from the provisions for beams and columns of frame elements covered in Section 6.5.

C6.8.1 Types of Concrete Shear Walls and Associated Components

Concrete shear walls are planar vertical elements or combinations of interconnected planar elements that serve as lateral-load-resisting elements in concrete structures. Shear walls (or wall segments) shall be considered slender if their aspect ratio (height/length) is $\geq$3.0, and shall be considered short or squat if their aspect ration is $\leq$1.5. Slender shear walls are normally controlled by flexural behavior; short walls are normally controlled by shear behavior. The response of walls with intermediate aspect ratios is influenced by both flexure and shear.

Identification of component types in concrete shear wall elements depends, to some degree, on the relative strengths of the wall segments. Vertical segments are often termed piers, while horizontal segments may be called coupling beams or spandrels. The design professional is referred to *FEMA 306* for additional information regarding the behavior of concrete wall components. Selected information from *FEMA 306* has been reproduced in the commentary of this standard, in Table C6-1 and Figure C6-1 to clarify wall component identification.

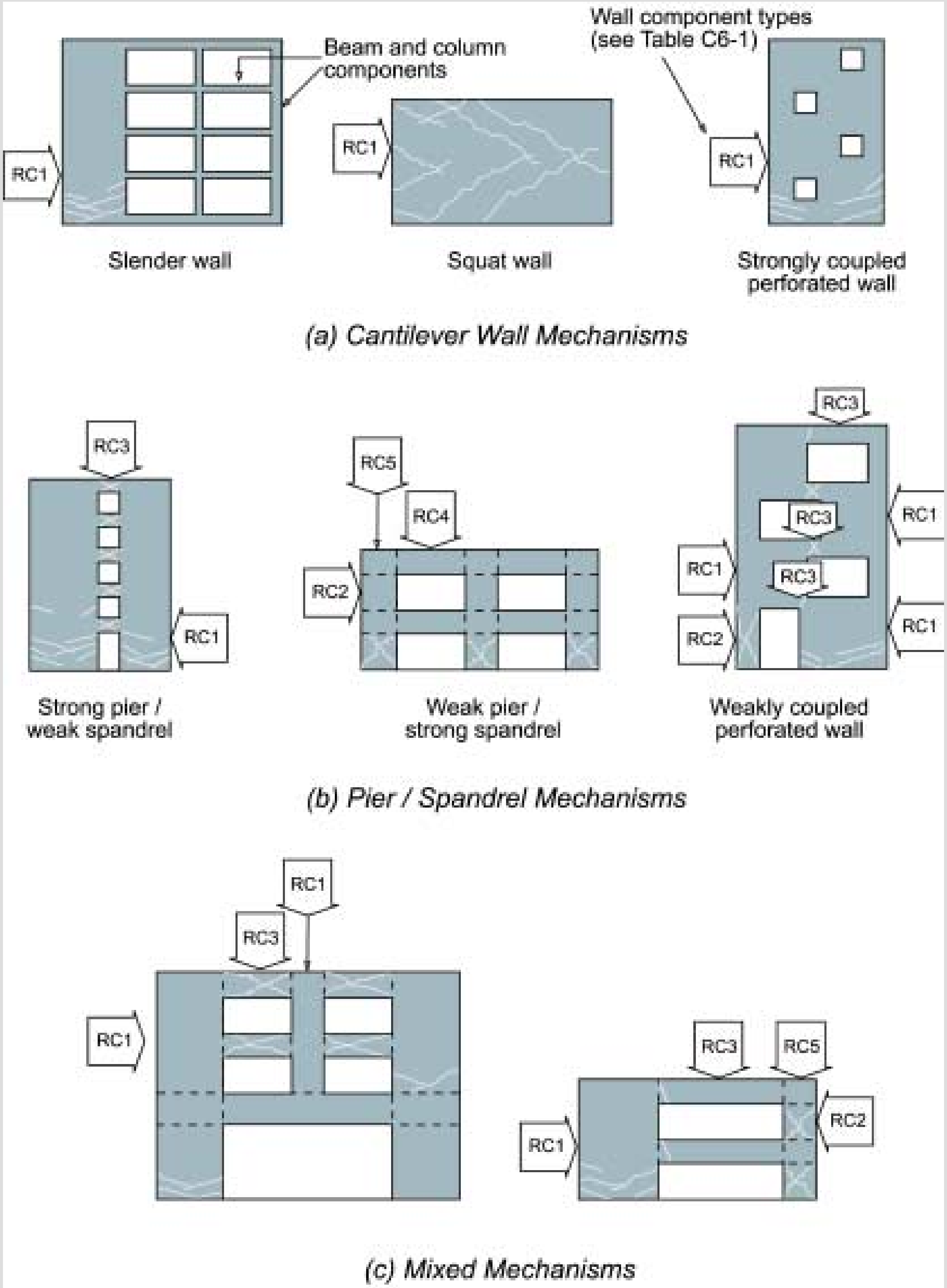

***Figure C6-1 Identification of Component Types in Concrete Shear Wall Elements (from* FEMA 306)**

Table C6-1 Reinforced Concrete Shear Wall Component Types (from FEMA 306)

Component Type per *FEMA 306*		Description	FEMA 356 Designation
RC1	Isolated Wall or Stronger Wall Pier	Stronger than beam or spandrel elements that may frame into it so that nonlinear behavior (and damage) is generally concentrated at the base, with a flexural plastic hinge, shear failure, etc. Includes isolated (cantilever) walls. If the component has a major setback or cutoff of reinforcement above the base, this section should be also checked for nonlinear behavior	Monolithic reinforced concrete wall or vertical wall segment
RC2	Weaker Wall Pier	Weaker than the spandrels to which it connects; characterized by flexural hinging top and bottom, or shear failure, etc.	
RC3	Weaker Spandrel or Coupling Beam	Weaker than the wall piers to which it connects; characterized by hinging at each end, shear failure, sliding shear failure, etc.	Horizontal wall segment or coupling beam
RC4	Stronger Spandrel	Should not suffer damage because it is stronger than attached piers. If this component is damaged, it should probably be re-classified as RC3.	
RC5	Pier-Spandrel Panel Zone	Typically not a critical area in RC walls	Wall segment

6.8.1.1 Monolithic Reinforced Concrete Shear Walls and Wall Segments

Monolithic reinforced concrete shear walls shall consist of vertical cast-in-place elements, either uncoupled or coupled, in open or closed shapes. These walls shall have relatively continuous cross sections and reinforcement and shall provide both vertical and lateral force resistance, in contrast with infilled walls defined in Section 6.7.1.3.

Shear walls or wall segments with axial loads greater than 0.35 P_o shall not be considered effective in resisting seismic forces. The maximum spacing of horizontal and vertical reinforcement shall not exceed 18 inches. Walls with horizontal and vertical reinforcement ratios less than 0.0025, but with reinforcement spacings less than 18 inches, shall be permitted where the shear force demand does not exceed the reduced nominal shear strength of the wall calculated in accordance with Section 6.8.2.3.

C6.8.1.1 Monolithic Reinforced Concrete Shear Walls and Wall Segments

The wall reinforcement is normally continuous in both the horizontal and vertical directions, and bars are typically lap-spliced for tension continuity. The reinforcement mesh may also contain horizontal ties around vertical bars that are concentrated either near the vertical edges of a wall with constant thickness, or in boundary members formed at the wall edges. The amount and spacing of these ties is important for determining how well the concrete at the wall edge is confined, and thus for determining the lateral deformation capacity of the wall.

In general, slender reinforced concrete shear walls will be governed by flexure and will tend to form a plastic flexural hinge near the base of the wall under severe lateral loading. The ductility of the wall will be a function of the percentage of longitudinal reinforcement concentrated near the boundaries of the wall, the level of axial load, the amount of lateral shear required to cause flexural yielding, and the thickness and reinforcement used in the web portion of the shear wall. In general, higher axial load stresses and higher shear stresses will reduce the flexural ductility and energy absorbing capability of the shear wall. Short or squat shear walls will normally be governed by shear. These walls will normally have a limited ability to deform beyond the elastic range and continue to carry lateral loads. Thus, these walls are typically designed either as displacement-controlled components with low ductility capacities or as force-controlled components.

6.8.1.2 Reinforced Concrete Columns Supporting Discontinuous Shear Walls

Reinforced concrete columns supporting discontinuous shear walls shall be evaluated and rehabilitated to comply with the requirements of Section 6.8.2.

C6.8.1.2 Reinforced Concrete Columns Supporting Discontinuous Shear Walls

In shear wall buildings it is not uncommon to find that some walls are terminated either to create commercial space in the first story or to create parking spaces in the basement. In such cases, the walls are commonly supported by columns. Such designs are not recommended in seismic zones because very large demands may be placed on these columns during earthquake loading. In older buildings such columns will often have “standard” longitudinal and transverse reinforcement; the behavior of such columns during past earthquakes indicates that tightly spaced closed ties with well-anchored 135-degree hooks will be required for the building to survive severe earthquake loading.

