6. Concrete (Systematic Rehabilitation)

6.1 Scope

Engineering procedures for estimating the seismic performance of lateral-force-resisting concrete components and elements are described in this chapter. Methods are applicable for concrete components that are either (1) existing components of a building system, (2) rehabilitated components of a building system, or (3) new components that are added to an existing building system.

Information needed for Systematic Rehabilitation of concrete buildings, as described in Chapter 2 and Chapter 3, is presented herein. Symbols used exclusively in this chapter are defined in Section 6.15. Section 6.2 provides a brief historical perspective of the use of concrete in building construction; a comprehensive historical perspective is contained in the *Commentary*. In Section 6.3, material and component properties are discussed in detail. Important properties of in-place materials and components are described in terms of physical attributes as well as how to determine and measure them. Guidance is provided on how to use the values in Tables 6-1 to 6-3 that might be used as default assumptions for material properties in a preliminary analysis.

General analysis and design assumptions and requirements are covered in Section 6.4. Critical modes of failure for beams, columns, walls, diaphragms, and foundations are discussed in terms of shear, bending, and axial forces. Components that are usually controlled by deformation are described in general terms. Other components that have limiting behavior controlled by force levels are presented along with Analysis Procedures.

Sections 6.5 through 6.13 cover the majority of the various structural concrete elements, including frames, braced frames, shear walls, diaphragms, and foundations. Modeling procedures, acceptance criteria, and rehabilitation measures for each component are discussed.

6.2 Historical Perspective

The components of concrete seismic resisting elements are columns, beams, slabs, braces, collectors, diaphragms, shear walls, and foundations. There has been a constant evolution in form, function, concrete strength, concrete quality, reinforcing steel strength, quality and detailing, forming techniques, and concrete placement techniques. All of these factors have a significant impact on the seismic resistance of a concrete building. Innovations such as prestressed and precast concrete, post tensioning, and lift slab construction have created a multivariant inventory of existing concrete structures.

The practice of seismic resistant design is relatively new to most areas of the United States, even though such practice has been evolving in California for the past 70 years. It is therefore important to investigate the local practices relative to seismic design when trying to analyze a specific building. Specific benchmark years can generally be determined for the implementation of seismic 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 has significant influence on 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. Also, early use of concrete was expanded by use of proprietary structural system designs and construction techniques. Some of these systems are described in the *Commentary* Section C6.2. The design professional is cautioned to fully examine available construction documents and in-place conditions in order to properly analyze and characterize historical concrete elements and components of buildings.

As indicated in Chapter 1, great care should be exercised in selecting the appropriate rehabilitation approaches and techniques for application to historic

Table 6-1 Tensile and Yield Properties of Concrete Reinforcing Bars for Various Periods

		Structural[1]	Intermediate[1]	Hard[1]			
	Grade	**33**	**40**	**50**	**60**	**70**	**75**
Year[3]	Minimum Yield[2] (psi)	33,000	40,000	50,000	50,000	60,000	75,000
	Minimum Tensile[2] (psi)	55,000	70,000	80,000	90,000	80,000	100,000
1911-1959		x	x	x			
1959-1966		x	x	x	x		x
1966-1972			x	x	x		
1972-1974			x	x	x		
1974-1987			x	x	x	x	
1987-present			x	x	x	x	x

General Note: An entry “x” indicates the grade was available in those years.

Specific Notes: 1. The terms structural, intermediate, and hard became obsolete in 1968.
2. Actual yield and tensile strengths may exceed minimum values.
3. Until about 1920, a variety of proprietary reinforcing steels were used. Yield strengths are likely to be in the range from 33,000 psi to 55,000 psi, but higher values are possible Plain and twisted square bars were sometimes used between 1900 and 1949.

buildings in order to preserve their unique characteristics.

Tables 6-1 and 6-2 contain a summary of reinforcing steel properties that might be expected to be encountered. Table 6-1 provides a range of properties for use where only the year of construction is known. Where both ASTM designations and year of construction are known, use Table 6-2. Properties of Welded Wire Fabric for various periods of construction can be obtained from the Wire Reinforcement Institute. Possible concrete strengths as a function of time are given in Table 6-3. A more detailed historical treatment is provided in Section C6.2 of the *Commentary*, and the reader is encouraged to review the referenced documents..

