Chapter 6: Concrete

6.0 Concrete

6.1 Scope

This chapter sets forth requirements for the Systematic Rehabilitation of concrete components of the lateral-force-resisting system of an existing 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. The provisions of this chapter do not apply to concrete encased steel composite components.

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

Concrete encased steel composite components frequently behave as over-reinforced sections. This type of component behavior was not represented in the data sets used to develop the force-deformation modeling relationships and acceptance criteria in Chapter 6. Further, the concrete encasement is often provided for fire protection rather than for strength or stiffness, and typically lacks confinement reinforcement or the confinement reinforcement does not meet detailing requirements in the AISC Code (AISC 2005). The lack of adequate confinement may result in large dilation strains which exacerbate bond slip and, consequently, undermine the fundamental principle that plane sections remain plane.

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

6.2 Material Properties and Condition Assessment

6.2.1 General

Mechanical properties of concrete materials and components shall be obtained from available drawings, specifications, and other documents for the existing construction in accordance with the requirements of Section 2.2. Where such documents fail to provide adequate information to
quantify concrete material properties or the condition of concrete components of the structure, such information shall be supplemented by materials tests and assessments of existing conditions in compliance with requirements of this chapter as specified in Section 2.2.6.

Material properties of existing concrete components shall be determined in accordance with Section 6.2.2. A condition assessment shall be conducted in accordance with Section 6.2.3. The extent of materials testing and condition assessment performed shall be used to determine the knowledge factor as specified in Section 6.2.4.

Use of default material properties shall be permitted in accordance with Section 6.2.2.5. Use of material properties based on historical information as default values shall be permitted as specified in Section 6.2.2.5.

C6.2.1 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 should be experienced in the proper implementation of testing practices and the interpretation of results.

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 where 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.
Documentation of properties and grades of material used in component and 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.

6.2.2 Properties of In-Place Materials and Components

6.2.2.1 Material Properties

6.2.2.1.1 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.

Where materials testing is required by Section 2.2.6, the test methods to quantify material properties shall comply with the requirements of Section 6.2.2.3. The frequency of sampling, including the minimum number of tests for property determination shall comply with the requirements of Section 6.2.2.4.

C6.2.2.1.1 General

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

1. Tensile strength and modulus of elasticity of concrete, 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, the quality and type of construction (absence of degradation), accessibility, and the 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 damage of the existing reinforcing steel as much as is practicable.

6.2.2.1.2 Nominal or Specified Properties

Nominal material properties, or properties specified in construction documents, shall be taken as lower-bound material properties. Corresponding expected material properties shall be calculated by multiplying lower-bound values by a factor taken from Table 6-4 to translate from lower-bound to expected values. Alternative factors shall be permitted where justified by test data.

6.2.2.2 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.


5. Presence of conditions that influence building performance.

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 should be performed to identify primary gravity- and lateral-force-resisting 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 investigation of the building to identify these elements, systems and components as indicated in Section 6.2.3.

6.2.2.3 Test Methods to Quantify Material Properties

6.2.2.3.1 General

Destructive and non-destructive test methods used to obtain in-place mechanical properties of materials identified in Section 6.2.2.1 and component properties identified in Section 6.2.2.2 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 6.2.3.2.
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.

Where Section 6.2.2.4.1 does not apply and the coefficient of variation is greater than 14%, the expected concrete strength shall not exceed the mean minus one standard deviation.

### 6.2.2.3.2 Sampling

For testing of concrete material, the sampling program shall consist of the removal of standard cores. Core drilling shall be preceded by nondestructive location of the reinforcing steel, and core holes shall be located to minimize damage to or drilling through the reinforcing steel. Core holes shall be filled with concrete or grout of comparable strength. 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.


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 by 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.

### C6.2.2.3 Test Methods to Quantify Material Properties

ACI 318 and FEMA 274 provide further guidance on correlating core strength to in-place strength and provide 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.

6.2.2.4 Minimum Number of Tests

Materials testing is not required if material properties are available from original construction
documents that include material test records or 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 6.2.2.4.1 through 6.2.2.4.4. The
minimum number of tests for usual data collection shall be as specified in Section 6.2.2.4.5. If
the existing gravity- or lateral-force-resisting system is being replaced in the rehabilitation
process, material testing shall be required only to quantify properties of existing materials at new
connection points.

6.2.2.4 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 of the lateral-force-resisting system. The minimum number
of tests is dictated by the data available from original construction, the type of structural system
employed, the 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 generally is 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.

6.2.2.4.1 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%.

6.2.2.4.2 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.

**C6.2.2.4.2 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.

**6.2.2.4.3 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
6.2.2.4 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 the 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.

6.2.2.4.5 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 the building for testing.

C6.2.2.4.5 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.

6.2.2.5 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 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. Where 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.

C6.2.2.5 Default Properties

Default values provided in this standard are generally conservative. While the strength of reinforcing steel may be fairly consistent throughout a building, the strength of concrete in a building could be highly variable, given variability in concrete mix designs and sensitivity to water-cement ratio and curing practices. It is recommended to conservatively assume the minimum value of the concrete compressive strength in the given range unless a higher strength is substantiated by construction documents, test reports, or material testing; it would be conservative to assume the maximum value in a given range where determining the force-controlled actions on other components.

Until about 1920, a variety of proprietary reinforcing steels was used. Yield strengths are likely to be in the range of 33,000 to 55,000 psi, 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.

Table 6-1 Default Lower-Bound Tensile and Yield Properties of Reinforcing Bars for Various Periods1 [Refer to end of chapter]
6.2.3 Condition Assessment

6.2.3.1 General

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

The condition assessment shall include the following:

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. Information needed to select a knowledge factor in accordance with Section 6.2.4 shall be obtained.

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

6.2.3.2 Scope and Procedures

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

C6.2.3.2 Scope and Procedures

The degree to which the condition assessment is performed will affect the knowledge ($\kappa$) factor as specified in Section 6.2.4.

6.2.3.2.1 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 percent of the components and connections shall be visually inspected at each floor level. If significant damage or degradation is found, the assessment sample of all critical components of similar type in the building shall be increased to 40 percent.

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

### 6.2.3.2.2 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.

### 6.2.3.2.3 Additional Testing

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

### C6.2.3.2.3 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 pachometer) 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.

### 6.2.3.3 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.

6.2.4 Knowledge Factor

A knowledge factor ($\kappa$) 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, $\kappa$, 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 $\kappa=1.0$.

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

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

6.3 General Assumptions and Requirements

6.3.1 Modeling and Design

6.3.1.1 General Approach

Seismic rehabilitation of concrete structural components 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.

C6.3.1.1 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.

6.3.1.2 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, the extent of cracking due to volumetric changes from temperature and shrinkage, 
and to deformation levels to which the component will be subjected under gravity and 
earthquake loading.

Table 6-5 Effective Stiffness Values Component Flexural Rigidity
Shear Rigidity Axial Rigidity [Refer to end of chapter]

C6.3.1.2 Stiffness

For columns with low axial loads, deformations due to bar slip can account for as much as 50% 
of the total deformations at yield. The design professional is referred to Elwood and Eberhard 
(2006) for further guidance regarding calculation of effective stiffness of reinforced concrete 
columns to include the effects of flexure, shear and bar slip.

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

The effective flexural rigidity values given in Table 6-5 for beams and columns account for the 
additional flexibility resulting from reinforcement slip within the beam-column joint or 
foundation prior to yielding. The values specified for columns were determined based on a 
database of 221 rectangular reinforced concrete column tests with axial loads less than 0.67 \( A_g f_c' \) 
and shear span-to-depth ratios greater than 1.4. Measured effective stiffnesses from the 
laboratory test data suggest that the effective flexural rigidity for low axial loads could be 
approximated as 0.2 \( EI_g \); however, considering the scatter in the effective flexural rigidity and to 
avoid under-estimating the shear demand on columns with low axial loads, 0.3 \( EI_g \) is 
recommended in Table 6-5. In addition to axial load, the shear span-to-depth ratio of the column 
influences the effective flexural rigidity. A more refined estimate of the effective flexural 
rigidity can be determined by calculating the displacement at yield due to flexure, slip, and shear 
(Elwood and Eberhard, 2006).

Note that the modeling recommendations for beam-column joints (section 6.4.2.2.1) do not 
include the influence of reinforcement slip. When the effective stiffness values for beams and
columns from Table 6-5 are used in combination with the modeling recommendations for beam-
column joints, the overall stiffness is in close agreement with results from beam-column
subassembly tests.

The effect of reinforcement slip can be accounted for by including rotational springs at the ends
of the beam or column elements (Saatcioglu et al. 1992). If this modeling option is selected, the
effective flexural rigidity of the column element should reflect only the flexibility due to flexural
deformations. In this case, for axial loads less than $0.3 A_{dlc}'$, the effective flexural rigidity can be
estimated as $0.5EI_{c}$, with linear interpolation to the value given in Table 6-5 for axial loads
greater than $0.5 A_{dlc}'$.

Components with plain longitudinal reinforcement (without deformations) and axial loads less
than $0.5 A_{dlc}'$ may have lower effective flexural rigidity values than those given in Table 6-5 due
to the low bond stress between the concrete and steel.

6.3.1.2.2 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 6.4 through 6.12. 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 6.3.2.1.

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 6.3.1.2.1. 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 6.4 through 6.12. Other load-deformation relations
shall be permitted if justified by experimental evidence or analysis.

Figure 6-1 Generalized Force-Deformation Relations for
Concrete Elements or Components [Refer to end of chapter]
C6.3.1.2.2 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. Where 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. Where 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.

Provisions for determining alternative modeling parameters and acceptance criteria based on experimental evidence are given in Section 2.8.

Displacement demands determined from nonlinear dynamic analysis are very sensitive to the rate of strength degradation included in the structural model. Unless there is experimental evidence of sudden strength loss for the particular component under consideration, use of a model with a sudden strength loss from point C to D in Figure 6-1 can result in an overestimation of the drift demands for a structural system and individual components. A more realistic model for many concrete components would have a linear degradation in resistance from point C to point E.

It is also noted that strength loss which occurs within a single cycle can result in dynamic instability of the structure, while strength loss which occurs between cycles is unlikely to cause such instability. The model shown in Figure 6-1 does not distinguish between these types of strength degradation, and may not accurately predict the displacement demands if the two forms of strength degradation are not properly accounted for.

6.3.1.3 Flanged Construction
In beams consisting of a web and flange that act integrally, the combined stiffness and strength for flexural and axial loading shall be calculated considering a width of effective flange on each side of the web equal to the 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. Where the flange is in compression, both the concrete and reinforcement within the effective width shall be considered effective in resisting flexure and axial load. Where the flange is in tension, longitudinal reinforcement within the effective width 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.

6.3.2 Strength and Deformability

6.3.2.1 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 6.3.2.2 and 6.3.2.3, respectively.

Components shall be classified as having low, moderate, or high ductility demands according to Section 6.3.2.4.

Where strength and deformation capacities are derived from test data, the tests shall be representative of proportions, details, and stress levels for the component and comply with 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 deformation 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.

C6.3.2.1 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.

6.3.2.2 Deformation-Controlled Actions
Strengths used for deformation-controlled actions shall be taken as equal to expected strengths, $Q_{CE}$, obtained experimentally, or calculated using accepted principles of mechanics. 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. Where 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, $\phi$, shall be taken equal to unity. Deformation capacities for acceptance of deformation-controlled actions calculated by nonlinear procedures shall be as specified in Sections 6.4 to Section 6.12. For components constructed of lightweight concrete, $Q_{CE}$ shall be modified in accordance with ACI 318 procedures for lightweight concrete.

### C6.3.2.2 Deformation-Controlled Actions

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

### 6.3.2.3 Force-Controlled Actions

Strengths used for force-controlled actions shall be taken as lower-bound strengths, $Q_{CL}$, obtained experimentally, or calculated using established principles of mechanics. Lower-bound strength is defined as the mean 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. Where calculations are used to define lower-bound strengths, lower-bound estimates of material properties shall be used. Unless other procedures are specified in this standard, procedures specified in ACI 318 to calculate design strengths shall be permitted, except that the strength reduction factor, $\phi$, 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.

### 6.3.2.4 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 | [Refer to end of chapter] |

### 6.3.3 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 6.3.1.3.

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 6.3.5.

Where flexural deformation capacities are calculated from basic principles of mechanics, reductions in deformation capacity due to applied shear shall be taken into consideration. Where 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. Where 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:

\[
\left(\frac{M_{UDx}}{m_x \kappa M_{CEx}}\right)^2 + \left(\frac{M_{UDy}}{m_y \kappa M_{CEy}}\right)^2 \leq 1
\]  

(6-1)

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 in accordance with Table 6-12
\(m_y\) = \(m\)-factor for column for bending about y axis in accordance with Table 6-12

Alternative approaches based on principles of mechanics shall be permitted.

### C6.3.3 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 where flexural deformation capacities are determined by calculation. FEMA 306 is a resource for guidance regarding the interaction between shear and flexure.

6.3.3.1 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 by the authority having jurisdiction. 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. Monotonic test results shall not be used to determine reinforcement strain limits. If experimental evidence is used to determine strain limits, the effects of spacing and size of transverse reinforcement and of low-cycle fatigue shall be included in the testing procedures, and results are subject to the approval of the authority having jurisdiction.

6.3.4 Shear and Torsion

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

Within yielding regions of components with moderate or high ductility demands, shear and torsional strength shall be calculated according to procedures for ductile components, such as the provisions in Chapter 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 $\mu$ shall be taken as equal to 70% of the value specified by ACI 318.

### 6.3.5 Development and Splices of Reinforcement

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

1. Deformed straight bars, hooked bars, and lap-spliced 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):

$$f_s = \frac{l_b}{l_d} f_y = 1.25 \left( \frac{l_b}{l_d} \right)^{2/5} f_y$$

but shall not exceed the expected or lower-bound yield strength, as applicable. In equation 6-2, where $f_s$ = maximum stress that can be developed in the bar for the straight development, hook development, or lap splice length $l_b$ provided; $f_y$ = lower bound yield strength of reinforcement; and $l_d$ = length required by Chapter 12 of ACI 318 for straight development, hook development, or lap splice length, except that 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 $f_s$ at a ductility demand or DCR equal to 1.0 to $0.2f_s$ at a ductility demand or DCR equal to 2.0.

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

$$f_s = \frac{2500}{d_b} l_e \leq f_y$$

(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). Where \( f_s \) is less than \( f_y \), and the calculated stress in the bar due to design loads equals or exceeds \( f_s \), the maximum developed stress shall be assumed to degrade from \( f_s \) to 0.2\( f_s \) 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 where 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 10\( d_b \).

   6. Minimum spacing of dowel bars is not less than 4\( l_e \) and minimum edge distance is not less than 2\( l_e \). 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 6.3.