6.8.1.3 Reinforced Concrete Coupling Beams

Reinforced concrete coupling beams used to link two shear walls together shall be evaluated and rehabilitated to comply with the requirements of Section 6.8.2.

C6.8.1.3 Reinforced Concrete Coupling Beams

The coupled walls are generally much stiffer and stronger than they would be if they acted independently. Coupling beams typically have a small span-to-depth ratio, and their inelastic behavior is normally affected by the high shear forces acting in these components. Coupling beams in most older reinforced concrete buildings will commonly have “conventional” reinforcement that consists of longitudinal flexural steel and transverse steel for shear. In some, more modern buildings, or in buildings where coupled shear walls are used for seismic rehabilitation, the coupling beams may use diagonal reinforcement as the primary reinforcement for both flexure and shear. The inelastic behavior of coupling beams that use diagonal reinforcement has been shown experimentally to be much better with respect to retention of strength, stiffness, and energy dissipation capacity than the observed behavior of coupling beams with conventional reinforcement.

6.8.2 Reinforced Concrete Shear Walls, Wall Segments, Coupling Beams, and RC Columns Supporting Discontinuous Shear Walls

6.8.2.1 General Considerations

The analytical model for a shear wall element shall represent the stiffness, strength, and deformation capacity of the shear wall. Potential failure in flexure, shear, and reinforcement development at any point in the shear wall shall be considered. Interaction with other structural and nonstructural elements shall be included.

Slender shear walls and wall segments shall be permitted to be modeled as equivalent beam-column elements that include both flexural and shear deformations. The flexural strength of beam-column elements shall include the interaction of axial load and bending. The rigid-connection zone at beam connections to this equivalent beam-column element shall represent the distance from the wall centroid to the edge of the wall. Unsymmetrical wall sections shall model the different bending capacities for the two loading directions.

A beam element that incorporates both bending and shear deformations shall be used to model coupling beams. The element inelastic response shall account for the loss of shear strength and stiffness during reversed cyclic loading to large deformations. For coupling beams that have diagonal reinforcement satisfying *ACI 318*, a beam element representing flexure only shall be permitted.

For columns supporting discontinuous shear walls, the model shall account for axial compression, axial tension, flexure, and shear response including rapid loss of resistance where this behavior is likely under design loadings. The diaphragm action of concrete slabs that interconnect shear walls and frame columns shall be represented in the model.

C6.8.2.1 General Considerations

For rectangular shear walls and wall segments with $h_w / l_w \leq 2.5$, and flanged wall sections with $h_w / l_w \leq 3.5$, either a modified beam-column analogy or a multiple-node, multiple-spring approach should be used. Because shear walls usually respond in single curvature over a story height, the use of one multiple-spring element per story should be permitted for modeling shear walls. Wall segments should be modeled with either the beam-column element or with a multiple-spring model with two elements over the length of the wall segment.

Coupling beams that have diagonal reinforcement satisfying *FEMA 222A* will commonly have a stable hysteretic response under large load reversals. Therefore, these members could adequately be modeled with beam elements used for typical frame analyses.

6.8.2.2 Stiffness

The effective stiffness of all the elements discussed in Section 6.8 shall be defined based on the material properties, component dimensions, reinforcement quantities, boundary conditions, and current state of the member with respect to cracking and stress levels. Alternatively, use of values for effective stiffness given in Table 6-5 shall be permitted. To obtain a proper distribution of lateral forces in bearing wall buildings, all of the walls shall be assumed to be either cracked or uncracked. In buildings where lateral load resistance is provided by either structural walls only, or a combination of walls and frame members, all shear walls and wall segments discussed in this section shall be considered to be cracked.

For coupling beams, the effective stiffness values given in Table 6-5 for nonprestressed beams shall be used unless alternative stiffnesses are determined by more detailed analysis. The effective stiffness of columns supporting discontinuous shear walls shall change between the values given for columns in tension and compression, depending on the direction of the lateral load being resisted by the shear wall.

6.8.2.2.1 Linear Static and Dynamic Procedures

Shear walls and associated components shall be modeled considering axial, flexural, and shear stiffness. For closed and open wall shapes, such as box, T, L, I, and C sections, the effective tension or compression flange widths shall be as specified in Section 6.4.1.3. The calculated stiffnesses to be used in analysis shall be in accordance with the requirements of Section 6.4.1.2.

Joints between shear walls and frame elements shall be modeled as stiff components or rigid components, as appropriate.

6.8.2.2.2 Nonlinear Static Procedure

Nonlinear load-deformation relations for use in analysis by nonlinear static and dynamic procedures shall comply with the requirements of Section 6.4.1.2.

Monotonic load-deformation relationships for analytical models that represent shear walls, wall elements, coupling beams, and columns that support discontinuous shear walls shall be in accordance with the generalized relation shown in Figure 6-1.

For shear walls and wall segments having inelastic behavior under lateral loading that is governed by flexure, as well as columns supporting discontinuous shear walls, the following approach shall be permitted. The load-deformation relationship in Figure 6-1 shall be used with the x-axis of Figure 6-1 taken as the rotation over the plastic hinging region at the end of the member shown in Figure 6-2. The hinge rotation at point *B* in Figure 6-1 corresponds to the yield point, θ_y, and shall be calculated in accordance with Equation (6-6):

$$\theta_y = \left(\frac{M_y}{E_c I}\right) l_p \qquad (6\text{-}6)$$

where:

M_y = Yield moment capacity of the shear wall or wall segment
E_c = Concrete modulus
I = Member moment of inertia
l_p = Assumed plastic hinge length

For analytical models of shear walls and wall segments, the value of l_p shall be set equal to 0.5 times the flexural depth of the element, but less than one story height for shear walls and less than 50% of the element length for wall segments. For columns supporting discontinuous shear walls, l_p shall be set equal to 0.5 times the flexural depth of the component.

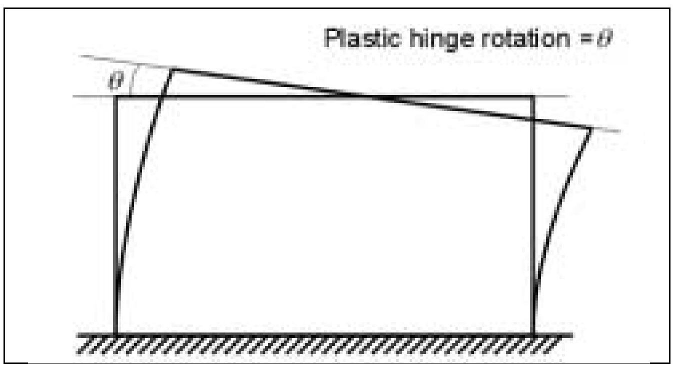

Figure 6-2 ***Plastic Hinge Rotation in Shear Wall where Flexure Dominates Inelastic Response***

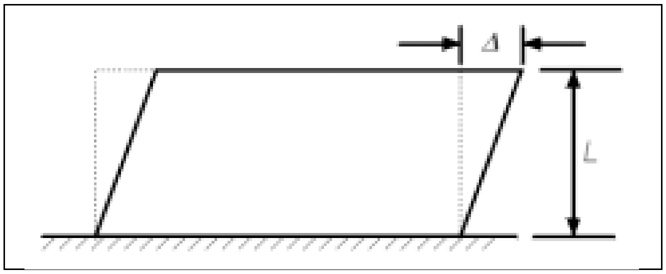

Figure 6-3 ***Story Drift in Shear Wall where Shear Dominates Inelastic Response***

Values for the variables *a*, *b*, and *c* required to define the location of points *C*, *D*, and *E* in Figure 6-1(a), shall be as specified in Table 6-18.

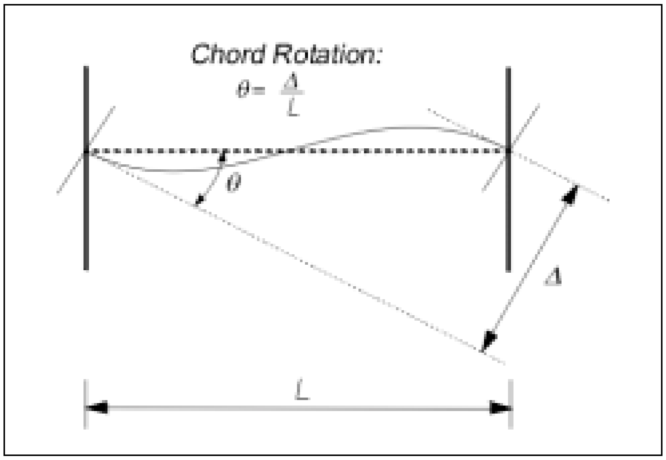

Figure 6-4 ***Chord Rotation for Shear Wall Coupling Beams***

For shear walls and wall segments whose inelastic response is controlled by shear, the following approach shall be permitted. The load-deformation relationship in Figure 6-1(b) shall be used, with the x-axis of Figure 6-1(b) taken as lateral drift. For shear walls, this drift shall be the story drift as shown in Figure 6-3. For wall segments, Figure 6-3 shall represent the member drift.