6.3 Material Properties and Condition Assessment

6.3.1 General

Quantification of in-place material properties and verification of existing system configuration and condition are necessary to analyze a building properly. This section identifies properties requiring consideration and provides guidelines for determining the properties of buildings. Also described is the need for a thorough condition assessment and utilization of knowledge gained in analyzing component and system behavior. Personnel involved in material property quantification and condition assessment shall be experienced in the proper implementation of testing practices, and interpretation of results.

The extent of in-place materials testing and condition assessment needed is related to the availability and accuracy of construction (as-built) records, quality of materials and construction, and physical condition. Documentation of properties and grades of material used in component/connection construction is invaluable and may be effectively used to reduce the amount of in-place testing required. The design professional is encouraged to research and acquire all available records from original construction.

6.3.2 Properties of In-Place Materials and Components

6.3.2.1 Material Properties

Mechanical properties of component and connection material strongly influence the structural behavior under load. Mechanical properties of greatest interest for concrete elements and components include the following:

- Concrete compressive and tensile strengths and modulus of elasticity

Table 6-2 *Tensile and Yield Properties of Concrete Reinforcing Bars for Various ASTM Specifications and Periods*

				Structural[1]	**Intermediate[1]**	**Hard[1]**			
			Grade	**33**	**40**	**50**	**60**	**70**	**75**
			Minimum Yield[2] (psi)	33,000	40,000	50,000	50,000	60,000	75,000
ASTM	Steel Type	Year Range[3]	Minimum Tensile[2] (psi)	55,000	70,000	80,000	90,000	80,000	100,000
A15	Billet	1911-1966		x	x	x			
A16	Rail[4]	1913-1966				x			
A61	Rail[4]	1963-1966					x		
A160	Axle	1936-1964		x	x	x			
A160	Axle	1965-1966		x	x	x	x		
A408	Billet	1957-1966		x	x	x			
A431	Billet	1959-1966							x
A432	Billet	1959-1966					x		
A615	Billet	1968-1972			x		x		x
A615	Billet	1974-1986			x		x		
A615	Billet	1987-1997			x		x		x
A616	Rail[4]	1968-1997				x	x		
A617	Axle	1968-1997			x		x		
A706	Low-Alloy[5]	1974-1997						x	
A955	Stainless	1996-1997			x		x		x

General Note: An entry “x” indicates the grade was available in those years.

Specific Notes:
1. The terms structural, intermediate, and hard became obsolete in 1968.
2. Actual yield and tensile strengths may exceed minimum values.
3. Until about 1920, a variety of proprietary reinforcing steels were used. Yield strengths are likely to be in the range from 33,000 psi to 55,000 psi, but higher values are possible Plain and twisted square bars were sometimes used between 1900 and 1949.
4. Rail bars should be marked with the letter “R.” Bars marked “s!” (ASTM 616) have supplementary requirements for bend tests.
5. ASTM steel is marked with the letter “W.”

Table 6-3 Compressive Strength of Structural Concrete (psi)[1]

Time Frame	Footings	Beams	Slabs	Columns	Walls
1900–1919	1000–2500	2000–3000	1500–3000	1500–3000	1000–2500
1920–1949	1500–3000	2000–3000	2000–3000	2000–4000	2000–3000
1950–1969	2500–3000	3000–4000	3000–4000	3000–6000	2500–4000
1970–Present	3000–4000	3000–5000	3000–5000	3000–10000[2]	3000–5000

1. Concrete strengths are likely to be highly variable within any given older structure.
2. Exceptional cases of very high strength concrete may be found.

- Yield and ultimate strength of conventional and prestressing reinforcing steel and metal connection hardware
- Ductility, toughness, and fatigue properties
- Metallurgical condition of the reinforcing steel, including carbon equivalent, presence of any degradation such as corrosion, bond with concrete, and chemical composition.

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

In general, the determination of material properties (other than connection behavior) is best accomplished through removal of samples and laboratory analysis. Sampling shall take place in primary gravity- and lateral-force-resisting components. Where possible, sampling shall occur in regions of reduced stress to limit the effects of reduced sectional area. The size of the samples and removal practices to be followed are referenced in the *Commentary*. The frequency of sampling, including the minimum number of tests for property determination, is addressed in Section 6.3.2.4.

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. For concrete, the sampling program shall consist of the removal of standard vertical or horizontal cores. Core drilling shall be preceded by nondestructive location of the reinforcing steel, and shall avoid damaging the existing reinforcing steel as much as practicable. Core holes shall be filled with comparable-strength concrete or grout. For conventional reinforcing and bonded prestressing steel, sampling shall consist of the removal of local bar segments (extreme care shall be taken with removal of any prestressing steels). Depending on the location and amount of bar removed, replacement spliced material shall be installed to maintain continuity.