C6.3.5 Development and Splices of Reinforcement

Development requirements in accordance with Chapter 12 of ACI 318 will be applicable to development of bars in all components. Chapter 21 of ACI 318 provides development requirements that are only intended for use in yielding components of reinforced concrete moment frames that comply with the cover and confinement provisions of Chapter 21. Chapter 12 permits reductions in lengths if minimum cover and confinement exist in an existing component.

Experimental tests by Melek and Wallace (2004) and Lynn (2001) have demonstrated that lap splices can achieve a higher flexural capacity than that calculated using the effective steel stress given in Equation (6-2). The possibility of a shear failure in lap-spliced columns may go undetected if the flexural capacity is underestimated. Cho and Pincheira (2006) suggest an alternative model for the effective steel stress in lap splice bars which provides a better estimate of the mean flexural strength observed in experimental tests. Equation 6-2 is a modified version of the model presented by Cho and Pincheira (2006). Equation 6-2 reflects the intent of the ACI Code development and splice equations to develop 1.25 times the nominal bar strength (referred in this document as the lower bound yield strength). The nonlinear relation between developed stress and development length reflects the effect of increasing slip, hence, reduced unit bond strength, for longer development lengths.
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 \textit{CRSI}.

\section*{6.3.5.1 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 6.3.5, 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 6.3.5.

\section*{6.3.6 Connections to Existing Concrete}
Connections used to connect two or more components shall be classified according to their anchoring systems as cast-in-place or as post-installed.

\subsection*{6.3.6.1 Cast-In-Place Systems}
Component actions on cast-in-place connection systems, including shear forces, tension forces, bending moments, and prying actions, 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.

\subsection*{6.3.6.2 Drilled-In Anchors}
Component actions on drilled-in anchor connection systems shall be considered force-controlled. The lower-bound capacity of drilled-in anchors shall be mean minus one standard deviation of ultimate values published in approved test reports.

\subsection*{6.3.6.3 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.

\section*{6.3.7 Rehabilitation-General Requirements}
Upon determining that concrete components in an existing building are deficient for the selected Rehabilitation Objective, these components shall be rehabilitated or replaced or the structure shall be otherwise rehabilitated so that the component is no longer deficient for the selected rehabilitation objective. If replacement of the component is selected, the new component shall be designed in accordance with this standard and detailed and constructed in accordance with a building code approved by the authority having jurisdiction.

Rehabilitation measures shall be evaluated in accordance with the 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 components shall satisfy the requirements of Section 6.3.6 and other requirements of this standard.

6.4 Concrete Moment Frames

6.4.1 Types of Concrete Moment Frames

Concrete moment frames shall be defined as elements comprising primarily 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.

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 6.5, 6.6, and 6.9, respectively.

6.4.1.1 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.

6.4.1.2 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.

6.4.1.3 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.

6.4.2 Reinforced Concrete Beam-Column Moment Frames

6.4.2.1 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 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 of 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.

For modeling purposes, 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 in accordance with Section 6.12.

Action of the slab as a diaphragm interconnecting vertical components 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 6.3.1.3.

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 6.4.2.4.

**6.4.2.2 Stiffness for Analysis**

**6.4.2.2.1 Linear Static and Dynamic Procedures**

Beams shall be modeled considering flexural and shear stiffnesses, including the effect of the slab acting as a flange in monolithic construction. Columns shall be modeled considering flexural, shear, and axial stiffnesses. Effective stiffnesses shall be computed according to Section 6.3.1.2. Where joint stiffness is not to be modeled explicitly, it shall be permitted to be modeled implicitly by adjusting a centerline model as follows; stiff or rigid components. Effective stiffnesses shall be according to Section 6.3.1.2.

1. For \( \frac{\Sigma M_{nc}}{\Sigma M_{nb}} > 1.2 \), column offsets are rigid and beam offsets are not.
2. For \( \frac{\Sigma M_{nc}}{\Sigma M_{nb}} < 0.8 \), beam offsets are rigid and column offsets are not.
3. For \( 0.8 \leq \frac{\Sigma M_{nc}}{\Sigma M_{nb}} \leq 1.2 \), column and beam offsets are half rigid.

where:

- \( \Sigma M_{nc} \) = the sum of nominal moment capacities of all columns framing into a joint
- \( \Sigma M_{nb} \) = the sum of nominal moment capacities of all beams framing into a joint
**C6.4.2.2.1 Linear Static and Dynamic Procedures**

Various approaches to explicitly model beam-column joints are available in the literature (e.g., Ghobarah and Biddah 1999; Lowes and Altoontash 2003). For simplicity, implementation in commercial structural analysis software, and agreement with calibration studies performed in the development of this standard, this section defines an implicit beam-column joint modeling technique using centerline models with semi-rigid joint offsets. Figure C6-1 shows an example of an explicit joint model and illustrates the implicit joint modeling approach. In the implicit joint model, only a portion of the beam and/or column within the geometric joint region is defined to be rigid. In typical commercial software packages, this portion can range from 0, in which case the model is a true centerline model, to 1.0, in which case the entire joint region is rigid. Note that this modeling approach only accounts for joint shear flexibility, and therefore appropriate stiffness values that include the flexibility resulting from bar slip should be used for the beams and/or columns. (see Section C6.3.1.2.1)

**Figure C6-1 Beam-Column Joint Modeling** (hatched portions are rigid)

**6.4.2.2 Nonlinear Static Procedure**

Nonlinear load-deformation relations shall follow the requirements of Section 6.3.1.2. 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 6.3.2 and 6.4.2.3.

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.

Columns not controlled by inadequate splices (condition iv in Table 6-8) shall be classified based on $V_p$ from Equation 6-4, the plastic shear capacity of the column, $V_p$ (i.e. shear demand at flexural yielding of plastic hinges), and the transverse reinforcement detailing, as shown below.

**Condition to be used in Table 6-8:**
C6.4.2.2.2  Nonlinear Static Procedure

The modeling parameters and acceptance criteria specified in Table 6-8 have been updated to reflect results from recent research on reinforced concrete columns. Section 6.4.2.2.2 provides the criteria to determine which condition in Table 6.8 should be used to select the modeling parameters and acceptance criteria. For columns with transverse reinforcement including 135° hooks, the specified conditions approximately correspond to the following failure modes:

- Condition i: Flexure failure
- Condition ii: Flexure-shear failure (where yielding in flexure is expected prior to shear failure)
- Condition iii: Shear failure

The specified condition is adjusted downward by one condition level for columns with 90° hooks or lap spliced transverse reinforcement to reflect the observation from experiments that poor transverse reinforcement details can result in decreased deformation capacity. The classification of columns based on \( V_p/(V_n/k) \) as described in Section 6.4.2.2.2 may be conservative for columns with \( V_p/(V_n/k) \approx 1.0 \) or \( V_p/(V_n/k) < 0.7 \). Experimental evidence may be used to determine the expected failure mode and select the appropriate modeling parameters.
The acceptance criteria in Table 6-8 are determined based on the modeling parameters “a” and “b” and the requirements of Chapter 2. The following paragraphs describe the methodology for selecting the modeling parameters “a” and “b” in Table 6-8.

The modeling parameters in Table 6-8 define the plastic rotations according to Figure 6-1(a). As shown in Figure 6-1(a), modeling parameter “a” provides the plastic rotation at significant loss of lateral load capacity. For the purposes of determining “a” values based on test data, it was assumed that this point represented a 20% reduction in the lateral load resistance from the measured peak shear capacity. For columns expected to experience flexural failures (condition i), such loss of lateral load resistance can be caused by concrete crushing, bar buckling, and other flexural damage mechanisms. For columns expected to experience shear failures, either before or after flexural yielding (conditions ii or iii), loss of lateral load resistance is commonly caused by severe diagonal cracking indicative of shear damage. Consistent with Section 2.4.4.3, modeling parameter “b” provides an estimate of the plastic rotation at the loss of gravity load support (i.e. axial load failure). Experimental evidence suggests that axial load failure can occur suddenly after lateral load failure for columns with axial loads above 0.6\( \frac{f'c}{A_g} \) (Sezen and Moehle 2006; Bayrak and Sheikh 1995). Based on this observation, the “a” and “b” parameters in Table 6-8 converge to a single value for high axial loads.

To achieve an appropriate estimate of the deformation capacities, interpolation between the values given in Table 6-8 is required. For Condition ii, the interpolation is performed on three variables, and this can be done in any order.

Considerable scatter exists in results from reinforced concrete column tested to lateral load and axial load failure, making it inappropriate to specify median or mean values for the plastic rotations in Table 6-8. The goal in selecting the values for parameter “a” given in Table 6-8 was to achieve a high level of safety (probability of failure, \( P_f \), less than 15%) for columns that may experience shear failures; but accept a slightly lower level of safety (\( P_f < 35\% \)) for columns that are expected to experience flexural failures. Given the potential of collapse resulting from axial load failure of individual columns, a high level of safety (\( P_f < 15\% \)) was also desired for parameter “b”. The target limits for the probabilities of failure given above were selected based on the judgment of the committee responsible for the development of Table 6-8.

To assess the degree of safety provided by Table 6-8, the tabulated values were interpolated and compared with data from laboratory tests on reinforced concrete columns appropriate for each of the conditions described above. Table C6-1 provides a summary of the results of this assessment. Note that the actual probabilities of failure achieved by the limits in Table 6-8 are considerably lower in many cases than the target probabilities of failure given above. Insufficient data exists to assess the probability of failure for parameter “b” for Conditions i, iii, and iv; however, limited experimental evidence suggests that the drift ratios for such columns will be greater than those for flexure-shear columns (Melek and Wallace 2004; Yoshimura et al. 2004), and hence, the “b” values for Condition ii are conservatively used for all conditions.

Table C6-1: Database results for modeling parameters in Table 6-8

<table>
<thead>
<tr>
<th>Modeling parameter</th>
<th>Number of tests</th>
<th>Mean(( \frac{\theta_{p \text{ meas}}}{\theta_{p \text{ table}}} ))</th>
<th>CoV(( \frac{\theta_{p \text{ meas}}}{\theta_{p \text{ table}}} ))</th>
<th>Probability of failure*</th>
</tr>
</thead>
</table>

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The database for modeling parameter “a” for Condition i only considered columns with $p^* \geq 0.002$ and $s/d \leq 0.5$, hence these limitations have been placed on the applicability of the modeling parameters for Condition i.

For columns expected to experience shear failure prior to flexural yielding (Condition iii), the deformation at shear failure is given by the effective stiffness of the component and the shear strength of the column ($V_n/k$ from Equation 6-4). Significant plastic deformations cannot be relied upon prior to shear failure; hence, parameter “a” has been set to zero. This assumption is very conservative for some columns since the classification method in Section 6.4.2.2.2 may result in some flexure-shear columns being classified as Condition iii and most will have some limited plastic rotation capacity prior to shear failure. Note that except for columns with high axial loads and very light transverse reinforcement, deformations beyond shear failure are expected prior to axial load failure.

Elwood and Moehle (2005b) have demonstrated that the drift at axial failure decreases as the following non-dimensional parameter increases:

$$\alpha = \frac{P}{A_f \rho_d s}$$

The database used to assess the probability of failure for parameter “b” included columns with $\alpha \leq 33$. Caution should be used when applying the values from Table 6-8 to columns with $\alpha > 33$.

The probabilities of failure shown in Table C6-1 were determined by considering $(\theta_p/p_{\text{table}})$ as a random variable with a lognormal distribution. Equation C6-1 shown below allows for the determination of the expected plastic rotation for a higher probability of failure, $P_{f_{\text{new}}}$:

$$\theta_p (P_{f_{\text{new}}}) = \theta_p \text{table} \exp \left[ \zeta \Phi^{-1} - \Phi^{-1} (P_{f_{\text{table}}}) \right] \quad (C6-1)$$

where $\zeta = \sqrt{\ln(1 + \beta^2)}$, $\beta$ is the coefficient of variation based on test data given in Table C6-1, $P_{f_{\text{table}}}$ is the probability of failure given in Table C6-1 and $\Phi^{-1}$ is the inverse standard normal cumulative distribution function (i.e. with a zero mean and unit standard deviation). The inverse standard normal cumulative distribution function, $\Phi^{-1}$, can be found in basic statistics textbooks and is available as a function in most spreadsheet programs.

Equation C6-1 can be used to establish the fragility curve (Figure C6-2) for the column which provides the probability of failure for a given normalized plastic rotation demand, $\frac{\theta_p}{\theta_p \text{table}}$.

Note that $P_f$ is the probability of failure for a column given a plastic rotation demand equal to $\theta_p$. The probability of failure considering the uncertainty in the ground motion is much lower than $P_f$. 

### Table 6-30: Seismic Rehabilitation Standard ASCE 41-06, Supplement No. 1

<table>
<thead>
<tr>
<th>Condition</th>
<th>Parameter</th>
<th>Value 1</th>
<th>Value 2</th>
<th>Value 3</th>
<th>Value 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>“a” for Condition i</td>
<td></td>
<td>141</td>
<td>1.44</td>
<td>0.50</td>
<td>30%</td>
</tr>
<tr>
<td>“a” for Condition ii</td>
<td></td>
<td>31</td>
<td>2.23</td>
<td>0.47</td>
<td>6%</td>
</tr>
<tr>
<td>“a” for Condition iii</td>
<td></td>
<td>34</td>
<td>4.66</td>
<td>0.48</td>
<td>0.1%</td>
</tr>
<tr>
<td>“b” for Condition ii</td>
<td></td>
<td>28</td>
<td>1.97</td>
<td>0.50</td>
<td>13%</td>
</tr>
</tbody>
</table>

* Assuming a lognormal distribution for $(\theta_{p,\text{meas}}/\theta_{p,\text{calc}})$.
The databases used to assess the conservatism of the models consisted of rectangular columns subjected to unidirectional lateral loads parallel to one face of the column. Actual columns have configurations and loadings that differ from those used in the database columns, so that some additional scatter in results may be anticipated. In particular, it should be noted that bidirectional loading on corner columns is expected to result in lower drift capacities; however, limited data exists to assess the degree of reduction anticipated.

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

Refer to Section C6.3.1.2 and C6.4.2.3.1 for discussion of alternative modeling parameters for reinforced concrete columns. Figure C6-3 illustrates the five beam-column joint classifications.

**Figure C6-2: Fragility curve for column**

**Figure C6-3 Joint Classification (for response in the plane of the page)**

### 6.4.2.3 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. Refer to Section 6.4.2.2.2 for the application of parameters in Table 6-8. Unloading and reloading properties shall represent significant stiffness and strength degradation characteristics.

### 6.4.2.3 Strength

Component strengths shall be computed according to the general requirements of Sections 6.3.2 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.