For coupling beams, the following approach shall be permitted. The load-deformation relationship in Figure 6-1(b) shall be used, with the x-axis of Figure 6-1(b) taken as the chord rotation as defined in Figure 6-4.

Values for the variables *d*, *e*, and *c* required to find the points *C*, *D*, and *E* in Figure 6-1(b), shall be as specified in Table 6-19 for the appropriate members. Linear interpolation between tabulated values shall be used if the member under analysis has conditions that are between the limits given in the tables.

6.8.2.2.3 Nonlinear Dynamic Procedure

For the NDP, the complete hysteretic behavior of each component shall be modeled using properties verified by experimental evidence. Use of the generalized load-deformation relation shown in Figure 6-1 to represent the envelope relation for the analysis shall be permitted. The unloading and reloading stiffnesses and strengths, and any pinching of the load-versus-rotation hysteresis loops, shall reflect the behavior experimentally observed for wall elements similar to the one under investigation.

6.8.2.3 Strength

Component strengths shall be computed according to the general requirements of Sections 6.4.2, with the additional requirements of this section. Strength shall be determined considering the potential for failure in flexure, shear, or development under combined gravity and lateral load.

Nominal flexural strength of shear walls or wall segments, M_n, shall be determined using the fundamental principles given in Chapter 10 of *ACI 318*. For calculation of nominal flexural strength, the effective compression and tension flange widths defined in Section 6.8.2.2 shall be used, except that the first limit shall be changed to one-tenth of the wall height. When determining the flexural yield strength of a shear wall, as represented by point *B* in Figure 6-1(a), only the longitudinal steel in the boundary of the wall shall be included. If the wall does not have a boundary member, then only the longitudinal steel in the outer 25% of the wall section shall be included in the calculation of the yield strength. When calculating the nominal flexural strength of the wall, as represented by point *C* in Figure 6-1(a), all longitudinal steel (including web reinforcement) shall be included in the calculation. For all moment strength calculations, the strength of the longitudinal reinforcement shall be taken as the expected yield strength to account for material overstrength and strain hardening, and the axial load acting on the wall shall include gravity loads as defined in Chapter 3.

The nominal shear strength of a shear wall or wall segment, V_n, shall be determined based on the principles and equations given in Chapter 21 of *ACI 318*. The nominal shear strength of columns supporting discontinuous shear walls shall be determined based on the principles and equations given in Chapter 21 of *ACI 318*. For all shear strength calculations, 1.0 times the specified reinforcement yield strength shall be used. There shall be no difference between the yield and nominal shear strengths, as represented by points *B* and *C* in Figure 6-1.

When a shear wall or wall segment has a transverse reinforcement percentage, ρ_n, less than the minimum value of 0.0025 but greater than 0.0015, the shear strength of the wall shall be analyzed using the *ACI 318* equations noted above. For transverse reinforcement percentages less than 0.0015, the contribution from the wall reinforcement to the shear strength of the wall shall be held constant at the value obtained using $\rho_n = 0.0015$.

Splice lengths for primary longitudinal reinforcement shall be evaluated using the procedures given in Section 6.4.5. Reduced flexural strengths shall be evaluated at locations where splices govern the usable stress in the reinforcement. The need for confinement reinforcement in shear wall boundary members shall be evaluated by the procedure in *ACI 318* or other approved procedure.

The nominal flexural and shear strengths of coupling beams shall be evaluated using the principles and equations contained in Chapter 21 of *ACI 318*. The expected strength of longitudinal or diagonal reinforcement shall be used.

The nominal shear and flexural strengths of columns supporting discontinuous shear walls shall be evaluated as defined in Section 6.5.2.3.

C6.8.2.3 Strength

Data presented by Wood (1990) indicate that wall strength is insensitive to the quantity of transverse reinforcement when it drops below a steel ratio of 0.0015.

The need for confinement reinforcement in shear wall boundary members may be evaluated by the method recommended by Wallace (1994 and 1995) for determining maximum lateral deformations in the wall and the resulting maximum compression strains in the wall boundary.

6.8.2.4 Acceptance Criteria

6.8.2.4.1 Linear Static and Dynamic Procedures

Shear walls, wall segments, coupling beams, and columns supporting discontinuous shear walls shall be classified as either deformation- or force-controlled, as defined in Section 2.4.4. For columns supporting discontinuous shear walls, deformation-controlled actions shall be restricted to flexure. In other components, deformation-controlled actions shall be restricted to flexure or shear. All other actions shall be defined as being force-controlled actions.

The nominal flexural strength of a shear wall or wall segment shall be used to determine the maximum shear force in shear walls, wall segments, and columns supporting discontinuous shear walls. For cantilever shear walls and columns supporting discontinuous shear walls, the design shear force shall be equal to the magnitude of the lateral force required to develop the nominal flexural strength at the base of the wall, assuming the lateral force is distributed uniformly over the height of the wall. For wall segments, the design force shall be equal to the shear corresponding to the development of the positive and negative nominal moment strengths at opposite ends of the wall segment.

Design actions (flexure, shear, or force transfer at rebar anchorages and splices) on components shall be determined as prescribed in Chapter 3. When determining the appropriate value for the design actions, proper consideration shall be given to gravity loads and to the maximum forces that can be transmitted considering nonlinear action in adjacent components. Design actions shall be compared with design strengths in accordance with Section 3.4.2.2. Table 6-20 and 6-21 specify *m* values for use in Equation (3-20). Alternate *m* values shall be permitted where justified by experimental evidence and analysis.

6.8.2.4.2 Nonlinear Static and Dynamic Procedures

In the design model, inelastic response shall be restricted to those elements and actions listed in Tables 6-18 and 6-19, except where it is demonstrated that other inelastic actions are justified for the selected performance levels. For members experiencing inelastic behavior, the magnitude of other actions (forces, moments, or torque) in the member shall correspond to the magnitude of the action causing inelastic behavior. The magnitude of these other actions shall be shown to be below their nominal capacities.

Components experiencing inelastic response shall satisfy the requirements of Section 3.4.3.2, and the maximum plastic hinge rotations, drifts, or chord rotation angles shall not exceed the values given in Tables 6-18 and 6-19, for the selected performance level. Linear interpolation between tabulated values shall be used if the member under analysis has conditions that are between the limits given in the tables.

Table 6-18 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— Members Controlled by Flexure

Conditions			Plastic Hinge Rotation (radians)		Residual Strength Ratio	Acceptable Plastic Hinge Rotation (radians)				
						Performance Level				
							Component Type			
							Primary		Secondary[4]	
			a	**b**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
i. Shear walls and wall segments										
$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c}$	$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$	Confined Boundary[1]								
≤ 0.1	≤3	Yes	0.015	0.020	0.75	0.005	0.010	0.015	0.015	0.020
≤ 0.1	≥ 6	Yes	0.010	0.015	0.40	0.004	0.008	0.010	0.010	0.015
≥ 0.25	≤ 3	Yes	0.009	0.012	0.60	0.003	0.006	0.009	0.009	0.012
≥ 0.25	≥ 6	Yes	0.005	0.010	0.30	0.0015	0.003	0.005	0.005	0.010
≤ 0.1	≤ 3	No	0.008	0.015	0.60	0.002	0.004	0.008	0.008	0.015
≤ 0.1	≥ 6	No	0.006	0.010	0.30	0.002	0.004	0.006	0.006	0.010
≥ 0.25	≤ 3	No	0.003	0.005	0.25	0.001	0.002	0.003	0.003	0.005
≥ 0.25	≥ 6	No	0.002	0.004	0.20	0.001	0.001	0.002	0.002	0.004
ii. Columns supporting discontinuous shear walls										
Transverse reinforcement[2]										
Conforming			0.010	0.015	0.20	0.003	0.007	0.010	n.a.	n.a.
Nonconforming			0.0	0.0	0.0	0.0	0.0	0.0	n.a.	n.a.
iii. Shear wall coupling beams										
Longitudinal reinforcement and transverse reinforcement[3]		$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$								
Conventional longitudinal reinforcement with conforming transverse reinforcement		≤ 3	0.025	0.050	0.75	0.010	0.02	0.025	0.025	0.050
		≥ 6	0.02	0.040	0.50	0.005	0.010	0.020	0.020	0.040
Conventional longitudinal reinforcement with nonconforming transverse reinforcement		≤ 3	0.020	0.035	0.50	0.006	0.012	0.020	0.020	0.035
		≥ 6	0.010	0.025	0.25	0.005	0.008	0.010	0.010	0.025
Diagonal reinforcement		n.a.	0.030	0.050	0.80	0.006	0.018	0.030	0.030	0.050

1. Requirements for a confined boundary are the same as those given in *ACI 318*.
2. Requirements for conforming transverse reinforcement in columns are: (a) hoops over the entire length of the column at a spacing ≤ $d/2$, and (b) strength of hoops $V_s \geq$ required shear strength of column.
3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the coupling beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of the coupling beam.
4. For secondary coupling beams spanning <8'-0", with bottom reinforcement continuous into the supporting walls, secondary values shall be permitted to be doubled.

Table 6-19 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Members Controlled by Shear

Conditions		Total Drift Ratio (%), or Chord Rotation (radians)[1] *d*	Total Drift Ratio (%), or Chord Rotation (radians)[1] *e*	Residual Strength Ratio *c*	Acceptable Total Drift (%) or Chord Rotation (radians)[1] Performance Level IO	Primary LS	Primary CP	Secondary LS	Secondary CP
i. Shear walls and wall segments									
All shear walls and wall segments[2]		0.75	2.0	0.40	0.40	0.60	0.75	0.75	1.5
ii. Shear wall coupling beams[4]									
Longitudinal reinforcement and transverse reinforcement[3]	$\frac{\text{Shear}}{t_w l_w \sqrt{f_c'}}$								
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	0.002	0.030	0.60	0.006	0.015	0.020	0.020	0.030
	≥ 6	0.016	0.024	0.30	0.005	0.012	0.016	0.016	0.024
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤ 3	0.012	0.025	0.40	0.006	0.008	0.010	0.010	0.020
	≥ 6	0.008	0.014	0.20	0.004	0.006	0.007	0.007	0.012

1. For shear walls and wall segments, use drift; for coupling beams, use chord rotation; refer to Figures 6-3 and 6-4.
2. For shear walls and wall segments where inelastic behavior is governed by shear, the axial load on the member must be ≤ 0.15 $A_g f_c'$; otherwise, the member must be treated as a force-controlled component.
3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the coupling beam at a spacing ≤ *d*/3, and (b) strength of closed stirrups V_s ≥ 3/4 of required shear strength of the coupling beam.
4. For secondary coupling beams spanning <8'-0", with bottom reinforcement continuous into the supporting walls, secondary values shall be permitted to be doubled.

Table 6-20 Numerical Acceptance Criteria for Linear Procedures—Members Controlled by Flexure

			***m*-factors**				
			Performance Level				
				Component Type			
				Primary		**Secondary**	
Conditions			**IO**	**LS**	**CP**	**LS**	**CP**
i. Shear walls and wall segments							
$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c}$	$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$ [5]	Confined Boundary[1]					
≤ 0.1	≤ 3	Yes	2	4	6	6	8
≤ 0.1	≥ 6	Yes	2	3	4	4	6
≥ 0.25	≤ 3	Yes	1.5	3	4	4	6
≥ 0.25	≥ 6	Yes	1.25	2	2.5	2.5	4
≤ 0.1	≤ 3	No	2	2.5	4	4	6
≤ 0.1	≥ 6	No	1.5	2	2.5	2.5	4
≥ 0.25	≤ 3	No	1.25	1.5	2	2	3
≥ 0.25	≥ 6	No	1.25	1.5	1.75	1.75	2
ii. Columns supporting discontinuous shear walls							
Transverse reinforcement[2]							
Conforming			1	1.5	2	n.a.	n.a.
Nonconforming			1	1	1	n.a.	n.a.
iii. Shear wall coupling beams[4]							
Longitudinal reinforcement and transverse reinforcement[3]		$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$ [5]					
Conventional longitudinal reinforcement with conforming transverse reinforcement		≤ 3	2	4	6	6	9
		≥ 6	1.5	3	4	4	7
Conventional longitudinal reinforcement with nonconforming transverse reinforcement		≤ 3	1.5	3.5	5	5	8
		≥ 6	1.2	1.8	2.5	2.5	4
Diagonal reinforcement		n.a.	2	5	7	7	10

1. Requirements for a confined boundary are the same as those given in *ACI 318*.
2. Requirements for conforming transverse reinforcement in columns are: (a) hoops over the entire length of the column at a spacing ≤ $d/2$, and (b) strength of hoops V_s ≥ required shear strength of column.
3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the coupling beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups V_s ≥ 3/4 of required shear strength of the coupling beam.
4. For secondary coupling beams spanning <8'-0", with bottom reinforcement continuous into the supporting walls, secondary values shall be permitted to be doubled.
5. Design shear shall be calculated using limit-state analysis procedures.

Table 6-21 ***Numerical Acceptance Criteria for Linear Procedures—Members Controlled by Shear***

Conditions		*m*-factors				
		Performance Level				
			Component Type			
			Primary		Secondary	
		IO	LS	CP	LS	CP
i. Shear walls and wall segments						
All shear walls and wall segments[1]		2	2	3	2	3
ii. Shear wall coupling beams[3]						
Longitudinal reinforcement and transverse reinforcement[2]	$\dfrac{\text{Shear}}{t_w l_w \sqrt{f_c'}}$					
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	1.5	3	4	4	6
	≥ 6	1.2	2	2.5	2.5	3.5
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤ 3	1.5	2.5	3	3	4
	≥ 6	1.25	1.2	1.5	1.5	2.5

1. For shear walls and wall segments where inelastic behavior is governed by shear, the axial load on the member must be ≤ 0.15 $A_g f'_c$, the longitudinal reinforcement must be symmetrical, and the maximum shear demand must be ≤ 6 $\sqrt{f_c'}$; otherwise, the shear shall be considered to be a force-controlled action.
2. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the coupling beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups V_s ≥ 3/4 of required shear strength of the coupling beam.
3. For secondary coupling beams spanning <8’-0”, with bottom reinforcement continuous into the supporting walls, secondary values shall be permitted to be doubled.

6.8.2.5 Rehabilitation Measures

Reinforced shear walls, wall segments, coupling beams, and columns supporting discontinuous shear walls that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 6.4.7 and other provisions of this standard.

C6.8.2.5 Rehabilitation Measures

The following measures may be effective in rehabilitating reinforced shear walls, wall segments, coupling beams, and reinforced concrete columns supporting discontinuous shear walls:

1. **Addition of wall boundary members.** Addition of boundary members may be an effective measure in strengthening shear walls or wall segments that have insufficient flexural strength. These members may be either cast-in-place reinforced concrete elements or steel sections. In both cases, proper connections should be made between the existing wall and the added members. The shear capacity of the rehabilitated wall should be re-evaluated.
2. **Addition of confinement jackets at wall boundaries.** Increasing the confinement at the wall boundaries by the addition of a steel or reinforced concrete jacket may be an effective measure in improving the flexural deformation capacity of a shear wall. For both types of jackets, the longitudinal should not be continuous from story to story unless the jacket is also being used to increase the flexural capacity. The minimum thickness for a concrete jacket should be three inches. Carbon fiber wrap should be permitted for improving the confinement of concrete in compression.
3. **Reduction of flexural strength.** Reduction in the flexural capacity of a shear wall to change the governing failure mode from shear to flexure may be an effective rehabilitation measure. It may be accomplished by saw-cutting a specified number of longitudinal bars near the edges of the shear wall.
4. **Increased shear strength of wall.** Increasing the shear strength of the web of a shear wall by casting additional reinforced concrete adjacent to the wall web may be an effective rehabilitation measure. The new concrete should be at least four inches thick and should contain horizontal and vertical reinforcement. The new concrete should be properly bonded to the existing web of the shear wall. The use of carbon fiber sheets, epoxied to the concrete surface, should also be permitted to increase the shear capacity of a shear wall.
5. **Confinement jackets to improve deformation capacity of coupling beams and columns supporting discontinuous shear walls.** The use of confinement jackets specified above as a rehabilitation measure for wall boundaries, and in Section 6.5 for frame elements, may also be effective in increasing both the shear capacity and the deformation capacity of coupling beams and columns supporting discontinuous shear walls.
6. **Infilling between columns supporting discontinuous shear walls.** Where a discontinuous shear wall is supported on columns that lack either sufficient strength or deformation capacity to satisfy design criteria, making the wall continuous by infilling the opening between these columns may be an effective rehabilitation measure. The infill and existing columns should be designed to satisfy all the requirements for new wall construction, including any strengthening of the existing columns required by adding a concrete or steel jacket for strength and increased confinement. The opening below a discontinuous shear wall should also be permitted to be “infilled” with steel bracing. The bracing members should be sized to satisfy all design requirements and the columns should be strengthened with a steel or a reinforced concrete jacket.