### 6.4.2.3.1 Columns

For columns, the shear strength, $V_n$ calculated according to Equation (6-4) shall be permitted.
\[ V_n = k \frac{A \cdot f_y \cdot d}{s} + \lambda k \left( \frac{6 \sqrt{f'_c} \cdot c}{M/Vd} \sqrt{1 + \frac{N_u}{6 \sqrt{f'_c} \cdot A_g}} \right) 0.8 A_g \]  

(6-4)

where \( k = 1.0 \) in regions where displacement ductility is less than or equal to 2, 0.7 in regions where displacement ductility is greater than or equal to 6, and varies linearly for displacement ductility between 2 and 6; \( \lambda = 0.75 \) for lightweight aggregate concrete and 1.0 for normal weight aggregate concrete; \( N_u = \) axial compression force in pounds (0 for tension force); \( M/Vd \) is the largest ratio of moment to shear times effective depth under design loadings for the column but shall not be taken greater than 4 or less than 2; \( d \) is the effective depth; and \( A_g \) is the gross cross-sectional area of the column. It shall be permitted to assume \( d = 0.8 h \), 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.

### C6.4.2.3.1 Columns

As discussed in C6.3.3, experimental evidence indicates 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.

The modeling parameters and acceptance criteria in Table 6-8 are generally conservative, and may be relaxed based on experimental evidence. The design professional is referred to reports by Berry and Eberhard 2005; Elwood and Moehle 2005a; Elwood and Moehle 2005b; Fardis and Biskinis 2003; Biskinis et al., 2004; Panagiotakos and Fardis 2001; Lynn et al., 1996; Sezen, 2002, and Elwood and Moehle, 2004 for further guidance regarding determination of modeling parameters and acceptance criteria for reinforced concrete columns.

Equation (6-4) provides an estimate of the mean observed shear strength for 51 rectangular reinforced concrete columns subjected to unidirectional lateral loads parallel to one face of the column (Sezen and Moehle, 2004). The coefficient of variation for the ratio of measured to calculated shear strength is 0.15. Elwood and Moehle (2005a) have demonstrated based on experimental evidence that Equation (6-4) does not provide a reliable estimate of the displacement ductility at shear failure.

### 6.4.2.3.2 Beam-Column Joints
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):

$$Q_{CL} = V_n = \lambda \gamma \sqrt{f'_c A_j} \text{ psi}$$

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 $\gamma$ is as defined in Table 6-10.

6.4.2.4 Acceptance Criteria

6.4.2.4.1 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 and indicated. 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) or (3-21), as applicable, shall comply with the performance criteria.

Where the average DCR $\bar{\text{eff}}$ for columns at a level exceeds the average value $\bar{\text{eff}}$ for 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 story element. For weak story 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 story 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 eliminate 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 element.

6.4.2.4.2 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 shear exceeds design shear strength, 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  [Refer to end of chapter]

Table 6-12 Numerical Acceptance Criteria for Linear Procedures-Reinforced Concrete Columns  [Refer to end of chapter]

Table 6-13 Numerical Acceptance Criteria for Linear Procedures-Reinforced Concrete Beam-Column Joints  [Refer to end of chapter]

C6.4.2.4.2 Nonlinear Static and Dynamic Procedures

Refer to Section C6.4.2.2.2 and C6.4.2.3.1 for discussion of alternative Table 6-8 and acceptance criteria for reinforced concrete columns.

6.4.2.5 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 6.3.7 and other provisions of this standard.

C6.4.2.5 Rehabilitation Measures

The following rehabilitation measures may be effective in rehabilitating reinforced concrete beam-column moment frames:
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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 components interfere with the frame, removing or separating the nonstructural 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 components and collectors for the new shear wall or braced frame, these should be checked for adequacy, considering strength, reinforcement development, and deformability. Diaphragms, including ties and collectors, should be evaluated and, if necessary, rehabilitated to ensure a complete load path to the new shear wall or braced frame element.

**6.4.3 Post-Tensioned Concrete Beam-Column Moment Frames**

**6.4.3.1 General Considerations**
The analytical model for a post-tensioned concrete beam-column frame element shall be established following the criteria specified in Section 6.4.2.1 for reinforced concrete beam-column moment frames. In addition to potential failure modes described in Section 6.4.2.1, 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.

6.4.3.2 Stiffness

6.4.3.2.1 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. Effective stiffnesses shall be computed according to Section 6.3.1.2. Joints stiffness shall be modeled as indicated in Section 6.4.2.2.1, either stiff or rigid components. Effective stiffnesses shall be according to Section 6.3.1.2.

6.4.3.2.2 Nonlinear Static Procedure

Nonlinear load-deformation relations shall comply with the requirements of Section 6.3.1.2 and the reinforced concrete frame requirements of Section 6.4.2.2.2.

Values of the generalized deformation at points \(B\), \(C\), and \(D\) in Figure 6-1 shall be either derived either from experiments or from 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 6.4.3.1 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 6.4.2.2.
6.4.3.2.3 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.

6.4.3.3 Strength

Component strengths shall be computed according to the general requirements of Sections 6.3.2 and the additional requirements of Section 6.4.2.3. 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.

6.4.3.4 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 6.4.2.4.

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

6.4.3.5 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 6.3.7 and other provisions of this standard.

6.4.4 Slab-Column Moment Frames

6.4.4.1 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. 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. Potential
failure in flexure, shear, shear-moment transfer (punching shear), and reinforcement
development at any section along the component length shall be considered. Interaction with
other components, including nonstructural components, shall be included. The effects of changes
in cross section, slab openings, and interaction with structural and nonstructural components
shall be considered.

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: An analytical model of the slab-column frame 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 connection, and the width
of the slab included in the model is adjusted to account for the flexibility of the slab-
column connection. The effective width shall be calculated in accordance with the
provisions of ACI 318.

2. An equivalent frame model in which the columns and slabs are represented by line
elements, and the stiffness of either the column or slab elements is adjusted to account for
the flexibility of the slab-column connection, 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 6.4.4.4.

C6.4.4.1 General Considerations

The stiffness of a slab – column frame is highly dependent on the ratio of the column cross
section dimensions ($c_1$ and $c_2$) to the slab plan dimensions ($l_1$ and $l_2$),

where: $c_1$ = column dimension parallel to span,

$c_2$ = column dimension perpendicular to span.
\[ l_1 = \text{center to center span length in the direction under consideration, and} \]
\[ l_2 = \text{center to center span length perpendicular to the direction under consideration.} \]

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

**Effective beam model.** An effective beam model (Pecknold, 1975) is one in which the width of the slab element is reduced to an effective width to adjust the elastic stiffness to more closely match measured values. Column behavior and slab-column moment and shear transfer are modeled separately.

**Equivalent frame model.** An equivalent frame model (Vanderbilt and Corley, 1983) is one in which shear and flexure in the slab beyond the width of the column are assumed to be transferred to the column through torsional elements perpendicular to the direction of the slab span. The flexibility of the torsional elements reduces the elastic stiffness of the overall frame. Torsional elements are lumped with the columns (typical) or the slab to produce a frame with equivalent stiffness, although it also is possible to model them separately. This approach is described in Chapter 13 of ACI 318.

**Finite element model.** A finite element model is one in which the distortion of the slab is modeled explicitly using finite elements.

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

6.4.4.2 Stiffness

6.4.4.2.1 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—Slab-column connections shall be modeled as either stiff or rigid components. The effective stiffnesses of components shall be determined according to the general principles of Section 6.3.1.2, but adjustments on the basis of experimental evidence shall be permitted.

6.4.4.2.1 Linear Static and Dynamic Procedures

**Effective beam model.** Guidance on determining effective slab width can be found in the literature. Allen and Darvall, 1977, provide tables of effective width coefficients for different combinations of plate aspect ratios \((l_1/l_2)\) and column width-to-slab span ratios \((c_1/l_1 \text{ or } c_2/l_1)\).
Research indicates that the effective width of exterior bays should be less than the effective width of interior bays due to the higher flexibility of one-sided slab-column connections at the end of a frame. Hwang and Moehle, 2000, provide equations for effective width that indicate the relationship between exterior and interior bays is about one-half. The following equations can be used in lieu of tables from Allen and Darvall, 1977:

For interior bays: \( b = 2c_1 + \frac{L_1}{3} \)

For exterior bays: \( b = c_1 + \frac{L_1}{6} \)

where:\n\( b = \) effective slab width\n\( c_1 = \) column dimension parallel to span\n\( L_1 = \) center to center span length in the direction under consideration

To account for cracking due to temperature, shrinkage, or nonlinear response, slab stiffness determined using gross section properties based on the above guidance should be reduced by an effective stiffness factor, \( \beta \). There is general agreement in the literature that \( \beta = 1/3 \) is appropriate for non-prestressed slabs (Vanderbilt and Corley, 1983). Somewhat higher, yet conservative, values can be obtained using the following equation from Hwang and Moehle, 2000:

\( \beta = 4\frac{c_1}{L_1} \geq 1/3 \)

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

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

### 6.4.4.2.2 Nonlinear Static Procedure

Nonlinear load-deformation relations shall comply with the requirements of Section 6.3.1.2.

Nonlinear static models. 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-Idealized load-deformation relations shall be modeled using according to the generalized relation shown in Figure 6-1, with definitions according to Section 6.4.2.2.2. The overall load-deformation relation shall be established so that the maximum resistance is consistent with the design strength specifications of Sections 6.3.2 and 6.4.4.3. Where the column, the generalized deformation shown in Figure 6-1 is shall be taken as the flexural plastic hinge rotation with parameters for the column, the plastic hinge rotation capacities shall be as defined by in Table 6-8. For slabs and slab-column connections, the generalized deformation shown in Figure 6-1 is shall be taken as the rotation of the slab-column connection, the plastic rotation with parameters capacities shall be as defined by in Table 6-14. Different relations shall be permitted where verified by experimentally obtained cyclic response relations of slab-column subassemblies.

C6.4.4.2.2 Nonlinear Static Procedure

The values provided in Table 6-14 are used to assess punching failures at slab – column connections. The information in Table 6-14 is based primarily on test data (Fig. C6-5) for interior connections summarized by Kang and Wallace (2006). Lateral drift ratio is typically reported for test data; therefore, plastic rotations were derived from the test data assuming column deformations were negligible and yield rotations of 0.01 and 0.015 radians for reinforced concrete and post-tensioned slabs, respectively. The larger rotation value for post-tensioned connections reflects the larger span-to-slab thickness ratios common for this type of construction. Continuity reinforcement for reinforced concrete connections is based on ACI –ASCE Committee 352 recommendations (Recommendations, 2002).

Plastic rotation values are approximately mean and mean minus one standard deviation values for connections with and without continuity reinforcement, respectively. Mean minus one standard deviation values give total (yield plus plastic) rotation values that are close to the maximum drift values allowed by ACI 318-05 S21.11.5 without the use of slab shear reinforcement. Few data exist for reinforced concrete connections subjected to gravity shear ratios greater than 0.6 and for post-tensioned connections subjected to reverse cyclic loading.

The residual strength capacity for post-tensioned connections is based on test results reported by Qaisrani, 1993. Although relatively few tests have been reported for edge connections, the limited data available suggest that the relationship between rotation and gravity shear ratio for exterior connections is similar to the trend for interior connections.

Modeling of slab – column connections is commonly accomplished using “beam” elements to represent the slab and a rigid-plastic “torsional” member to represent the connection between the slab and the column (moment and shear transfer), as shown in Figure C6-4. If the punching capacity of the slab – column connection is insufficient to develop the nominal capacity for the developed slab flexural reinforcement provided within the column strip, then all yielding is assumed to occur in the torsional element using the modeling parameters provided in Table 6-14. For cases where yielding of slab reinforcement within the column strip is expected (i.e., strong connection), plastic rotations should be modeled only within the beam elements framing into the torsional element (i.e., with plastic hinges with positive and negative nominal capacities) using the plastic rotation modeling parameters provided in Table 6-14 to define the plastic hinges at the beam ends.
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6.4.4.2.3 Nonlinear Dynamic Procedure

The requirements of Sections 6.3.2 and 6.4.2.2.3 for reinforced concrete beam-column moment frames shall apply to slab-column moment frames.

6.4.4.3 Strength

Component strengths shall be computed according to the general requirements of Sections 6.4.2, as modified in this section. For columns, evaluation of shear strength according to Section 6.4.2.3 shall be permitted.

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 6.4.2.3 shall be permitted.

Slab-column connections shall be investigated for potential failure in shear and moment transfer. 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 $\Sigma M_n$ = 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; $\gamma_f$ = 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 $0.5V_c$, 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.

**C6.4.4.3 Strength**

Alternative expressions for calculating moment transfer strength of interior and exterior slab-
column connections can be found in Luo, et al., 1994, and detailed modeling recommendations
for reinforced and post-tensioned concrete slab – column frames as well as comparisons with
shake table tests can be found in Kang et al., 2006.

**6.4.4 Acceptance Criteria**

**6.4.4.1 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. $m$-
factors for slab-column frame components shall be selected from 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 for columns at a level exceeds the average value for slabs at the
same level, and exceeds the greater of 1.0 and $m/2$, the element shall be defined as a weak story
element and shall be evaluated by the procedure for weak story elements described in Section
6.4.2.4.1.

**6.4.4.2 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 actions are acceptable for the selected performance levels.
Other actions shall be defined as force-controlled.

Calculated component actions shall satisfy the requirements of Chapter 3 Section 3.4.3.2.
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.

C6.4.4.4.2 Nonlinear Static and Dynamic Procedures

Refer to Section C6.4.4.2 for discussion of Table 6-14 and acceptance criteria for reinforced concrete slab-column connections. Refer to Section C6.4.2.2.2 for discussion of Table 6-8 and acceptance criteria for reinforced concrete columns.

6.4.4.5 Rehabilitation Measures

Reinforced concrete slab-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 6.3.7 and other provisions of this standard.

C6.4.4.5 Rehabilitation Measures

The rehabilitation measures described in C6.5.4.2.5 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 [Refer to end of chapter]

Table 6-15 Numerical Acceptance Criteria for Linear Procedures-Two-way Slabs and Slab-Column Connections [Refer to end of chapter]

6.5 Precast Concrete Frames

6.5.1 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.

6.5.1.1 Precast Concrete Frames Expected to Resist Lateral Load
Frames of this classification shall be assembled using either reinforcement and wet concrete or 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.

C6.5.1.1 Precast Concrete Frames Expected to Resist Lateral Load

These systems are recognized and accepted by FEMA 450, and are based on ACI 318, which specifies safety and serviceability levels expected from precast concrete frame construction. In the referenced documents precast frames are classified not by the method of construction (wet or dry joints) but by the expected behavior resulting from the detailing used. In addition to recognizing varying levels of ductile performance as a result of overall frame detailing, ACI 318 (in Section 21.6) acknowledges three types of unit-to-unit connections that can result in the highest level of performance. Such connections are either “strong” or “ductile” as defined in Sections 21.1 and 21.6 of ACI 318, or have demonstrated acceptable performance where tested in accordance with ACI T1.1-01.