All of the above rehabilitation measures require an evaluation of the wall foundation, diaphragms, and connections between existing structural elements and any elements added for rehabilitation purposes.

6.9 Precast Concrete Shear Walls

6.9.1 Types of Precast Shear Walls

Precast concrete shear walls shall consist of story-high or half-story-high precast wall segments that are made continuous through the use of either mechanical connectors or reinforcement splicing techniques with or without a cast-in-place connection strip. Connections between precast segments shall be permitted along both the horizontal and vertical edges of a wall segment.

The design of the following types of precast shear walls shall meet the requirements of Section 6.9:

1. Construction that emulates cast-in-place shear wall defined as that construction in which the reinforcement connections are made to be stronger than the adjacent precast panels so that the lateral load response of the precast wall system will be comparable to that for monolithic shear walls.

2. Jointed construction defined as construction in which inelastic action is permitted to occur at the connections between precast panels.

3. Tilt-up construction defined as a special technique for precast wall construction where there are vertical joints between adjacent panels and horizontal joints at the foundation level, and where the roof or floor diaphragm connects with the tilt-up panel.

6.9.1.1 Construction that Emulates Cast-In-Place Shear Wall

For this type of precast wall, the connections between precast wall elements shall be designed and detailed to be stronger than the panels they connect. The shear walls and wall segments of precast walls that emulate cast-in-place construction shall be evaluated by the criteria defined in Section 6.8.

C6.9.1.1 Construction that Emulates Cast-In-Place Shear Wall

When the precast shear wall is subjected to lateral loading, any yielding and inelastic behavior should take place in the panel elements away from the connections. If the reinforcement detailing in the panel is similar to that for cast-in-place shear walls, then the inelastic response of a precast shear wall should be very similar to that for a cast-in-place wall.

Modern building codes permit the use of precast shear wall construction in high seismic zones if it satisfies the criteria for construction that emulates a cast-in-place shear wall.

6.9.1.2 Jointed Construction

Shear walls and wall segments of jointed construction type of precast walls shall be evaluated by the criteria defined in Section 6.9.2.

C6.9.1.2 Jointed Construction

For most older structures that contain precast shear walls, and for some modern construction, inelastic activity can be expected in the connections between precast wall panels during severe lateral loading. Because joints between precast shear walls in older buildings have often exhibited brittle behavior during inelastic load reversals, jointed construction had not been permitted in high seismic zones. Therefore, when evaluating older buildings that contain precast shear walls that are likely to respond as jointed construction, the permissible ductilities and rotation capacities given in Section 6.8 should be reduced.

For some modern structures, precast shear walls have been constructed with special connectors that are detailed to exhibit ductile response and energy absorption characteristics. Many of these connectors are proprietary and only limited experimental evidence concerning their inelastic behavior is available. Although this type of construction is clearly safer than jointed construction in older buildings, the experimental evidence is not sufficient to permit the use of the same ductility and rotation capacities given for cast-in-place construction. Thus, the permissible values given in Section 6.8 should be reduced.

6.9.1.3 Tilt-up Construction

Shear walls and wall segments of tilt-up type of precast walls shall be evaluated by the criteria defined in Section 6.9.2.

C6.9.1.3 Tilt-up Construction

Tilt-up construction should be considered to be a special case of jointed construction. The walls for most buildings constructed by the tilt-up method are longer than their height. Shear would usually govern their in-plane design. The major concern for most tilt-up construction is the connection between the tilt-up wall and the roof diaphragm. That connection should be carefully analyzed to be sure the diaphragm forces can be safely transmitted to the precast wall system.

6.9.2 Precast Concrete Shear Walls and Wall Segments

6.9.2.1 General Considerations

The analytical model for a precast concrete shear wall or wall segment shall represent the stiffness, strength, and deformation capacity of the overall member, as well as the connections and joints between any precast panel components that compose the wall. Potential failure in flexure, shear, and reinforcement development at any point in the shear wall panels or connections shall be considered. Interaction with other structural and nonstructural elements shall be included.

Modeling of precast concrete shear walls and wall segments within the precast panels as equivalent beam-columns that include both flexural and shear deformations shall be permitted. The rigid-connection zone at beam connections to these equivalent beam-columns shall represent the distance from the wall centroid to the edge of the wall or wall segment. The different bending capacities for the two loading directions of unsymmetrical precast wall sections shall be modeled.

For precast shear walls and wall segments where shear deformations have a more significant effect on behavior than flextural deformation, a multiple spring model shall be used.

The diaphragm action of concrete slabs interconnecting precast shear walls and frame columns shall be represented in the model.

6.9.2.2 Stiffness

The modeling assumptions defined in Section 6.8.2.2 for monolithic concrete shear walls and wall segments shall also be used for precast concrete walls. In addition, the analytical model shall model the axial, shear, and rotational deformations of the connections between the precast components that compose the wall by either softening the model used to represent the precast panels or by adding spring elements between panels.

6.9.2.2.1 Linear Static and Dynamic Procedures

The modeling procedures given in Section 6.8.2.2.1, combined with a procedure for including connection deformations as noted above, shall be used.

6.9.2.2.2 Nonlinear Static Procedure

Nonlinear load-deformation relations shall comply with the requirements of Section 6.4.1.2. The monotonic load-deformation relationships for analytical models that represent precast shear walls and wall elements within precast panels shall be in accordance with the generalized relation shown in Figure 6-1, except that alternative approaches shall be permitted where verified by experiments. Where the relations are according to the Figure 6-1, the following approach shall be permitted.

Values for plastic hinge rotations or drifts at points *B*, *C*, and *E* for the two general shapes shall be as defined below. The strength levels at points *B* and *C* shall correspond to the yield strength and nominal strength, as defined in Section 6.8.2.3. The residual strength for the line segment *D*–*E* shall be as defined below.

For precast shear walls and wall segments whose inelastic behavior under lateral loading is governed by flexure, the general load-deformation relationship shall be defined as in Figure 6-1(a). For these members, the x-axis of Figure 6-1(a) shall be taken as the rotation over the plastic hinging region at the end of the member as shown in Figure 6-2. If the requirements for construction that emulates cast-in- place shear wall are satisfied, the value of the hinge rotation at point *B* shall correspond to the yield rotation, θ_y, and shall be calculated by Equation (6-6). The same expression shall also be used for wall segments within a precast panel if flexure controls the inelastic response of the segment. If the precast wall is of jointed construction and flexure governs the inelastic response of the member, then the value of θ_y shall be increased to account for rotation in the joints between panels or between the panel and the foundation.

For precast shear walls and wall segments whose inelastic behavior under lateral loading is governed by shear, the general load-deformation relationship shall be defined as in Figure 6-1(b). For these members, the x-axis of Figure 6-1(b) shall be taken as the story drift for shear walls, and as the element drift for wall segments as shown in Figure 6-3.

For construction that emulates cast-in-place shear walls, the values for the variables *a*, *b*, and *c* required to define the location of points *C*, *D*, and *E* in Figure 6-1(a), shall be as specified in Table 6-18. For construction classified as jointed construction, the values of *a*, *b*, and *c* specified in Table 6-18 shall be reduced to 50% of the given values, unless experimental evidence available to justify higher values is approved. In no case, however, shall values larger than those specified in Table 6-18 be used.

For construction that emulates cast-in-place shear walls, values for the variables *d*, *e*, and *c* required to find the points *C*, *D*, and *E* in Figure 6-1(b), shall be as specified in Table 6-19 for the appropriate member conditions. For construction classified as jointed construction, the values of *d*, *e*, and *c* specified in Table 6-19 shall be reduced to 50% of the specified values unless experimental evidence available to justify higher values is approved. In no case, however, shall values larger than those specified in Table 6-19 be used.

For Tables 6-18 and 6-19, linear interpolation between tabulated values shall be permitted if the member under analysis has conditions that are between the limits given in the tables.

6.9.2.2.3 Nonlinear Dynamic Procedure

Nonlinear load-deformation relations for use in analysis by Nonlinear Dynamic Procedure (NDP) shall model the complete hysteretic behavior of each component using properties verified by experimental evidence. The generalized relation shown in Figure 6-1 shall be taken to represent the envelope for the analysis. The unloading and reloading stiffnesses and strengths, and any pinching of the load versus rotation hysteresis loops, shall reflect the behavior experimentally observed for wall elements similar to the one under investigation.

6.9.2.3 Strength

The strength of precast concrete shear walls and wall segments within the panels shall be computed according to the general requirement of Sections 6.4.2, except as modified here. For construction emulating cast-in-place shear wall, the strength calculation procedures given in Section 6.8.2.3 shall be followed.

For jointed construction, calculations of axial, shear, and flexural strength of the connections between panels shall be based on fundamental principles of structural mechanics. Expected yield strength for steel reinforcement of connection hardware used in the connections shall be used when calculating the axial and flexural strength of the connection region. The unmodified specified yield strength of the reinforcement and connection hardware shall be used when calculating the shear strength of the connection region.

For all precast concrete shear walls of jointed construction, no difference shall be taken between the computed yield and nominal strengths in flexure and shear. The values for strength represented by the points *B* and *C* in Figure 6-1 shall be computed following the procedures given in Section 6.8.2.3.

C6.9.2.3 Strength

In older construction, particular attention must be given to the technique used for splicing reinforcement extending from adjacent panels into the connection. These connections may be insufficient and can often govern the strength of the precast shear wall system.

6.9.2.4 Acceptance Criteria

6.9.2.4.1 Linear Static and Dynamic Procedures

For precast shear wall construction that emulates cast-in-place construction and for wall segments within a precast panel, the acceptance criteria defined in Section 6.8.2.4.1 shall be followed. For precast shear wall construction defined as jointed construction, the acceptance criteria procedure given in Section 6.8.2.4.1 shall be followed; however, the *m* values specified in Tables 6-20 and 6-21 shall be reduced by 50%, unless experimental evidence justifies the use of a larger value. In no case shall an *m* value be taken as less than 1.0.

6.9.2.4.2 Nonlinear Static and Dynamic Procedures

Inelastic response shall be restricted to those shear walls (and wall segments) and actions listed in Tables 6-18 and 6-19, except where it is demonstrated by experimental evidence and analysis that other inelastic action is acceptable for the selected performance levels. For components experiencing inelastic behavior, the magnitude of the other actions (forces, moments, or torques) in the component shall correspond to the magnitude of the action causing the inelastic behavior. The magnitude of these other actions shall be shown to be below their nominal capacities.

For precast shear walls that emulate cast-in-place construction and wall segments within a precast panel, the maximum plastic hinge rotation angles or drifts during inelastic response shall not exceed the values specified in Tables 6-18 and 6-19. For precast shear walls of jointed construction, the maximum plastic hinge rotation angles or drifts during inelastic response shall not exceed one-half of the values specified in Tables 6-18 and 6-19 unless experimental evidence justifies a higher value. However, in no case shall deformation values larger than those specified in these tables be used for jointed type construction.

If the maximum deformation value exceeds the corresponding tabular value, the element shall be considered to be deficient, and either the element or structure shall be rehabilitated.

Alternative approaches or values shall be permitted where justified by experimental evidence and analysis.

6.9.2.5 Rehabilitation Measures

Precast concrete shear walls or wall segments that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 6.4.7 and other provisions of this standard.

C6.9.2.5 Rehabilitation Measures

Precast concrete shear wall systems may suffer from some of the same deficiencies as cast-in-place walls. These may include inadequate flexural capacity, inadequate shear capacity with respect to flexural capacity, lack of confinement at wall boundaries, and inadequate splice lengths for longitudinal reinforcement in wall boundaries. A few deficiencies unique to precast wall construction are inadequate connections between panels, to the foundation, and to floor or roof diaphragms.

The rehabilitation measures described in Section 6.8.2.5 may also be effective in rehabilitating precast concrete shear walls. In addition, the following rehabilitation measures may be effective:

1. **Enhancement of connections between adjacent or intersecting precast wall panels.** Mechanical connectors such as steel shapes and various types of drilled-in-anchors, or cast-in-place strengthening methods, or a combination of the two, may be effective in strengthening connections between precast panels. Cast-in-place strengthening methods may include exposing the reinforcing steel at the edges of adjacent panels, adding vertical and transverse (tie) reinforcement, and placing new concrete.

2. **Enhancement of connections between precast wall panels and foundations.** Increasing the shear capacity of the wall panel-to-foundation connection by using supplemental mechanical connectors or by using a cast-in-place overlay with new dowels into the foundation may be an effective rehabilitation measure. Increasing the overturning moment capacity of the panel-to-foundation connection by using drilled-in dowels within a new cast-in-place connection at the edges of the panel may also be an effective rehabilitation measure. Adding connections to adjacent panels may also be an effective rehabilitation measure in eliminating some of the forces transmitted through the panel-to-foundation connection.

3. **Enhancement of connections between precast wall panels and floor or roof diaphragms.** Strengthening these connections by using either supplemental mechanical devices or cast-in-place connectors may be an effective rehabilitation measure. Both in-plane shear and out-of-plane forces should be considered when strengthening these connections.

6.10 Concrete-Braced Frames

6.10.1 Types of Concrete-Braced Frames

Reinforced concrete-braced frames shall be defined as those frames with monolithic, non-prestressed, reinforced concrete beams, columns, and diagonal braces that are coincident at beam-column joints and that resist lateral loads primarily through truss action.

Where masonry infills are present in concrete-braced frames, requirements for masonry infilled frames as specified in Section 6.7 shall also apply.

The provisions of Section 6.10 shall apply to existing reinforced concrete-braced frames and existing reinforced concrete-braced frames rehabilitated by addition or removal of material.

6.10.2 General Considerations

The analytical model for a reinforced concrete-braced frame shall represent the strength, stiffness, and deformation capacity of beams, columns, braces, and all connections and components of the element. Potential failure in tension, compression (including instability), flexure, shear, anchorage, and reinforcement development at any section along the component length shall be considered. Interaction with other structural and nonstructural elements and components shall be included.

The analytical model that represents the framing, using line elements with properties concentrated at component centerlines, shall be permitted. The analytical model also shall comply with the requirements specified in Section 6.5.2.1.

In frames having braces in some bays and no braces in other bays, the restraint of the brace shall be represented in the analytical model as specified above, and the nonbraced bays shall be modeled as frames in compliance with the applicable provisions in other sections of this chapter. Where braces create a vertically discontinuous frame, the effects of the discontinuity on overall building performance shall be considered.

Inelastic deformations in primary components shall be restricted to flexure and axial load in beams, columns, and braces. Other inelastic deformations shall be permitted in secondary components. Acceptance criteria for design actions shall be as specified in Section 6.10.5.

6.10.3 Stiffness

6.10.3.1 Linear Static and Dynamic Procedures

Modeling of beams, columns, and braces in braced portions of the frame considering only axial tension and compression flexibilities shall be permitted. Nonbraced portions of frames shall be modeled according to procedures described elsewhere for frames. Effective stiffnesses shall be according to Section 6.4.1.2.

6.10.3.2 Nonlinear Static Procedure

Nonlinear load-deformation relations shall comply with the requirements of Section 6.4.1.2.

Beams, columns, and braces in braced portions shall be modeled using nonlinear truss components or other models whose behavior has been demonstrated to adequately represent behavior of concrete components dominated by axial tension and compression loading. Models for beams and columns in nonbraced portions shall comply with requirements for frames specified in Section 6.5.2.2.2. The model shall be capable of representing inelastic response along the component lengths, as well as within connections.

Monotonic load-deformation relations shall be according to the generalized load-deformation relation shown in Figure 6-1, except that different relations are permitted where verified by experiments. The overall load-deformation relation shall be established so that the maximum resistance is consistent with the design strength specifications of Sections 6.4.2 and 6.5.2.3. Numerical quantities in Figure 6-1 shall be derived from tests, rational analyses, or criteria of

Section 6.7.2.2.2, with braces modeled as columns in accordance with Table 6-16.

6.10.3.3 Nonlinear Dynamic Procedure

Nonlinear load-deformation relations for use in analysis by Nonlinear Dynamic Procedure (NDP) shall model the complete hysteretic behavior of each component using properties verified by experimental evidence. Unloading and reloading properties shall represent stiffness and strength degradation characteristics.

6.10.4 Strength

Component strengths shall be computed according to the general requirements of Sections 6.4.2 and the additional requirements of Section 6.5.2.3. The possibility of instability of braces in compression shall be considered.

6.10.5 Acceptance Criteria

6.10.5.1 Linear Static and Dynamic Procedures

All actions shall be classified as being either deformation-controlled or force-controlled, as defined in Section 2.4.4. In primary components, deformation-controlled actions shall be restricted to flexure and axial actions in beams and columns, and axial actions in braces. In secondary components, deformation-controlled actions shall be restricted to those actions identified for the braced or isolated frame in this chapter.