6.5.1.2 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.

6.5.2 Precast Concrete Frames Expected to Resist Lateral Load

6.5.2.1 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 components, including nonstructural components, shall be included. All other considerations of Section 6.4.2.1 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. Where dry joints are used in assembling the precast system, consideration shall be given to the effect of those joints on overall behavior. Where connections yield under design lateral loads, the analysis model shall take this into account.

6.5.2.2 Stiffness

Stiffness for analysis shall be as defined in Section 6.4.2.2. The effects of prestressing shall be considered where computing the effective stiffness values using Table 6-5. Flexibilities associated with connections shall be included in the analytical model.
6.5.2.3  **Strength**

Component strength shall be computed according to the requirements of Section 6.4.2.3, 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.

Effects of connection strength shall be considered in accordance with Section 6.3.6.

6.5.2.4  **Acceptance Criteria**

Acceptance criteria for precast concrete frames expected to resist lateral load shall be as specified in Section 6.4.2.4, except that the factors defined in Section 6.4.2.3 shall also be considered. Connections shall comply with the requirements of Section 6.3.6.

6.5.2.5  **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 6.3.7 and other provisions of this standard.

C6.5.2.5  **Rehabilitation Measures**

The rehabilitation measures described in C6.5.4.2.5 for reinforced concrete beam-column moment frames may also be effective in rehabilitating precast concrete moment frames. When installing new components or materials to the existing system, existing prestressing strands should be protected.

6.5.3  **Precast Concrete Frames Not Expected to Resist Lateral Loads Directly**

6.5.3.1  **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 6.5.2.1 and shall include the effects of deformations that are calculated to occur under the design earthquake loadings.

6.5.3.2  **Stiffness**
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The analytical model shall include either 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.

C6.5.3.2 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.

6.5.3.3 Strength

Component strength shall be computed according to the requirements of Section 6.5.2.3. 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.

6.5.3.4 Acceptance Criteria

Acceptance criteria for components in precast concrete frames not expected to resist lateral loads directly shall be as specified in Section 6.5.3.4. All moments, shear forces, and axial loads induced through the deformation of the structural system shall be checked using appropriate criteria in the referenced section.

6.5.3.5 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 6.3.7 and other provisions of this standard.

C6.5.3.5 Rehabilitation Measures

The rehabilitation measures described in C6.4.2.5 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 components or materials to the existing system, existing prestressing strands should be protected.

6.6 Concrete Frames with Infills

6.6.1 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 6.6 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.

6.6.1 Types of Frames

The provisions of Section 6.6 shall apply to concrete frames as defined in Sections 6.4, 6.5, and 6.9, where those frames interact with infills.

6.6.2 Masonry Infills

The provisions of Section 6.6 shall apply to masonry infills as defined in Chapter 7, where those infills interact with concrete frames.

6.6.3 Concrete Infills

The provisions of Section 6.6 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.

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.

6.6.2 Concrete Frames with Masonry Infills

6.6.2.1 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 nonstructural 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.

### C6.6.2.1 General Considerations

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

### 6.6.2.2 Stiffness

#### 6.6.2.2.1 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 6.6.2.1, and the noninfilled bays shall be modeled as frames as specified in appropriate portions of Sections 6.4, 6.5, and 6.9. 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 6.3.1.2.

#### 6.6.2.2.2 Nonlinear Static Procedure

Nonlinear load-deformation relations for use in analysis by Nonlinear Static Procedure (NSP) shall follow the requirements of Section 6.3.1.2.2.

Modeling beams and columns using nonlinear truss elements shall be permitted in infilled portions of the frame. Beams and columns in non-infilled portions of the frame shall be modeled using the relevant specifications of Sections 6.4, 6.5, and 6.9. The model shall be capable of representing inelastic response along the component lengths.
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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 non-infilled 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.

6.6.2.2.3 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.

6.6.2.3 Strength

Strengths of reinforced concrete components shall be calculated according to the general requirements of Sections 6.3.2, 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:

4.1. Limitations imposed by beams, columns, and joints in non-infilled portions of frames.

5.2. Tensile and compressive capacity of columns acting as boundary components of infilled frames.

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

7.4. Strength of the infill.

8.5. Connections with adjacent components.
6.6.2.4 Acceptance Criteria

6.6.2.4.1 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 6.4, 6.5, and 6.9, as appropriate, and for the masonry infill in Section 7.4.

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 7.4.2.3 for masonry infills; applicable portions of Sections 6.4, 6.5, and 6.9 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.

### Table 6-17 Numerical Acceptance Criteria for Linear Procedures-Reinforced Concrete Infilled Frames [Refer to end of chapter]

6.6.2.4.2 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 6.4, 6.5, and 6.9, as well as for masonry infills as specified in Section 7.4.

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 6.4, 6.5, and 6.9, 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.

6.6.2.5 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 6.3.7 and other provisions of this standard.

C6.6.2.5 Rehabilitation Measures

The rehabilitation measures described in relevant commentary of Sections 6.4, 6.5, and 6.9 for isolated frames, and rehabilitation measures described in relevant commentary or Section 7.4 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.

6.6.3 Concrete Frames with Concrete Infills

6.6.3.1 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 nonstructural 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 6.6.2.
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 6.4, 6.5, and 6.9. Where infills create a discontinuous wall, the effects of the discontinuity on overall building performance shall be evaluated.

6.6.3.2 Stiffness

6.6.3.2.1 Linear Static and Dynamic Procedures

Effective stiffnesses shall be calculated according to the principles of Section 6.3.1.2 and the procedure of Section 6.6.2.2.1.

6.6.3.2.2 Nonlinear Static Procedure

Nonlinear load-deformation relations for use in analysis by Nonlinear Static Procedure (NSP) shall follow the requirements of Section 6.3.1.2.2. 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 6.6.2.2.2 shall be permitted for the development of nonlinear modeling parameters for concrete frames with concrete infills.

6.6.3.2.3 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.

6.6.3.3 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:

7.1. Limitations imposed by beams, columns, and joints in unfilled portions of frames.
§6.2. Tensile and compressive capacity of columns acting as boundary components of infilled frames.

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

4. Strength of the infill.

5. Connections with adjacent components.

Strengths of existing concrete infills shall be determined considering shear strength of the infill panel. For this calculation, procedures specified in Section 6.7.2.3 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 components, and (2) the infill wall, including anchorage of the infill reinforcement in the boundary frame.

6.6.3.4 Acceptance Criteria

The acceptance criteria for concrete frames with concrete infills shall comply with relevant acceptance criteria of Sections 6.6.2.4, 6.7, and 6.8.

6.6.3.5 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 6.3.7 and other provisions of this standard.

C6.6.3.5 Rehabilitation Measures

Rehabilitation measures described in C6.6.2.5 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.

6.7 Concrete Shear Walls

6.7.1 Types of Concrete Shear Walls and Associated Components

The provisions of Section 6.7 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 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 wall pier and deep beam components referred to as wall segments.

Coupling beams and columns that support discontinuous shear walls shall comply with provisions of Section 6.7.2 and. These special frame components associated with shear walls shall be exempted from the provisions for beams and columns of frame components covered in Section 6.4.

C6.7.1 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 >3.0, and shall be considered short or squat if their aspect ratio is <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 wall 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 C6-5 to clarify wall component identification.

6.7.1.1 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 6.6.1.3.

Shear walls or wall segments with axial loads greater than 0.35 $P_o$ shall not be considered effective in resisting seismic forces. For the purpose of determining effectiveness of shear walls or wall segments, the use of axial loads based on a limit state analysis shall be permitted. 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 6.7.2.3.
C6.7.1.1 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.

6.7.1.2 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 6.7.2.

C6.7.1.3 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 6.7.2.
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.

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

6.7.2.1 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 components 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.

C6.7.2.1 General Considerations

For rectangular shear walls and wall segments with $h/l_w \leq 2.5$, and flanged wall sections with $h/l_w \leq 3.5$, 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 450 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.

6.7.2.2 Stiffness

The effective stiffness of all the elements discussed in Section 6.7 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.

6.7.2.2.1 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 6.3.1.3. The calculated stiffnesses to be used in analysis shall be in accordance with the requirements of Section 6.3.1.2.

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

6.7.2.2.2 Nonlinear Static Procedure

Nonlinear load-deformation relations for use in analysis by nonlinear static and dynamic procedures shall comply with the requirements of Section 6.3.1.2.

Monotonic load-deformation relationships for analytical models that represent shear walls, wall elements, and 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
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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, $\theta_y$, and shall be calculated in accordance with Equation (6-6):

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

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 [Refer to end of chapter]

Figure 6-3 Story Drift in Shear Wall where Shear Dominates Inelastic Response [Refer to end of chapter]

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 [Refer to end of chapter]

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 ratio. Alternatively, the load-deformation relationship in Figure 6-1(b) shall be permitted, with the x-axis of Figure 6-1(b) taken as lateral drift ratio. 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 $d$, $e$, $f$, $g$ and $c$ required to find the points $B$, $C$, $D$, $E$, and $F$ in Figure 6-1(b) or Figure 6-1(c), 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.

### C6.7.2.2 Nonlinear Static Procedure

The recommended backbone shape and parameters provided for concrete shear walls differs from the general backbone description in Chapter 2. For walls with shear span-to-depth ratios below 2.5, the load-deformation relationship in Figure 6-1 (c) provides a better representation of the behavior than that in Figure 6-1 (b). The reason is that in walls with low shear-span-to-depth ratios the deformations related to shear are not negligible compared with the deformations related to flexure. The proposed relationship is based on a model in which the total deflection is calculated as the sum of contributions of components related to flexure, shear, and slip of the reinforcement. The drift ratio and shear force corresponding to inclined cracking in Figure 6-1 (c) were obtained by simplifying expressions for principal stresses for a limiting concrete tensile strength of approximately $4\sqrt{f', c}$ (Sozen and Moehle, 1993). Definition of the yield point and the lateral strength degradation point are based on limited test data (e.g., Hidalgo, 2002), as summarized by Wallace (EERI notes, 2006). Note that variables $F$, $g$, and $f$ in Figure 6-1(c) are not the same as those used in Chapter 2.

### 6.7.2.3 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.

### 6.7.2.3 Strength

Component strengths shall be computed according to the general requirements of Sections 6.3.2, 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 6.7.2.2 shall be used, except that the first limit shall be changed to one-tenth of the wall height. Where 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. Where 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
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, except that the restriction on the number of curtains of reinforcement shall not apply to existing walls. 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.

Where a shear wall or wall segment has a transverse reinforcement percentage, $\rho_n$, less than the minimum value of 0.0025 but greater than 0.0015 and reinforcement is spaced no greater than 18 inches, 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 6.3.5. 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 6.4.2.3.

### C6.7.2.3 Strength

Data presented by Wood (1990) indicate that wall strength is insensitive to the quantity of transverse reinforcement where 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.

Strength calculations based on ACI 318, excluding Chapter 22, assume a maximum spacing of wall reinforcement. No data is available to justify performance for walls that do not meet the maximum spacing requirements. If plain concrete is encountered in an existing building, Chapter 22 of ACI 318 can be used to derive capacities, while Section 2.8 of this standard can be used to develop acceptance criteria.
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Chapter of the ACI 318 Code requires at least two curtains of reinforcement be used in a wall if 
\[ V_u \text{ exceeds } 2A_v \sqrt{f'c} \]. Experimental results by Hidalgo et al. (2002) show that for relatively thin 
walls there is no significant difference between the strength of walls with one or two curtains of 
web reinforcement.

6.7.2.4 Acceptance Criteria

6.7.2.4.1 Linear Static and Dynamic Procedures

Shear walls, wall segments, and 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 these other components, deformation-controlled actions shall be 
restricted to flexure or shear. All other actions shall be treated 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, and wall segments, and columns supporting discontinuous 
and 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, axial, or force transfer at rebar anchorages and splices) on 
components shall be determined as prescribed in Chapter 3. Where 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.

6.7.2.4.1 Linear Static and Dynamic Procedures

For shear-controlled coupling beams, ductility is a function of the shear in the member as 
determined by the expected shear capacity of the member. In accordance with Section 6.3.2, 
expected strengths are calculated using the procedures specified in ACI 318. For coupling 
beams, \( V'_c \) is nearly always zero.

6.7.2.4.2 Nonlinear Static and Dynamic Procedures

In the design model, inelastic response shall be restricted to those components and actions listed 
in Tables 6-18 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>
<thead>
<tr>
<th>Table 6-18</th>
<th>Modeling Parameters and Numerical Acceptance</th>
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<tbody>
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<td>Criteria for Nonlinear Procedures-Members Controlled by Flexure</td>
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<td>[Refer to end of chapter]</td>
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<table>
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<th>Table 6-19</th>
<th>Modeling Parameters and Numerical Acceptance</th>
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<tr>
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<td>Criteria for Nonlinear Procedures-Members Controlled by Shear</td>
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<td>[Refer to end of chapter]</td>
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<th>Table 6-20</th>
<th>Numerical Acceptance Criteria for Linear Procedures-Members Controlled by Flexure</th>
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<tr>
<th>Table 6-21</th>
<th>Numerical Acceptance Criteria for Linear Procedures-Members Controlled by Shear</th>
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<td>[Refer to end of chapter]</td>
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**6.7.2.5 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 6.3.7 and other
provisions of this standard.

**C6.7.2.5 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 components.** Addition of boundary components 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
components or steel sections. In both cases, proper connections should be made between
the existing wall and the added components. 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 steel 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 6.4 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.

6.8 **Precast Concrete Shear Walls**

6.8.1 **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 6.8:

1. Effectively monolithic construction, 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.

6.8.1.1 Effectively Monolithic Construction

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. Precast shear walls and wall segments of effectively monolithic construction shall be evaluated by the criteria defined in Section 6.7.

6.8.1.2 Jointed Construction

Precast shear walls and wall segments of jointed construction shall be evaluated by the criteria defined in Section 6.8.2.

6.8.1.3 Tilt-up 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, where 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 6.7 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 6.7 should be reduced.

Section 9.6 of FEMA 450 provides testing criteria that may be used to validate design values consistent with the highest performance of monolithic shear wall construction.

### 6.8.1.3 Tilt-up Construction

Shear walls and wall segments of tilt-up type of precast walls shall be evaluated by the criteria defined in Section 6.8.2.

### C6.8.1.3 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 analyzed carefully to be sure the diaphragm forces can be transmitted safely to the precast wall system.

### 6.8.2 Precast Concrete Shear Walls and Wall Segments

#### 6.8.2.1 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 components 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 flexural 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.

6.8.2.2 Stiffness

The modeling assumptions defined in Section 6.7.2.2 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.

6.8.2.2.1 Linear Static and Dynamic Procedures

The modeling procedures given in Section 6.7.2.2.1, combined with a procedure for including connection deformations as noted above, shall be used.