Calculated component actions shall satisfy the requirements of Section 3.4.2.2. The *m*-factors for concrete frames shall be as specified in other applicable sections of this chapter, and *m*-factors for beams, columns, and braces modeled as tension and compression components shall be as specified for columns in Table 6-17. The *m*-factors shall be reduced to half the values in that table, but need not be less than 1.0, where component buckling is a consideration. Alternate approaches or values shall be permitted where justified by experimental evidence and analysis.

6.10.5.2 Nonlinear Static and Dynamic Procedures

Calculated component actions shall satisfy the requirements of Section 3.4.2.2 and shall not exceed the numerical values listed in Table 6-16 or the relevant tables for isolated frames specified in other sections of this chapter. Where inelastic action is indicated for a component or action not listed in these tables, the performance shall be deemed unacceptable. Alternate approaches or values shall be permitted where justified by experimental evidence and analysis.

6.10.6 Rehabilitation Measures

Concrete-braced frame components that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 6.4.7 and other provisions of this standard.

C6.10.6 Rehabilitation Measures

Rehabilitation measures that may be effective in rehabilitating concrete-braced frames include the general approaches listed for other concrete elements in this chapter, plus other approaches based on rational principles.

6.11 Cast-in-Place Concrete Diaphragms

6.11.1 Components of Concrete Diaphragms

Cast-in-place concrete diaphragms transmit inertial forces within a structure to vertical lateral-force-resisting elements.

Concrete diaphragms shall be made up of slabs, struts, collectors, and chords. Alternatively, diaphragm action may be provided by a structural truss in the horizontal plane. Diaphragms consisting of structural concrete topping on metal deck shall comply with the requirements of Section 5.9.2.

6.11.1.1 Slabs

Slabs shall consist of cast-in-place concrete systems that, in addition to supporting gravity loads, transmit inertial loads developed within structure from one vertical lateral-force-resisting element to another, and provide out-of-plane bracing to other portions of the building.

6.11.1.2 Struts and Collectors

Collectors are components that serve to transmit the inertial forces within the diaphragm to elements of the lateral-force-resisting system. Struts are components of a structural diaphragm used to provide continuity around an opening in the diaphragm. Struts and collectors shall be monolithic with the slab, occurring either within the slab thickness or being thicker than the slab.

6.11.1.3 Diaphragm Chords

Diaphragm chords are components along diaphragm edges with increased longitudinal and transverse reinforcement, acting primarily to resist tension and compression forces generated by bending in the diaphragm. Exterior walls shall be permitted to serve as chords provided there is adequate strength to transfer shear between the slab and wall.

C6.11.1.3 Diaphragm Chords

When evaluating an existing building, special care should be taken to evaluate the condition of the lap splices. Where the splices are not confined by closely spaced transverse reinforcement, splice failure is possible if stress levels reach critical values. In rehabilitation construction, new laps should be confined by closely spaced transverse reinforcement.

6.11.2 Analysis, Modeling, and Acceptance Criteria

6.11.2.1 General Considerations

The analytical model for a diaphragm shall represent the strength, stiffness, and deformation capacity of each component and the diaphragm as a whole. Potential failure in flexure, shear, buckling, and reinforcement development shall be considered.

Modeling of the diaphragm as a continuous or simple span horizontal beam supported by elements of varying stiffness shall be permitted. The beam shall be modeled as rigid, stiff, or flexible considering the deformation characteristics of the actual system.

C6.11.2.1 General Considerations

Some computer models assume a rigid diaphragm. Few cast-in-place diaphragms would be considered flexible; however, a thin concrete slab on a metal deck might be stiff depending on the length-to-width ratio of the diaphragm.

6.11.2.2 Stiffness

Diaphragm stiffness shall be modeled according to Section 6.11.2.1 and shall be determined using a linear elastic model and gross section properties. The modulus of elasticity used shall be that of the concrete as specified in *ACI 318*. When the length-to-width ratio of the diaphragm exceeds 2.0 (where the length is the distance between vertical elements), the effects of diaphragm flexibility shall be considered when assigning lateral forces to the resisting vertical elements.

C6.11.2.2 Stiffness

The concern is for relatively flexible vertical members that may be displaced by the diaphragm, and for relatively stiff vertical members that may be overloaded by the same diaphragm displacement.

6.11.2.3 Strength

Strength of cast-in-place concrete diaphragm components shall comply with the requirements of Sections 6.4.2 as modified in this section.

The maximum component strength shall be determined considering potential failure in flexure, axial load, shear, torsion, development, and other actions at all points in the component under the actions of design gravity and lateral load combinations. The shear strength shall be as specified in Chapter 21 of *ACI 318*. Strut, collector, and chord strengths shall be as determined for frame components in Section 6.5.2.3.

6.11.2.4 Acceptance Criteria

Diaphragm shear and flexure shall be considered deformation-controlled. Acceptance criteria for slab component actions shall be as specified for shear walls in Section 6.8.2.4, with m values taken according to similar components in Tables 6-20 and 6-21 for use in Equation (3-20). Acceptance criteria for struts, chords, and collectors shall be as specified for frame components in Section 6.5.2.4. Connections shall be considered force-controlled.

6.11.3 Rehabilitation Measures

Concrete diaphragms that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 6.4.7 and other provisions of this standard.

C6.11.3 Rehabilitation Measures

Cast-in-place concrete diaphragms can have a wide variety of deficiencies; see Chapter 10 and *FEMA 310*.

Two general alternatives may be effective in correcting deficiencies: either improve the strength and ductility, or reduce the demand in accordance with *FEMA 172*. Providing additional reinforcement and encasement may be an effective measure to strengthen or improve individual components. Increasing the diaphragm thickness may also be effective, but the added weight may overload the footings and increase the seismic loads. Lowering seismic demand by providing additional lateral-force-resisting elements, introducing additional damping, or base isolating the structure may also be effective rehabilitation measures.

6.12 Precast Concrete Diaphragms

6.12.1 Components of Precast Concrete Diaphragms

Precast concrete diaphragms are elements composed primarily of precast components with or without topping, that transmit shear forces from within a structure to vertical lateral-force-resisting elements.

Precast concrete diaphragms shall be classified as topped or untopped. A topped diaphragm shall be defined as one that includes a reinforced structural concrete topping slab poured over the completed precast horizontal system. An untopped diaphragm shall be defined as one constructed of precast components without a structural cast-in-place topping.

C6.12.1 Components of Precast Concrete Diaphragms

Section 6.11 provided a general overview of concrete diaphragms. Components of precast concrete diaphragms are similar in nature and function to those of cast-in-place diaphragms with a few critical differences. One is that precast diaphragms do not possess the inherent unity of cast-in-place monolithic construction. Additionally, precast components may be highly stressed due to prestressed forces. These forces cause long-term shrinkage and creep, which shorten the component over time. This shortening tends to fracture connections that restrain the component.

Most floor systems have a topping system, but some hollow core floor systems do not. The topping slab generally bonds to the top of the precast elements, but may have an inadequate thickness at the center of the span, or may be inadequately reinforced. Also, extensive cracking of joints may be present along the panel joints. Shear transfer at the edges of precast concrete diaphragms is especially critical.

Some precast roof systems are constructed as untopped systems. Untopped precast concrete diaphragms have been limited to lower seismic zones by recent versions of the Uniform Building Code. This limitation has been imposed because of the brittleness of connections and lack of test data concerning the various precast systems. Special consideration shall be given to diaphragm chords in precast construction.

6.12.2 Analysis, Modeling, and Acceptance Criteria

Analysis and modeling of precast concrete diaphragms shall conform to Section 6.11.2.2, with the added requirement that the analysis and modeling shall account for the segmental nature of the individual components.

Component strengths shall be determined in accordance with Section 6.11.2.3. Welded connection strength shall be based on rational procedures, and connections shall be assumed to have little ductility capacity unless test data verify higher ductility values. Precast concrete diaphragms with reinforced concrete topping slabs shall be considered deformation-controlled in shear and flexure. *m*-factors shall be taken as 1.0, 1.25, and 1.5 for IO, LS, and CP performance levels, respectively.

Untopped precast concrete diaphragms shall be considered force-controlled.

C6.12.2 Analysis, Modeling, and Acceptance Criteria

Welded connection strength can be determined using the latest version of the *Precast Concrete Institute (PCI) Handbook.*

6.12.3 Rehabilitation Measures

Precast concrete diaphragms that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 6.4.7 and other provisions of this standard.

C6.12.3 Rehabilitation Measures

Section 6.11.3 provides guidance for rehabilitation measures for concrete diaphragms in general. Special care should be taken to overcome the segmental nature of precast concrete diaphragms, and to avoid damaging prestressing strands when adding connections.

6.13 Concrete Foundation Elements

6.13.1 Types of Concrete Foundations

Foundations shall be defined as those elements that serve to transmit loads from the vertical structural subsystems (columns and walls) of a building to the supporting soil or rock. Concrete foundations for buildings shall be classified as either shallow or deep foundations as defined in Chapter 4. Requirements of Section 6.13 shall apply to shallow foundations that include spread or isolated footing, strip or line footing, combination footing, and concrete mat footing; and to deep foundations that include pile foundations and cast-in-place piers. Concrete-grade beams shall be permitted in both shallow and deep foundation systems and shall comply with the requirements of Section 6.13.

The provisions of Section 6.13 shall apply to existing foundation elements and to new materials or elements that are required to rehabilitate an existing building.

6.13.1.1 Shallow Foundations

Existing spread footings, strip footings, and combination footings are reinforced or unreinforced. Vertical loads are transmitted by these footings to the soil by direct bearing; and lateral loads are transmitted by a combination of friction between the bottom of the footing and the soil, and passive pressure of the soil on the vertical face of the footing.

Concrete mat footings shall be reinforced to resist the flexural and shear stresses resulting from the superimposed concentrated and line structural loads and the distributed resisting soil pressure under the footing. Lateral loads shall be resisted by friction between the soil and the bottom of the footing, and by passive pressure developed against foundation walls that are part of the system.

6.13.1.2 Deep Foundations

6.13.1.2.1 Driven Pile Foundations

Concrete pile foundations shall be composed of a reinforced concrete pile cap supported on driven piles. The piles shall be concrete (with or without prestressing), steel shapes, steel pipes, or composite (concrete in a driven steel shell). Vertical loads shall be transmitted to the piling by the pile cap, and are resisted by direct bearing of the pile tip in the soil or by skin friction or cohesion of the soil on the surface area of the pile. Lateral loads are resisted by passive pressure of the soil on the vertical face of the pile cap, in combination with interaction of the piles in bending and passive soil pressure on the pile surface.

6.13.1.2.2 Cast-in-Place Pile Foundations

Cast-in-place concrete pile foundations shall consist of reinforced concrete placed in a drilled or excavated shaft. Cast-in-place pile or pier foundations shall resist vertical and lateral loads in the same manner as that of driven pile foundations specified in Section 6.13.1.2.1.

C6.13.1.2 Deep Foundations

C6.13.1.2.1 Driven Pile Foundations

In poor soils, or soils subject to liquefaction, bending of the piles may be the only dependable resistance to lateral loads.

C6.13.1.2.2 Cast-in-Place Pile Foundations

Segmented steel cylindrical liners are available to form the shaft in weak soils and allow the liner to be removed as the concrete is placed. Various slurry mixes are often used to protect the drilled shaft from caving soils. The slurry is then displaced as the concrete is placed by the tremie method.

6.13.2 Analysis of Existing Foundations

For concrete buildings with columns or walls cast monolithically with the foundation, the vertical structural elements shall be considered fixed against rotation at the top of the foundation if the foundations and supporting soil are shown to be capable of resisting the induced moments. When columns are not monolithic with their foundations, or are designed to not resist flexural moments, they shall be modeled with pinned ends. In such cases, the column base shall be evaluated for the resulting axial and shear forces as well as the ability to accommodate the necessary end rotation of the columns. The effects of base fixity of columns shall be taken into account at the point of maximum displacement of the superstructure.

If a more rigorous analysis procedure is used, appropriate vertical, lateral, and rotational soil springs shall be incorporated in the analytical model as described in Section 4.4.2. The spring characteristics shall be as specified in Chapter 4. Rigorous analysis of structures with deep foundations in soft soils shall be based on special soil/pile interaction studies to determine the probable location of the point of fixity in the foundation and the resulting distribution of forces and displacements in the superstructure. In these analyses, the appropriate representation of the connection of the pile to the pile cap shall be included in the model. Piles with less than six inches of embedment without any dowels into the pile cap shall be modeled as being “pinned” to the cap. Unless the pile and pile cap connection detail is identified as otherwise from the available construction documents, the “pinned” connection shall be used in the analytical model.

When the foundations are included in the analytical model, the responses of the foundation components shall be considered. The reactions of structural elements attached at the foundation (axial loads, shears, and moments) shall be used to evaluate the individual components of the foundation system.

C6.13.2 Analysis of Existing Foundations

Overturning moments and economics may dictate the use of more rigorous analysis procedures.

6.13.3 Evaluation of Existing Condition

Allowable soil capacities (subgrade modulus, bearing pressure, passive pressure) and foundation displacements for the selected performance level shall be as prescribed in Chapter 4 or as established with project-specific data. All components of existing foundation elements and all new material, components, or elements required for rehabilitation shall be considered to be force-controlled based on the mechanical and analytical properties in Section 6.3.3. However, the capacity of the foundation components need not exceed 1.25 times the capacity of the supported vertical structural component or element (column or wall).

6.13.4 Rehabilitation Measures

Existing foundations that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 6.4.7 and other provisions of this standard.

C6.13.4 Rehabilitation Measures

Rehabilitation measures described in Section C6.13.4.1 for shallow foundations and in Section C6.13.4.2 for deep foundations may be effective in rehabilitating existing foundations.

C6.13.4.1 Rehabilitation Measures for Shallow Foundations

1. **Enlarging the existing footing by lateral additions**. Enlarging the existing footing may be an effective rehabilitation measure. The enlarged footing may be considered to resist subsequent actions produced by the design loads, provided that adequate shear and moment transfer capacity are provided across the joint between the existing footing and the additions.

2. **Underpinning the footing.** Underpinning an existing footing involves the removal of unsuitable soil underneath, coupled with replacement using concrete, soil cement, suitable soil, or other material. Underpinning should be staged in small increments to prevent endangering the stability of the structure. This technique may be used to enlarge an existing footing or to extend it to a more competent soil stratum.

3. **Providing tension hold-downs.** Tension ties (soil and rock anchors—prestressed and unstressed) may be drilled and grouted into competent soils and anchored in the existing footing to resist uplift. Increased soil bearing pressures produced by the ties should be checked against the acceptance criteria for the selected Performance Level specified in Chapter 4. Piles or drilled piers may also be effective in providing tension hold-downs of existing footings.

4. **Increasing effective depth of footing.** This method involves pouring new concrete to increase shear and moment capacity of the existing footing. The new concrete must be adequately doweled or otherwise connected so that it is integral with the existing footing. New horizontal reinforcement should be provided, if required, to resist increased moments.

5. **Increasing the effective depth of a concrete mat foundation with a reinforced concrete overlay.** This method involves pouring an integral topping slab over the existing mat to increase shear and moment capacity.

6. **Providing pile supports for concrete footings or mat foundations.** Adding new piles may be effective in providing support for existing concrete footing or mat foundations, provided the pile locations and spacing are designed to avoid overstressing the existing foundations.

7. **Changing the building structure to reduce the demand on the existing elements.** This method involves removing mass or height of the building or adding other materials or components (such as energy dissipation devices) to reduce the load transfer at the base level. New shear walls or braces may be provided to reduce the demand on existing foundations.

8. **Adding new grade beams.** This approach involves the addition of grade beams to tie existing footings together when poor soil exists, to provide fixity to column bases, and to distribute lateral loads between individual footings, pile caps, or foundation walls.

9. **Improving existing soil.** This approach involves grouting techniques to improve existing soil.

C6.13.4.2 Rehabilitation Measures for Deep Foundations

1. **Providing additional piles or piers.** Providing additional piles or piers may be effective, provided extension and additional reinforcement of existing pile caps comply with the requirements for extending existing footings in Section C6.13.4.1.

2. **Increasing the effective depth of the pile cap.** New concrete and reinforcement to the top of the pile cap may be effective in increasing its shear and moment capacity, provided the interface is designed to transfer actions between the existing and new materials.

3. **Improving soil adjacent to existing pile cap.** Soil improvement adjacent to existing pile caps may be effective if undertaken in accordance with guidance provided in Section 4.3.

4. **Increasing passive pressure bearing area of pile cap.** Addition of new reinforced concrete extensions to the existing pile cap may be effective in increasing the vertical foundation bearing area and load resistance.

5. **Changing the building system to reduce the demands on the existing elements.** New lateral-load-resisting elements may be effective in reducing demand.

6. **Adding batter piles or piers.** Adding batter piles or piers to existing pile or pier foundation may be effective in resisting lateral loads. It should be noted that batter piles have performed poorly in recent earthquakes when liquefiable soils were present. This is especially important to consider around wharf structures and in areas having a high water table. Addition of batter pile to foundations in areas of such seismic hazards should be in accordance with requirements in [Sections 4.3.2 and 4.4.2.2B].

7. **Increasing tension tie capacity from pile or pier to superstructure.** Added reinforcement should satisfy the requirements of Section 6.4.