6.8.2.2.2 Nonlinear Static Procedure

Nonlinear load-deformation relations shall comply with the requirements of Section 6.3.1.2. 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 6.7.2.3. 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 effectively monolithic construction are satisfied, the value of the hinge rotation at point $B$ shall correspond to the yield rotation, $\theta_y$, 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 $\theta_y$ 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 effectively monolithic construction, 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 by the authority having jurisdiction. In no case, however, shall values larger than those specified in Table 6-18 be used.

For effectively monolithic construction, 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 by the authority having jurisdiction. 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.

6.8.2.2.3 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.

6.8.2.3 Strength

The strength of precast concrete shear walls and wall segments within the panels shall be computed according to the general requirement of Section 6.3.2, except as modified here. For effectively monolithic construction, the strength calculation procedures given in Section 6.7.2.3 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 where calculating the axial and flexural strength of the connection region. The unmodified specified yield strength of the reinforcement and connection hardware shall be used where 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 6.7.2.3.
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 often can govern the strength of the precast shear wall system.

6.8.2.4 Acceptance Criteria

6.8.2.4.1 Linear Static and Dynamic Procedures

For precast shear wall construction that is effectively monolithic and for wall segments within a precast panel, the acceptance criteria defined in Section 6.7.2.4.1 shall be followed. For precast shear wall construction defined as jointed construction, the acceptance criteria procedure given in Section 6.7.2.4.1 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. An $m$ value need not be taken as less than 1.0, and in no case shall be taken larger than the values specified in these tables.

6.8.2.4.2 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 are effectively monolithic 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.

C6.8.2.4 Acceptance Criteria

The procedures outlined in Section 9.6 of FEMA 450 may be used to establish acceptance criteria for precast shear walls.

6.8.2.5 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 6.3.7 and other provisions of this standard.

### C6.8.2.5 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 6.7.2.5 may 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 where strengthening these connections.

### 6.9 Concrete-Braced Frames

#### 6.9.1 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 6.6 shall also apply.

The provisions of Section 6.9 shall apply to existing reinforced concrete-braced frames and
existing reinforced concrete-braced frames rehabilitated by addition or removal of material.

### 6.9.2 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 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 6.4.2.1.

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 6.9.5.

### 6.9.3 Stiffness

#### 6.9.3.1 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 6.3.1.2.

#### 6.9.3.2 Nonlinear Static Procedure

Nonlinear load-deformation relations shall comply with the requirements of Section 6.3.1.2.
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 6.4.2.2.2. 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 6.3.2 and 6.4.2.3. Numerical quantities in Figure 6-1 shall be derived from tests, rational analyses, or criteria of Section 6.6.2.2.2, with braces modeled as columns in accordance with Table 6-16.

6.9.3.3 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.

6.9.4 Strength

Component strengths shall be computed according to the general requirements of Sections 6.3.2 and the additional requirements of Section 6.4.2.3. The possibility of instability of braces in compression shall be considered.

6.9.5 Acceptance Criteria

6.9.5.1 Linear Static and Dynamic Procedure

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.

6.9.5.2 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.

6.9.6 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 6.3.7 and other provisions of this standard.

C6.9.6 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.

6.10 Cast-in-Place Concrete Diaphragms

6.10.1 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.

6.10.1.1 Slabs

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

6.10.1.2 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.

6.10.1.3 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.

C6.10.1.3 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.

6.10.2 Analysis, Modeling, and Acceptance Criteria

6.10.2.1 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.

C6.10.2.1 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.

6.10.2.2 Stiffness

Diaphragm stiffness shall be modeled according to Section 6.10.2.1 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. Where 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 where assigning lateral forces to the resisting vertical elements.

C6.10.2.2 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.

6.10.2.3 Strength
Strength of cast-in-place concrete diaphragm components shall comply with the requirements of Sections 6.3.2 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 6.4.2.3.

6.10.2.4 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 6.7.2.4, with $\mu$ 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 6.4.2.4. Connections shall be considered force-controlled.

6.10.3 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 6.3.7 and other provisions of this standard.

C6.10.3 Rehabilitation Measures

Cast-in-place concrete diaphragms can have a wide variety of deficiencies; see Chapter 10 and ASCE 31.

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.

6.11 Precast Concrete Diaphragms

6.11.1 Components of Precast Concrete Diaphragms

Precast concrete diaphragms are elements comprising primarily 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.

C6.11.1 Components of Precast Concrete Diaphragms

Section 6.10 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 components, 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.

6.11.2 Analysis, Modeling, and Acceptance Criteria

Analysis and modeling of precast concrete diaphragms shall conform to Section 6.10.2.2, 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 6.10.2.3. 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.

C6.11.2 Analysis, Modeling, and Acceptance Criteria

Welded connection strength can be determined using the latest version of the Precast Concrete Institute (PCI) Handbook. A discussion of design provisions for untopped precast diaphragms can be found in the Appendix to Chapter 9 of FEMA 368.
The appendix to Chapter 9 of FEMA 450 provides discussion of the behavior of untopped precast diaphragms and outlines a design approach that may be used for such diaphragms to satisfy the requirements of this standard.

6.11.3 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 6.3.7 and other provisions of this standard.

C6.11.3 Rehabilitation Measures

Section 6.10.3 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.

6.12 Concrete Foundation Components

6.12.1 Types of Concrete Foundations

Foundations shall be defined as those components 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 6.12 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 6.12.

The provisions of Section 6.12 shall apply to existing foundation components and to new materials or components that are required to rehabilitate an existing building.

6.12.1.1 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.

6.12.1.2 Deep Foundations
6.12.1.2 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 piles by the pile cap. Pile foundation resistance to vertical loads shall be calculated based on the direct bearing of the pile tip in the soil, the skin friction or cohesion of the soil on the surface area of the pile, or based on a combination of these mechanisms. Lateral loads resistance shall be calculated based on 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.

6.12.1.2.2 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 resistance to vertical and lateral loads shall be calculated in the same manner as that of driven pile foundations specified in Section 6.12.1.2.1.

C6.12.1.2 Deep Foundations

C6.12.1.2.1 Driven Pile Foundations

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

C6.12.1.2.2 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.

6.12.2 Analysis of Existing Foundations

For concrete buildings, components shall be considered fixed against rotation at the top of the foundation if the connections between components and foundations, the foundations and supporting soil are shown to be capable of resisting the induced moments. Where components are not designed to resist flexural moments, or the connections between components and foundations are not capable of resisting the induced 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.

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

C6.12.2 Analysis of Existing Foundations

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

6.12.3 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 systems and all new material, components, or components required for rehabilitation shall be evaluated as force-controlled actions. 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).

6.12.4 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 6.3.7 and other provisions of this standard.

C6.12.4 Rehabilitation Measures

Rehabilitation measures described in Section C6.12.4.1 for shallow foundations and in Section C6.12.4.2 for deep foundations may be effective in rehabilitating existing foundations.

C6.12.4.1 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 tie-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 tie-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 where 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.

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C6.12.4.2 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 C6.12.4.1.

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 where liquefiable soils were present. This is especially important to consider around wharf structures and in areas having a high water table. Addition of batter piles to foundations in areas of such seismic hazards should be in accordance with requirements in Section 4.4.

7. **Increasing tension tie capacity from pile or pier to superstructure.** Added reinforcement should satisfy the requirements of Section 6.3.
Figure 6-1  Generalized Force-Deformation Relations for Concrete Elements or Components

(a) Deformation

(b) Deformation ratio

(c) Tri-linear response - Deformation ratio
Figure C6-1  Beam-Column Joint Modeling (hatched portions are rigid)

a) Example of explicit joint model
b) Offsets for implicit joint model

c) $\Sigma M_{nc}/\Sigma M_{nb} > 1.2$
d) $\Sigma M_{nc}/\Sigma M_{nb} < 0.8$
e) $0.8 \leq \Sigma M_{nc}/\Sigma M_{nb} \leq 1.2$
Figure C6-2  Fragility curve

\[ P_f \]

\[ \theta_p / \theta_p^{table} \]
Figure C6-3 Joint Classification (for response in the plane of the page)

a) Interior joint with transverse beams
b) Interior joint without transverse beams

c) Exterior joint with transverse beams
d) Exterior joint without transverse beams
e) Knee joint with or without transverse beams
Figure C6-4 Modeling of slab-column connection

Elastic column

Column plastic hinge

Torsional connection element

Joint region

Slab-beam plastic hinge

Elastic slab beam

Plastic hinges for slab beams or for torsional element

Elastic relation for slab beam or column

1Slab-beams and columns only connected by rigid-plastic torsional connection element.
Figure C6-15  Identification of Component Types in Concrete Shear Wall Elements (from FEMA 306)

(a) Cantilever Wall Mechanisms

(b) Pier / Spandrel Mechanisms

(c) Mixed Mechanisms
Figure 6-2  Plastic Hinge Rotation in Shear Wall where Flexure Dominates Inelastic Response

Plastic Hinge Rotation = $\theta$

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

$\Delta$ $L$
Figure 6-4  Chord Rotation for Shear Wall Coupling Beams

Chord Rotation:
\[ \theta = \frac{\Delta}{L} \]
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<th>Structural²</th>
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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.
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<th>Component Type per FEMA 306</th>
<th>Description</th>
<th>ASCE 41 Designation</th>
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<td>RC1</td>
<td>Isolated Wall or Stronger Wall Pier Stronger than beam or spandrel components 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</td>
<td>Monolithic reinforced concrete wall or vertical wall segment</td>
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<td>RC2</td>
<td>Weaker Wall Pier Weaker than the spandrels to which it connects; characterized by flexural hinging top and bottom, or shear failure, etc.</td>
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<td>RC3</td>
<td>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.</td>
<td>Horizontal wall segment or coupling beam</td>
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<td>RC4</td>
<td>Stronger Spandrel Should not suffer damage because it is stronger than attached wall piers. If this component is damaged, it should probably be re-classified as RC3.</td>
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<td>RC5</td>
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<td>Wall segment</td>
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</tr>
<tr>
<td>A616³</td>
<td>Rail³</td>
<td>1968-Present</td>
</tr>
<tr>
<td>A617</td>
<td>Axle</td>
<td>1968-Present</td>
</tr>
<tr>
<td>A706</td>
<td>Low-Alloy</td>
<td>1974-Present</td>
</tr>
<tr>
<td>A955</td>
<td>Stainless</td>
<td>1996-Present</td>
</tr>
</tbody>
</table>

**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."
6. ASTM A706 has a minimum tensile strength of 80 ksi, but not less than 1.25 times the actual yield strength.
<table>
<thead>
<tr>
<th>Time Frame</th>
<th>Footings</th>
<th>Beams</th>
<th>Slabs</th>
<th>Columns</th>
<th>Walls</th>
</tr>
</thead>
<tbody>
<tr>
<td>1900-1919</td>
<td>1000-2500</td>
<td>2000-3000</td>
<td>1500-3000</td>
<td>1500-3000</td>
<td>1000-2500</td>
</tr>
<tr>
<td>1950-1969</td>
<td>2500-3000</td>
<td>3000-4000</td>
<td>3000-4000</td>
<td>3000-6000</td>
<td>2500-4000</td>
</tr>
<tr>
<td>1970-Present</td>
<td>3000-4000</td>
<td>3000-5000</td>
<td>3000-5000</td>
<td>3000-10000</td>
<td>3000-5000</td>
</tr>
<tr>
<td>Material Property</td>
<td>Factor</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----------------------------------</td>
<td>--------</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete Compressive Strength</td>
<td>1.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reinforcing Steel Tensile &amp; Yield Strength</td>
<td>1.25</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Connector Steel Yield Strength</td>
<td>1.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Component</td>
<td>Flexural Rigidity</td>
<td>Shear Rigidity</td>
<td>Axial Rigidity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>------------------------------------------------</td>
<td>-------------------</td>
<td>----------------</td>
<td>---------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beams-nonprestressed</td>
<td>$0.30EJ_g$</td>
<td>$0.4E_Aw$</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beams-prestressed</td>
<td>$EJ_g$</td>
<td>$0.4E_Aw$</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Columns with compression due to design gravity loads $\geq 0.5 A_g'f_c$</td>
<td>$0.7E_J_g$</td>
<td>$0.4E_Aw$</td>
<td>$E_Ag$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Columns with compression due to design gravity loads $\leq 0.3 - 0.1 A_g'f_c$ or with tension</td>
<td>$0.30-0.5E_J_g$</td>
<td>$0.4E_Aw$</td>
<td>$E_Ag$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam-column joints</td>
<td>See Section 6.4.2.2.1</td>
<td>$E_Ag$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Walls-uncracked† (on inspection)</td>
<td>$0.8E_J_g$</td>
<td>$0.4E_Aw$</td>
<td>$E_Ag$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Walls-cracked†</td>
<td>$0.5E_J_g$</td>
<td>$0.4E_Aw$</td>
<td>$E_Ag$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat Slabs-nonprestressed</td>
<td>See Section 6.5.4.2</td>
<td>$0.4E_Ag$</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat Slabs-prestressed</td>
<td>See Section 6.5.4.2</td>
<td>$0.4E_Ag$</td>
<td>-</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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 6.3.1.3. For columns with axial compression falling between the limits provided, linear interpolation shall be permitted. Alternatively, if interpolation is not performed, the more conservative effective stiffnesses shall be used.

† See Section 6.7.2.2
<table>
<thead>
<tr>
<th>Maximum value of DCR or displacement ductility</th>
<th>Descriptor</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 2</td>
<td>Low Ductility Demand</td>
</tr>
<tr>
<td>2 to 4</td>
<td>Moderate Ductility Demand</td>
</tr>
<tr>
<td>&gt; 4</td>
<td>High Ductility Demand</td>
</tr>
</tbody>
</table>
Table 6-7 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Plastic Rotations Angle, radians</th>
<th>Residual Strength Ratio</th>
<th>Acceptance Criteria&lt;sup&gt;3,4&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a</td>
<td>b</td>
<td>c</td>
</tr>
<tr>
<td>i. Beams controlled by flexure&lt;sup&gt;4&lt;/sup&gt;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\rho - \rho'$</td>
<td>$\rho_{ul}$</td>
<td>Trans. Reinf.&lt;sup&gt;2&lt;/sup&gt;</td>
<td>$\frac{V}{b_d f'_c}$</td>
</tr>
<tr>
<td>$\leq 0.0$</td>
<td>C</td>
<td>$\leq 3$</td>
<td>0.025</td>
</tr>
<tr>
<td>$\leq 0.0$</td>
<td>C</td>
<td>$\geq 6$</td>
<td>0.02</td>
</tr>
<tr>
<td>$\geq 0.5$</td>
<td>C</td>
<td>$\leq 3$</td>
<td>0.02</td>
</tr>
<tr>
<td>$\geq 0.5$</td>
<td>C</td>
<td>$\geq 6$</td>
<td>0.015</td>
</tr>
<tr>
<td>$\leq 0.0$</td>
<td>NC</td>
<td>$\leq 3$</td>
<td>0.02</td>
</tr>
<tr>
<td>$\leq 0.0$</td>
<td>NC</td>
<td>$\geq 6$</td>
<td>0.01</td>
</tr>
<tr>
<td>$\geq 0.5$</td>
<td>NC</td>
<td>$\leq 3$</td>
<td>0.01</td>
</tr>
<tr>
<td>$\geq 0.5$</td>
<td>NC</td>
<td>$\geq 6$</td>
<td>0.005</td>
</tr>
<tr>
<td>ii. Beams controlled by shear&lt;sup&gt;4&lt;/sup&gt;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stirrup spacing $\leq d/2$</td>
<td>0.0030</td>
<td>0.02</td>
<td>0.2</td>
</tr>
<tr>
<td>Stirrup spacing $&gt; d/2$</td>
<td>0.0030</td>
<td>0.01</td>
<td>0.2</td>
</tr>
<tr>
<td>iii. Beams controlled by inadequate development or splicing along the span&lt;sup&gt;4&lt;/sup&gt;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stirrup spacing $\leq d/2$</td>
<td>0.0030</td>
<td>0.02</td>
<td>0.0</td>
</tr>
<tr>
<td>Stirrup spacing $&gt; d/2$</td>
<td>0.0030</td>
<td>0.01</td>
<td>0.0</td>
</tr>
<tr>
<td>iv. Beams controlled by inadequate embedment into beam-column joint&lt;sup&gt;4&lt;/sup&gt;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.015</td>
<td>0.03</td>
<td>0.2</td>
</tr>
</tbody>
</table>

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

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 $\leq d/3$, 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.

4. Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.
<table>
<thead>
<tr>
<th>Conditions</th>
<th>Modeling Parameters(^4)</th>
<th>Acceptance Criteria(^4,6)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plastic Rotations Angle, radians</td>
<td>Residual Strength Ratio</td>
</tr>
<tr>
<td></td>
<td>a</td>
<td>b</td>
</tr>
<tr>
<td>i. Columns controlled by flexure(^4)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( P / F_y )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( A_{c} / F_y )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( w_{c} / d_{c} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( V_{c} / f_{y} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \leq 0.1 )</td>
<td>C</td>
<td>( \leq 3 )</td>
</tr>
<tr>
<td>( \leq 0.1 )</td>
<td>C</td>
<td>( \geq 6 )</td>
</tr>
<tr>
<td>( \geq 0.4 )</td>
<td>C</td>
<td>( \leq 3 )</td>
</tr>
<tr>
<td>( \geq 0.4 )</td>
<td>C</td>
<td>( \geq 6 )</td>
</tr>
<tr>
<td>( \leq 0.1 )</td>
<td>NC</td>
<td>( \leq 3 )</td>
</tr>
<tr>
<td>( \leq 0.1 )</td>
<td>NC</td>
<td>( \geq 6 )</td>
</tr>
<tr>
<td>( \geq 0.4 )</td>
<td>NC</td>
<td>( \leq 3 )</td>
</tr>
<tr>
<td>( \geq 0.4 )</td>
<td>NC</td>
<td>( \geq 6 )</td>
</tr>
<tr>
<td>ii. Columns controlled by shear(^1,3)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>All cases</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( P / F_y )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( A_{c} / F_y )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( w_{c} / d_{c} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( V_{c} / f_{y} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hoop spacing ( \leq d / 2 )</td>
<td>0.01</td>
<td>0.02</td>
</tr>
<tr>
<td>Hoop spacing ( &gt; d / 2 )</td>
<td>0.0</td>
<td>0.01</td>
</tr>
<tr>
<td>iii. Columns controlled by inadequate development or splicing along the clear height(^1,3)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conforming hoops over the entire length</td>
<td>0.015</td>
<td>0.025</td>
</tr>
<tr>
<td>All other cases</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

1. Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
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 \( \leq d / 3 \), 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 6.4.2.4.2 for primary component acceptance criteria.
6. Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.
Table 6-8 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Columns

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Plastic Rotations Angle, radians</th>
<th>Residual Strength Ratio</th>
<th>Acceptance Criteria</th>
<th>Plastic Rotations Angle, radians</th>
<th>Residual Strength Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>b</td>
<td>c</td>
<td>IO</td>
<td>Primary</td>
<td>Secondary</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Condition i</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$P / f'_c$</td>
<td>$\rho = \frac{A_g}{b_h s}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>≥ 0.006</td>
<td>0.035</td>
<td>0.060</td>
<td>0.2</td>
<td>0.005</td>
</tr>
<tr>
<td>≥ 0.6</td>
<td>≥ 0.006</td>
<td>0.010</td>
<td>0.010</td>
<td>0.0</td>
<td>0.003</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>= 0.002</td>
<td>0.027</td>
<td>0.034</td>
<td>0.2</td>
<td>0.005</td>
</tr>
<tr>
<td>≥ 0.6</td>
<td>= 0.002</td>
<td>0.005</td>
<td>0.005</td>
<td>0.0</td>
<td>0.002</td>
</tr>
<tr>
<td>Condition ii</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$P / f'_c$</td>
<td>$\rho = \frac{A_g}{b_h s}$</td>
<td>$v = \frac{b_d f'_c}{f'_c}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>≥ 0.006</td>
<td>0.032</td>
<td>0.060</td>
<td>0.2</td>
<td>0.005</td>
</tr>
<tr>
<td>≥ 0.1</td>
<td>≥ 0.006</td>
<td>0.025</td>
<td>0.060</td>
<td>0.2</td>
<td>0.005</td>
</tr>
<tr>
<td>≥ 0.6</td>
<td>≥ 0.006</td>
<td>0.010</td>
<td>0.010</td>
<td>0.2</td>
<td>0.003</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>≤ 0.0005</td>
<td>0.008</td>
<td>0.008</td>
<td>0.2</td>
<td>0.003</td>
</tr>
<tr>
<td>≥ 0.1</td>
<td>≤ 0.0005</td>
<td>0.012</td>
<td>0.012</td>
<td>0.0</td>
<td>0.005</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>≤ 0.0005</td>
<td>0.006</td>
<td>0.006</td>
<td>0.0</td>
<td>0.004</td>
</tr>
<tr>
<td>≥ 0.6</td>
<td>≤ 0.0005</td>
<td>0.004</td>
<td>0.004</td>
<td>0.0</td>
<td>0.002</td>
</tr>
<tr>
<td>≥ 0.6</td>
<td>≤ 0.0005</td>
<td>0.006</td>
<td>0.006</td>
<td>0.0</td>
<td>0.004</td>
</tr>
<tr>
<td>Condition iii</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$P / f'_c$</td>
<td>$\rho = \frac{A_g}{b_h s}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>≥ 0.006</td>
<td>0.0</td>
<td>0.060</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>≥ 0.6</td>
<td>≥ 0.006</td>
<td>0.0</td>
<td>0.008</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>≤ 0.0005</td>
<td>0.0</td>
<td>0.006</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>≥ 0.6</td>
<td>≤ 0.0005</td>
<td>0.0</td>
<td>0.006</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Condition iv. Columns controlled by inadequate development or splicing along the clear height</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$P / f'_c$</td>
<td>$\rho = \frac{A_g}{b_h s}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>≥ 0.006</td>
<td>0.0</td>
<td>0.060</td>
<td>0.4</td>
<td>0.0</td>
</tr>
<tr>
<td>≥ 0.6</td>
<td>≥ 0.006</td>
<td>0.0</td>
<td>0.008</td>
<td>0.4</td>
<td>0.0</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>≤ 0.0005</td>
<td>0.0</td>
<td>0.006</td>
<td>0.2</td>
<td>0.0</td>
</tr>
<tr>
<td>≥ 0.6</td>
<td>≤ 0.0005</td>
<td>0.0</td>
<td>0.006</td>
<td>0.2</td>
<td>0.0</td>
</tr>
</tbody>
</table>

1. Refer to Section 6.4.2.2.2 for definition of conditions i, ii, and iii. Columns will be considered to be controlled by inadequate development or splices when the calculated steel stress at the splice exceeds the steel stress specified by Equation 6-2. Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.

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

3. Linear interpolation between values listed in the table shall be permitted.

4. Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.
### Table 6-9  Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures-Reinforced Concrete Beam-Column Joints

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Plastic Rotations Angle, radians</th>
<th>Residual Strength Ratio</th>
<th>Acceptance Criteria</th>
<th>Component Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a</td>
<td>b</td>
<td>c</td>
<td>IO</td>
</tr>
<tr>
<td>i. Interior joints$^{2,3}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>(\frac{P}{A_g f_y^{\prime}})</td>
<td>Trans. Reinf.$^1$</td>
<td>(\frac{V}{V_n})</td>
<td>Plastic Rotations Angle, radians</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>C</td>
<td>≤ 1.2</td>
<td>0.015</td>
<td>0.03</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>C</td>
<td>≥ 1.5</td>
<td>0.015</td>
<td>0.03</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>C</td>
<td>≤ 1.2</td>
<td>0.015</td>
<td>0.025</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>C</td>
<td>≥ 1.5</td>
<td>0.015</td>
<td>0.02</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>NC</td>
<td>≤ 1.2</td>
<td>0.005</td>
<td>0.02</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>NC</td>
<td>≥ 1.5</td>
<td>0.005</td>
<td>0.015</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>NC</td>
<td>≤ 1.2</td>
<td>0.005</td>
<td>0.015</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>NC</td>
<td>≥ 1.5</td>
<td>0.005</td>
<td>0.015</td>
</tr>
<tr>
<td>ii. Other joints$^{2,3}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>(\frac{P}{A_g f_y^{\prime}})</td>
<td>Trans. Reinf.$^1$</td>
<td>(\frac{V}{V_n})</td>
<td>Plastic Rotations Angle, radians</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>C</td>
<td>≤ 1.2</td>
<td>0.01</td>
<td>0.02</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>C</td>
<td>≥ 1.5</td>
<td>0.01</td>
<td>0.015</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>C</td>
<td>≤ 1.2</td>
<td>0.01</td>
<td>0.02</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>C</td>
<td>≥ 1.5</td>
<td>0.01</td>
<td>0.015</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>NC</td>
<td>≤ 1.2</td>
<td>0.005</td>
<td>0.01</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>NC</td>
<td>≥ 1.5</td>
<td>0.005</td>
<td>0.01</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>NC</td>
<td>≤ 1.2</td>
<td>0.0</td>
<td>0.0075</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>NC</td>
<td>≥ 1.5</td>
<td>0.0</td>
<td>0.0075</td>
</tr>
</tbody>
</table>

1. “C” and “NC” are abbreviations for conforming and nonconforming transverse reinforcement. A joint transverse reinforcement is conforming if hoops are spaced at \(\leq \frac{h_c}{3}\) within the joint. Otherwise, the component transverse reinforcement is considered nonconforming.
2. \(P\) is the design axial force on the column above the joint calculated using limit-state analysis procedures in accordance with Section 6.4.2.4 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 shear strength shall be calculated according to Section 6.4.2.3.
4. Linear interpolation between values listed in the table shall be permitted.
5. Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.
<table>
<thead>
<tr>
<th>Trans. Reinforcement $\rho''$</th>
<th>Interior joint with transverse beams</th>
<th>Interior joint without transverse beams</th>
<th>Exterior joint with transverse beams</th>
<th>Exterior joint without transverse beams</th>
<th>Knee joint with or without transverse beams</th>
</tr>
</thead>
<tbody>
<tr>
<td>(&lt;0.003)</td>
<td>12</td>
<td>10</td>
<td>8</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>(\geq0.003)</td>
<td>20</td>
<td>15</td>
<td>15</td>
<td>12</td>
<td>8</td>
</tr>
<tr>
<td>NC</td>
<td>12</td>
<td>10</td>
<td>8</td>
<td>6</td>
<td>4</td>
</tr>
</tbody>
</table>

$\rho''$ = volumetric ratio of horizontal confinement reinforcement in the joint;
1. “C” and “NC” are abbreviations for conforming and nonconforming transverse reinforcements. Joint transverse reinforcement is conforming if hoops are spaced at $h_c/2$ within the joint. Otherwise, the transverse reinforcement is considered nonconforming.
<table>
<thead>
<tr>
<th>Conditions</th>
<th>Performance Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>i. Beams controlled by flexure&lt;sup&gt;1&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>$\rho - \rho^*$</td>
<td>$\frac{\rho}{\rho_{tol}}$</td>
</tr>
<tr>
<td>$\leq 0.0$</td>
<td>C</td>
</tr>
<tr>
<td>$\leq 0.0$</td>
<td>C</td>
</tr>
<tr>
<td>$\geq 0.5$</td>
<td>C</td>
</tr>
<tr>
<td>$\leq 0.0$</td>
<td>NC</td>
</tr>
<tr>
<td>$\geq 0.5$</td>
<td>NC</td>
</tr>
<tr>
<td>$\geq 0.5$</td>
<td>NC</td>
</tr>
<tr>
<td>ii. Beams controlled by shear&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Stirrup spacing $\leq d/2$</td>
</tr>
<tr>
<td></td>
<td>Stirrup spacing $&gt; d/2$</td>
</tr>
<tr>
<td>iii. Beams controlled by inadequate development or splicing along the span&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Stirrup spacing $\leq d/2$</td>
</tr>
<tr>
<td></td>
<td>Stirrup spacing $&gt; d/2$</td>
</tr>
<tr>
<td>iv. Beams controlled by inadequate embedment into beam-column joint&lt;sup&gt;1&lt;/sup&gt;</td>
<td>2</td>
</tr>
</tbody>
</table>

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

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 $\leq d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops ($V'$) 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.

4. $V$ is the design shear force calculated using limit-state analysis procedures in accordance with Section 6.4.2.4.1.
Table 6-12  Numerical Acceptance Criteria for Linear Procedures-Reinforced Concrete Columns

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Performance Level</th>
<th>Component Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Primary</td>
<td>Secondary</td>
</tr>
<tr>
<td>i. Columns controlled by flexure$^1$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$P / A_g f'_c^6$</td>
<td>Trans. Reinf.$^5$</td>
<td>$V / b_o d \sqrt{f'_c^5}$</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>C</td>
<td>≤ 3</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>C</td>
<td>≥ 6</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>C</td>
<td>≤ 3</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>C</td>
<td>≥ 6</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>NC</td>
<td>≤ 3</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>NC</td>
<td>≥ 6</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>NC</td>
<td>≤ 3</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>NC</td>
<td>≥ 6</td>
</tr>
<tr>
<td>ii. Columns controlled by shear$^{1,3}$</td>
<td>Hoop spacing ≤ d/2,</td>
<td></td>
</tr>
<tr>
<td>or $P / A_g f'_c^6 ≤ 0.1$</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Other cases</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>iii. Columns controlled by inadequate development or splicing along the clear height$^{1,3}$</td>
<td>Hoop spacing ≤ d/2</td>
<td>1.25</td>
</tr>
<tr>
<td>Hoop spacing &gt; d/2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>iv. Columns with axial loads exceeding 0.70P_o$^{1,3}$</td>
<td>Conforming hoops over the entire length</td>
<td>1</td>
</tr>
<tr>
<td>All other cases</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

1. Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
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) 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. $V$ is the design shear force calculated using limit-state analysis procedures in accordance with Section 6.4.2.4.1.
6. $P$ is the design axial force in the member. Alternatively, use of axial loads determined based on a limit state analysis shall be permitted.
### Table 6-12 Numerical Acceptance Criteria for Linear Procedures-Reinforced Concrete Columns

<table>
<thead>
<tr>
<th>Conditions Id.</th>
<th>Component Type</th>
<th>Performance Level</th>
<th>m-factors&lt;sup&gt;3&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Primary</td>
<td>Secondary</td>
<td>IO</td>
</tr>
<tr>
<td>Condition i.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P/A&lt;sub&gt;g&lt;/sub&gt;f'&lt;sub&gt;c&lt;/sub&gt;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>≤ 0.1</td>
<td></td>
<td></td>
<td>2</td>
</tr>
<tr>
<td>≥ 0.6</td>
<td></td>
<td></td>
<td>1.25</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td></td>
<td></td>
<td>2</td>
</tr>
<tr>
<td>≥ 0.6</td>
<td></td>
<td></td>
<td>1.1</td>
</tr>
</tbody>
</table>

**Condition ii.**

| P/A<sub>g</sub>f'<sub>c</sub> | ρ = A<sub>s</sub>/b<sub>s</sub>, V/d<sub>f</sub> |  |
| ≤ 0.1          |                |                   | 2 | 2.5 | 3 | 4 | 5 |
| ≥ 0.6          |                |                   | 1.25 | 1.5 | 1.6 | 1.6 | 1.8 |
| ≤ 0.1          |                |                   | 1.2 | 1.3 | 1.4 | 1.4 | 1.6 |
| ≥ 0.6          |                |                   | 1 | 1 | 1.1 | 1.1 | 1.2 |
| ≥ 0.6          |                |                   | 1 | 1 | 1 | 1 | 1 |

**Condition iii.**

| P/A<sub>g</sub>f'<sub>c</sub> | ρ = A<sub>s</sub>/b<sub>s</sub> |  |
| ≤ 0.1          |                |                   | 1 | 1 | 1 | 4 | 5 |
| ≥ 0.6          |                |                   | 1 | 1 | 1 | 1.6 | 1.8 |
| ≤ 0.1          |                |                   | 1 | 1 | 1 | 1.1 | 1.2 |
| ≥ 0.6          |                |                   | 1 | 1 | 1 | 1 | 1 |

**Condition iv. Columns controlled by inadequate development or splicing along the clear height**

| P/A<sub>g</sub>f'<sub>c</sub> | ρ = A<sub>s</sub>/b<sub>s</sub> |  |
| ≤ 0.1          |                |                   | 1 | 1 | 1 | 4 | 5 |
| ≥ 0.6          |                |                   | 1 | 1 | 1 | 1.6 | 1.8 |
| ≤ 0.1          |                |                   | 1 | 1 | 1 | 1.1 | 1.2 |
| ≥ 0.6          |                |                   | 1 | 1 | 1 | 1 | 1 |

---

1. Refer to Section 6.4.2.2.2 for definition of conditions i, ii, and iii. Columns will be considered to be controlled by inadequate development or splicing when the calculated steel stress at the splice exceeds the steel stress specified by Equation 6-2. Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.

2. Where P > 0.7A<sub>f'c</sub>, the m-factor shall be taken as unity for all performance levels unless columns have transverse reinforcement consisting of hoops with 135 degree hooks spaced at ≤ d/3 and the strength provided by the hoops (V<sub>s</sub>) is at least three-fourths of the design shear. P is the design axial force in the member. Alternatively, use of axial loads determined based on a limit state analysis shall be permitted.

3. Linear interpolation between values listed in the table shall be permitted.

4. V is the design shear force calculated using limit-state analysis procedures in accordance with Section 6.4.2.4.1.
Table 6-13  Numerical Acceptance Criteria for Linear Procedures- Reinforced Concrete Beam-Column Joints

<table>
<thead>
<tr>
<th>Conditions</th>
<th>IO</th>
<th>LS</th>
<th>CP</th>
<th>LS</th>
<th>CP</th>
</tr>
</thead>
<tbody>
<tr>
<td>i. Interior joints$^{2,3}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\frac{P}{A_g f'_c}$</td>
<td>Trans. Reinf.$^1$</td>
<td>$\frac{V}{V_u}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>C</td>
<td>≤ 1.2</td>
<td>1-</td>
<td>1-</td>
<td>1-</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>C</td>
<td>≥ 1.5</td>
<td>1-</td>
<td>1-</td>
<td>1-</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>C</td>
<td>≤ 1.2</td>
<td>1-</td>
<td>1-</td>
<td>1-</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>C</td>
<td>≥ 1.5</td>
<td>1-</td>
<td>1-</td>
<td>1-</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>NC</td>
<td>≤ 1.2</td>
<td>1-</td>
<td>1-</td>
<td>1-</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>NC</td>
<td>≥ 1.5</td>
<td>1-</td>
<td>1-</td>
<td>1-</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>NC</td>
<td>≤ 1.2</td>
<td>1-</td>
<td>1-</td>
<td>1-</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>NC</td>
<td>≥ 1.5</td>
<td>1-</td>
<td>1-</td>
<td>1-</td>
</tr>
<tr>
<td>ii. Other joints$^{2,3}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\frac{P}{A_g f'_c}$</td>
<td>Trans. Reinf.$^1$</td>
<td>$\frac{V}{V_u}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>C</td>
<td>≤ 1.2</td>
<td>1-</td>
<td>1-</td>
<td>1-</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>C</td>
<td>≥ 1.5</td>
<td>1-</td>
<td>1-</td>
<td>1-</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>C</td>
<td>≤ 1.2</td>
<td>1-</td>
<td>1-</td>
<td>1-</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>C</td>
<td>≥ 1.5</td>
<td>1-</td>
<td>1-</td>
<td>1-</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>NC</td>
<td>≤ 1.2</td>
<td>1-</td>
<td>1-</td>
<td>1-</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>NC</td>
<td>≥ 1.5</td>
<td>1-</td>
<td>1-</td>
<td>1-</td>
</tr>
<tr>
<td>≥ 0.4</td>
<td>NC</td>
<td>≤ 1.2</td>
<td>1-</td>
<td>1-</td>
<td>1-</td>
</tr>
</tbody>
</table>

1. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcements. A joint transverse reinforcement is conforming if hoops are spaced at $\leq h_c/3$ within the joint. Otherwise, the component transverse reinforcement is considered nonconforming.

2. $P$ is the design axial force on the column above the joint calculated using limit-state analysis procedures in accordance with Section 6.4.2.4. $A_g$ is the gross cross-sectional area of the joint.

3. $V$ is the design shear force and $V_u$ is the shear strength for the joint. The design shear force and shear strength shall be calculated according to Section 6.4.2.4.1 and Section 6.4.2.3, respectively.

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

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Modeling Parameters</th>
<th>Acceptance Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plastic Rotation Angle, radians</td>
<td>IO</td>
</tr>
<tr>
<td></td>
<td>Residual Strength Ratio</td>
<td>LS</td>
</tr>
<tr>
<td></td>
<td>a</td>
<td>b</td>
</tr>
<tr>
<td>i. Slabs controlled by flexure, and slab-column connections(^1)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(\frac{V_g}{V_o})^2</td>
<td>Continuity Reinforcement(^3)</td>
<td></td>
</tr>
<tr>
<td>(\leq 0.2)</td>
<td>Yes</td>
<td>0.02</td>
</tr>
<tr>
<td>(\geq 0.4)</td>
<td>Yes</td>
<td>0.0</td>
</tr>
<tr>
<td>(\leq 0.2)</td>
<td>No</td>
<td>0.02</td>
</tr>
<tr>
<td>(\geq 0.4)</td>
<td>No</td>
<td>0.0</td>
</tr>
<tr>
<td>ii. Slabs controlled by inadequate development or splicing along the span(^1)</td>
<td>0.0</td>
<td>0.02</td>
</tr>
<tr>
<td>iii. Slabs controlled by inadequate embedment into slab-column joint(^1)</td>
<td>0.015</td>
<td>0.03</td>
</tr>
</tbody>
</table>

1. Where 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 listed in the table shall be permitted.

5. Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.
### Table 6-14  Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures-Two-way Slabs and Slab-Column Connections

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Plastic Rotation Angle, radians</th>
<th>Residual Strength Ratio</th>
<th>Acceptance Criteria&lt;sup&gt;5,6&lt;/sup&gt;</th>
<th>Component Type</th>
<th>Performance Level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a</td>
<td>b</td>
<td>c</td>
<td>IO</td>
<td>LS</td>
</tr>
<tr>
<td>i. Reinforced Concrete slab-column connections&lt;sup&gt;1&lt;/sup&gt;</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$V_g/V_o$</td>
<td>Continuity Reinforcement&lt;sup&gt;3&lt;/sup&gt;</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>Yes</td>
<td>0.035</td>
<td>0.05</td>
<td>0.2</td>
<td>0.01</td>
</tr>
<tr>
<td>0.2</td>
<td>Yes</td>
<td>0.03</td>
<td>0.04</td>
<td>0.2</td>
<td>0.01</td>
</tr>
<tr>
<td>0.4</td>
<td>Yes</td>
<td>0.02</td>
<td>0.03</td>
<td>0.2</td>
<td>0</td>
</tr>
<tr>
<td>$\geqslant$0.6</td>
<td>Yes</td>
<td>0</td>
<td>0.02</td>
<td>0</td>
<td>0</td>
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<tr>
<td>0</td>
<td>No</td>
<td>0.025</td>
<td>0.025</td>
<td>0</td>
<td>0.01</td>
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<tr>
<td>0.2</td>
<td>No</td>
<td>0.02</td>
<td>0.02</td>
<td>0</td>
<td>0.01</td>
</tr>
<tr>
<td>0.4</td>
<td>No</td>
<td>0.01</td>
<td>0.01</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.6</td>
<td>No</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>$&gt;$0.6</td>
<td>No</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>ii. Post-Tensioned slab-column connections&lt;sup&gt;1&lt;/sup&gt;</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$V_g/V_o$</td>
<td>Continuity Reinforcement&lt;sup&gt;3&lt;/sup&gt;</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>Yes</td>
<td>0.035</td>
<td>0.05</td>
<td>0.4</td>
<td>0.01</td>
</tr>
<tr>
<td>0.6</td>
<td>Yes</td>
<td>0.005</td>
<td>0.03</td>
<td>0.2</td>
<td>0</td>
</tr>
<tr>
<td>$\geqslant$0.6</td>
<td>Yes</td>
<td>0</td>
<td>0.02</td>
<td>0.2</td>
<td>0</td>
</tr>
<tr>
<td>0</td>
<td>No</td>
<td>0.025</td>
<td>0.025</td>
<td>0</td>
<td>0.01</td>
</tr>
<tr>
<td>0.6</td>
<td>No</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>$&gt;$0.6</td>
<td>No</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>iii. Slabs controlled by inadequate development or splicing along the span&lt;sup&gt;1&lt;/sup&gt;</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0.02</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>iv. Slabs controlled by inadequate embedment into slab-column joint&lt;sup&gt;1&lt;/sup&gt;</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.015</td>
<td>0.03</td>
<td>0.2</td>
<td>0.01</td>
<td>0.01</td>
<td>0.015</td>
</tr>
</tbody>
</table>

1. Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
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 the area of effectively continuous main bottom bars passing through the column cage in each direction is greater than or equal to $0.5V_g/(\phi f_y)$. Where the slab is post-tensioned, use "Yes" where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, use "No".
4. Action shall be treated as force-controlled
5. Linear interpolation between values listed in the table shall be permitted.
6. Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.
<table>
<thead>
<tr>
<th>Conditions</th>
<th>m-factors$^4$</th>
<th>Performance Level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Component Type</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Primary</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LS</td>
</tr>
<tr>
<td>i. Slabs controlled by flexure, and slab-column connections$^1$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\frac{V_g}{V_o}$ $^2$</td>
<td>Continuity Reinforcement$^3$</td>
<td></td>
</tr>
<tr>
<td>$\leq 0.2$</td>
<td>Yes</td>
<td>2</td>
</tr>
<tr>
<td>$\geq 0.4$</td>
<td>Yes</td>
<td>1</td>
</tr>
<tr>
<td>$\leq 0.2$</td>
<td>No</td>
<td>2</td>
</tr>
<tr>
<td>$\geq 0.4$</td>
<td>No</td>
<td>1</td>
</tr>
<tr>
<td>ii. Slabs controlled by inadequate development or splicing along the span$^1$</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>iii. Slabs controlled by inadequate embedment into slab-column joint$^1$</td>
<td></td>
<td>2</td>
</tr>
</tbody>
</table>

1. Where 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 listed in the table shall be permitted.
Table 6-15  Numerical Acceptance Criteria for Linear Procedures-
Two-way Slabs and Slab-Column Connections

<table>
<thead>
<tr>
<th>Conditions</th>
<th>m-factors⁵</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Performance Level</td>
</tr>
<tr>
<td>i. Reinforced Concrete slab-column connections¹</td>
<td></td>
</tr>
<tr>
<td>$\frac{V_g}{V_o}$²</td>
<td>Continuity Reinforcement³</td>
</tr>
<tr>
<td>0</td>
<td>Yes</td>
</tr>
<tr>
<td>0.2</td>
<td>Yes</td>
</tr>
<tr>
<td>0.4</td>
<td>Yes</td>
</tr>
<tr>
<td>✔= 0.6</td>
<td>Yes</td>
</tr>
<tr>
<td>0</td>
<td>No</td>
</tr>
<tr>
<td>0.2</td>
<td>No</td>
</tr>
<tr>
<td>0.4</td>
<td>No</td>
</tr>
<tr>
<td>0.6</td>
<td>No</td>
</tr>
<tr>
<td>✔&gt;0.6</td>
<td>No</td>
</tr>
<tr>
<td>ii. Post-Tensioned slab-column connections¹</td>
<td></td>
</tr>
<tr>
<td>$\frac{V_g}{V_o}$²</td>
<td>Continuity Reinforcement³</td>
</tr>
<tr>
<td>0</td>
<td>Yes</td>
</tr>
<tr>
<td>0.6</td>
<td>Yes</td>
</tr>
<tr>
<td>✔&gt;0.6</td>
<td>Yes</td>
</tr>
<tr>
<td>0</td>
<td>No</td>
</tr>
<tr>
<td>0.6</td>
<td>No</td>
</tr>
<tr>
<td>✔&gt;0.6</td>
<td>No</td>
</tr>
<tr>
<td>iii. Slabs controlled by inadequate development or splicing along the span¹</td>
<td></td>
</tr>
<tr>
<td>––⁴</td>
<td>––⁴</td>
</tr>
<tr>
<td>iv. Slabs controlled by inadequate embedment into slab-column joint¹</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>

1. Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
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 the area of effectively continuous main bottom bars passing through the column cage in each direction is greater than or equal to $0.5V_g/(\phi f_t)$. 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. Action shall be treated as force-controlled.
5. Linear interpolation between values listed in the table shall be permitted.
<table>
<thead>
<tr>
<th>Conditions</th>
<th>Modeling Parameters(^4)</th>
<th>Acceptance Criteria(^5)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total Strain</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Residual Strength Ratio</td>
<td></td>
</tr>
<tr>
<td></td>
<td>d</td>
<td>e</td>
</tr>
<tr>
<td>i. Columns modeled as compression chords(^3)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Columns confined along entire length(^2)</td>
<td>0.02</td>
<td>0.04</td>
</tr>
<tr>
<td>All other cases</td>
<td>0.003</td>
<td>0.01</td>
</tr>
<tr>
<td>ii. Columns modeled as tension chords(^5)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Columns with well-confined splices, or no splices</td>
<td>0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>All other cases</td>
<td>See note 1</td>
<td>0.03</td>
</tr>
</tbody>
</table>

1. Potential for splice failure shall be evaluated directly to determine the modeling and acceptance criteria. For these cases, refer to the generalized procedure of Sections 6.3.2. For primary components, 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 where 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 components of concrete shear walls. The maximum longitudinal spacing of sets of hoops shall not exceed either \(h/3\) or \(8d_c\).

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.

5. Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.
<table>
<thead>
<tr>
<th>Conditions</th>
<th>m-factors$^3$</th>
<th>Performance Level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Component Type</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Primary</td>
<td>Secondary</td>
</tr>
<tr>
<td></td>
<td>LS</td>
<td>CP</td>
</tr>
</tbody>
</table>

### 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 where 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 components of concrete shear walls. The maximum longitudinal spacing of sets of hoops shall not exceed either $h/3$ or $8d_c$.

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.
### Table 6-18: Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures-
R/C Shear Walls and Associated Components Controlled by Flexure

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Confining Boundary&lt;sup&gt;1&lt;/sup&gt;</th>
<th>Plastic Hinge Rotation&lt;sup&gt;0&lt;/sup&gt; (radians)</th>
<th>Residual Strength Ratio</th>
<th>Acceptable Plastic Hinge Rotation&lt;sup&gt;0&lt;/sup&gt;&lt;sup&gt;3,5&lt;/sup&gt; (radians)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Plastic Hinge Rotation (radians)</td>
<td>Residual Strength Ratio</td>
<td>Performance Level</td>
</tr>
<tr>
<td></td>
<td></td>
<td>IO</td>
<td>LS</td>
<td>CP</td>
</tr>
</tbody>
</table>

#### i. Shear walls and wall segments

\[
\frac{(A_i - A_b) f_y + P}{t_i f_y} = \frac{V}{t_i f_y \sqrt{f_y'}}
\]

<table>
<thead>
<tr>
<th>Condition</th>
<th>Confined Boundary&lt;sup&gt;1&lt;/sup&gt;</th>
<th>Plastic Hinge Rotation&lt;sup&gt;0&lt;/sup&gt; (radians)</th>
<th>Residual Strength Ratio</th>
<th>Acceptable Plastic Hinge Rotation&lt;sup&gt;0&lt;/sup&gt;&lt;sup&gt;3,5&lt;/sup&gt; (radians)</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 0.1</td>
<td>Yes</td>
<td>0.015 0.20 0.75 0.005 0.010 0.015 0.015 0.020</td>
<td></td>
<td></td>
</tr>
<tr>
<td>≥ 0.25</td>
<td>Yes</td>
<td>0.010 0.015 0.40 0.004 0.008 0.010 0.010 0.015</td>
<td></td>
<td></td>
</tr>
<tr>
<td>≥ 0.25</td>
<td>No</td>
<td>0.009 0.012 0.60 0.003 0.006 0.009 0.009 0.012</td>
<td></td>
<td></td>
</tr>
<tr>
<td>≤ 0.25</td>
<td>No</td>
<td>0.010 0.015 0.40 0.004 0.008 0.010 0.010 0.015</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### ii. Columns supporting discontinuous shear walls

- Conforming
  - Longitudinal reinforcement and transverse reinforcement<sup>2</sup>
  - ≤ 3: 0.025 0.050 0.75 0.010 0.02 0.025 0.025 0.050
  - ≥ 6: 0.020 0.040 0.50 0.005 0.010 0.020 0.020 0.040
- Nonconforming
  - 0.0 0.0 0.0 0.0 0.0 0.0 n.a. n.a.

#### iii. Shear wall coupling beams<sup>3</sup>

- Conventional longitudinal reinforcement with conforming 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
- Conventional longitudinal reinforcement with nonconforming transverse reinforcement
  - 0.020 0.035 0.50 0.006 0.012 0.020 0.020 0.035

#### Diagonal reinforcement

- 0.030 0.050 0.80 0.006 0.018 0.030 0.030 0.050

<sup>1</sup> A boundary element shall be considered confined where transverse reinforcement exceeds 75% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed 6d<sub>b</sub>. It shall be permitted to take modeling parameters and acceptance criteria as 80% of confined values where boundary elements have at least 50% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed 8d<sub>b</sub>. Otherwise, boundary elements shall be considered not confined.

<sup>2</sup> Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of: (a) hoops over the entire length of the coupling beam at a spacing ≤ d/4, and (b) strength of closed stirrups Vs ≥ 3/4 of required shear strength of the coupling beam.

<sup>3</sup> For secondary coupling beams spanning <8'-0">, with bottom reinforcement continuous into the supporting walls, secondary values shall be permitted to be doubled.

<sup>4</sup> Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.

<sup>5</sup> For secondary coupling beams spanning <8'-0", with bottom reinforcement continuous into the supporting walls, secondary values shall be permitted to be doubled.

<sup>6</sup> Linear interpolation between values listed in the table shall be permitted.
<table>
<thead>
<tr>
<th>Conditions</th>
<th>Total Drift Ratio (%), or Chord Rotation (radians)¹</th>
<th>Residual Strength Ratio</th>
<th>Acceptable Total Drift (%) or Chord Rotation (radians)¹</th>
<th>Performance Level</th>
<th>Component Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>d</td>
<td>e</td>
<td>c</td>
<td>LS</td>
<td>CP</td>
</tr>
<tr>
<td>i. Shear walls and wall segments</td>
<td>0.75</td>
<td>2.0</td>
<td>0.40</td>
<td>0.60</td>
<td>0.75 1.5</td>
</tr>
<tr>
<td>ii. Shear wall coupling beams¹</td>
<td>Conventional longitudinal reinforcement</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Conventional longitudinal reinforcement with transverse reinforcement</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≤ 3</td>
<td>0.02</td>
<td>0.030</td>
<td>0.60</td>
<td>0.006 0.015 0.020 0.020 0.030</td>
</tr>
<tr>
<td></td>
<td>≥ 6</td>
<td>0.016</td>
<td>0.024</td>
<td>0.30</td>
<td>0.005 0.012 0.016 0.016 0.024</td>
</tr>
<tr>
<td></td>
<td>Conventional longitudinal reinforcement with nonconforming transverse reinforcement</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≤ 3</td>
<td>0.012</td>
<td>0.025</td>
<td>0.40</td>
<td>0.006 0.008 0.010 0.010 0.020</td>
</tr>
<tr>
<td></td>
<td>≥ 6</td>
<td>0.008</td>
<td>0.014</td>
<td>0.20</td>
<td>0.004 0.006 0.007 0.007 0.012</td>
</tr>
</tbody>
</table>

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.
5. Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.
Table 6-19 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures-
R/C Shear Walls and Associated Components Controlled by Shear

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Total Drift Ratio (%), or Chord Rotation (radians)</th>
<th>Strength Ratio</th>
<th>Acceptable Total Drift (%) or Chord(^5) Rotation (radians)(^1)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(d)</td>
<td>(e)</td>
<td>(g)</td>
</tr>
<tr>
<td>i. Shear walls and wall segments(^2)</td>
<td>(A_s - A_v)</td>
<td>(f_s + P)</td>
<td>(t_s f'_c)</td>
</tr>
<tr>
<td>(A_s - A_v)</td>
<td>(f_s + P)</td>
<td>(t_s f'_c)</td>
<td>(&gt; 0.05)</td>
</tr>
<tr>
<td>ii. Shear wall coupling beams(^4)</td>
<td>Longitudinal reinforcement and transverse reinforcement(^3)</td>
<td>(V)</td>
<td>(t_s f'_c)</td>
</tr>
<tr>
<td>Conventional longitudinal reinforcement with conforming transverse reinforcement</td>
<td>(\geq 6)</td>
<td>0.016</td>
<td>0.024</td>
</tr>
<tr>
<td>Conventional longitudinal reinforcement with nonconforming transverse reinforcement</td>
<td>(\leq 3)</td>
<td>0.012</td>
<td>0.025</td>
</tr>
<tr>
<td>(\geq 6)</td>
<td>0.008</td>
<td>0.014</td>
<td>0.20</td>
</tr>
</tbody>
</table>

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 \(\leq 0.15 A_s f'_c\); 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 \(\leq d/3\); 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.
5. Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.
**Table 6-20  Numerical Acceptance Criteria for Linear Procedures- R/C Shear Walls and Associated Components Controlled by Flexure**

<table>
<thead>
<tr>
<th>Conditions</th>
<th>(m)-factors(^{2})</th>
<th>Performance Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>(f_{c} = f_{u} + P / \bar{A}_{c} ) [^{6}]</td>
<td></td>
<td>Component Type</td>
</tr>
<tr>
<td>IO</td>
<td>LS</td>
<td>CP</td>
</tr>
<tr>
<td>i. Shear walls and wall segments</td>
<td></td>
<td></td>
</tr>
<tr>
<td>((A_{c} - A_{t}) / f_{t} ) [^{6}]</td>
<td>(V / \bar{A}<em>{c} \sqrt{f</em>{t}^{c}} ) [^{5}]</td>
<td>Confined Boundary(^{1})</td>
</tr>
<tr>
<td>(\leq 0.1) [^{1}]</td>
<td>(\leq 3) [^{1}]</td>
<td>Yes</td>
</tr>
<tr>
<td>(\leq 0.1) [^{1}]</td>
<td>(\geq 6) [^{1}]</td>
<td>Yes</td>
</tr>
<tr>
<td>(\geq 0.25) [^{1}]</td>
<td>(\leq 3) [^{1}]</td>
<td>Yes</td>
</tr>
<tr>
<td>(\geq 0.25) [^{1}]</td>
<td>(\leq 6) [^{1}]</td>
<td>Yes</td>
</tr>
<tr>
<td>(\leq 0.1) [^{1}]</td>
<td>(\leq 3) [^{1}]</td>
<td>No</td>
</tr>
<tr>
<td>(\geq 0.25) [^{1}]</td>
<td>(\leq 6) [^{1}]</td>
<td>No</td>
</tr>
<tr>
<td>(\geq 0.25) [^{1}]</td>
<td>(\geq 6) [^{1}]</td>
<td>No</td>
</tr>
<tr>
<td>ii. Columns supporting discontinuous shear walls</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse reinforcement(^{2})</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conforming</td>
<td>(\leq 3) [^{5}]</td>
<td>2</td>
</tr>
<tr>
<td>Nonconforming</td>
<td>(\geq 6) [^{5}]</td>
<td>1.5</td>
</tr>
<tr>
<td>iii. Shear wall coupling beams(^{4})</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal reinforcement and transverse reinforcement(^{2})</td>
<td>(V / \bar{A}<em>{c} \sqrt{f</em>{t}^{c}} ) [^{5}]</td>
<td></td>
</tr>
<tr>
<td>Conventional longitudinal reinforcement with conforming transverse reinforcement</td>
<td>(\leq 3) [^{2}]</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>(\geq 6) [^{2}]</td>
<td>1.5</td>
</tr>
<tr>
<td>Conventional longitudinal reinforcement with nonconforming transverse reinforcement</td>
<td>(\leq 3) [^{2}]</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>(\geq 6) [^{2}]</td>
<td>1.2</td>
</tr>
<tr>
<td>Diagonal reinforcement</td>
<td>n.a. [^{2}]</td>
<td>2</td>
</tr>
</tbody>
</table>

1. A boundary element shall be considered confined where transverse reinforcement exceeds 75% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed 8. It shall be permitted to take modeling parameters and acceptance criteria as 80% of confined values where boundary elements have at least 50% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed 8. Otherwise, boundary elements shall be considered not confined. 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 \(\leq d / 2\), and (b) strength of hoops \(V_{s} \geq 3 / 4 \) of 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 \(\leq d / 3\), 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.

5. \(V\) is the design shear force calculated using limit-state analysis procedures in accordance with Section 6.7.2.4.

6. \(P\) is the design axial force in the member. Alternatively, use of axial loads determined based on a limit state analysis shall be permitted.

6. Linear interpolation between values listed in the table shall be permitted.
### Table 6-21 Numerical Acceptance Criteria for Linear Procedures- R/C Shear Walls and Associated Components Controlled by Shear

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Performance Level</th>
<th>Component Type</th>
<th>m-factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>i. Shear walls and wall segments(^1)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \frac{(A_i - A_i') f_y + P}{t_i f'c} ) ( \leq 0.05 ) All shear walls and wall segments(^4)</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>( \frac{(A_i - A_i') f_y + P}{t_i f'c} ) ( &gt; 0.05 )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ii. Shear wall coupling beams(^3)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal reinforcement and transverse reinforcement(^7)</td>
<td>( \frac{V}{t_i f'c} ) ( \leq 3 ) Conventional longitudinal reinforcement with conforming transverse reinforcement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conventional longitudinal reinforcement</td>
<td>( \frac{V}{t_i f'c} ) ( \geq 6 ) Conventional longitudinal reinforcement with nonconforming transverse reinforcement</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1. The shear shall be considered to be a force-controlled action for shear walls and wall segments where inelastic behavior is governed by shear and the design axial load is greater than \( 0.15 A_g f'c \). It shall be permitted to calculate the axial load based on a limit state analysis.

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 \( \leq d/3 \), and (b) strength of closed stirrups \( V_s \geq 3/4 \) 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.

4. \( V \) is the design shear force calculated using limit-state analysis procedures in accordance with Section 6.7.2.4.1.