6.3.2.2 Component Properties

Structural elements often utilize both primary and secondary components to perform their load- and deformation-resisting function. Behavior of the components, including beams, columns, and walls, is dictated by such properties as cross-sectional dimensions and area, reinforcing steel location, width-to-thickness and slenderness ratios, lateral buckling resistance, and connection details. This behavior may also be altered by the presence of degradation or physical damage. The following component properties shall be established during the condition assessment phase of the seismic rehabilitation process to aid in evaluating component behavior (see Section 6.3.3 for assessment guidelines):

- Original and current cross-sectional dimensions
- As-built configuration and physical condition of primary component end connections, and intermediate connections such as those between diaphragms and supporting beams/girders
- Size, anchorage, and thickness of other connector materials, including metallic anchor bolts, embeds, bracing components, and stiffening materials, commonly used in precast and tilt-up construction

- Characteristics that may influence the continuity, moment-rotation, or energy dissipation and load transfer behavior of connections
- Confirmation of load transfer capability at component-to-element connections, and overall element/structure behavior

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

In the absence of degradation, component dimensions and properties from original drawings may be used in structural analyses without introducing significant error. Variance from nominal dimensions, such as reinforcing steel size and effective area, is usually small.

6.3.2.3 Test Methods to Quantify Properties

To obtain the desired in-place mechanical properties of materials and components, it is necessary to use proven destructive and nondestructive testing methods. Certain field tests—such as estimation of concrete compressive strength from hardness and impact resistance tests—may be performed, but laboratory testing shall be used where strength is critical. Critical properties of concrete commonly include the compressive and tensile strength, modulus of elasticity, and unit weight. Samples of concrete and reinforcing and connector steel shall also be examined for physical condition (see Section 6.3.3.2).

Accurate determination of existing concrete strength properties is typically achieved through removal of core samples and performance of laboratory destructive testing. Removal of core samples should employ the procedures contained in *ASTM C 42*. Testing should follow the procedures contained in *ASTM C 42*, *C 39*, and *C 496*. The measured strength from testing must be correlated to in-place concrete compressive strength; the *Commentary* provides further guidance on correlating core strength to in-place strength and other recommendations. The *Commentary* provides references for various test methods that may be used to estimate material properties.

Accurate determination of existing reinforcing steel strength properties is typically achieved through removal of bar or tendon length samples and performance of laboratory destructive testing. The primary strength measures for reinforcing and prestressing steels are the tensile yield strength and ultimate strength, as used in the structural analysis. Strength values may be obtained by using the procedures contained in *ASTM A 370*. Prestressing materials must also meet the supplemental requirements in *ASTM A 416*, *A 421*, or *A 722*, depending on material type. The chemical composition may also be determined from the retrieved samples.

Particular test methods that may be used for connector steels include wet and dry chemical composition tests, and direct tensile and compressive strength tests. For each test, industry standards published by ASTM, including Standard *A 370*, exist and shall be followed. For embedded connectors, the strength of the material may also be assessed in situ using the provisions of *ASTM E 488*. The *Commentary* provides references for these tests.

Usually, the reinforcing steel system used in 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. In developing a testing program, the design professional shall consider the possibility of varying concrete classes. Historical research and industry documents also contain insight on material mechanical properties used in different construction eras. Section 6.3.2.5 provides strength data for most primary concrete and reinforcing steels used. This information, with laboratory and field test data, may be used to gain confidence in in situ strength properties.

6.3.2.4 Minimum Number of Tests

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

Testing is not required on components other than those of the lateral-force-resisting system. If the existing lateral-force-resisting system is being replaced in the rehabilitation process, minimum material testing is needed to qualify properties of existing materials at new connection points.

A. 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 tests shall be done for the complete concrete building structure, subject to the limitations noted below. If varying concrete classes/grades were employed in building construction, a minimum of three samples and tests shall be performed for each class. Test results shall be compared with strength values specified in the construction documents. The core strength shall be converted to in situ concrete compressive strength (f'_c) as in Section C6.3.2.3 of the *Commentary.* The unit weight and modulus of elasticity shall be derived or estimated during strength testing. Samples should be taken at random locations in components critical to structural behavior of the building. Tests shall also be performed on samples from components that are damaged or degraded, to quantify their condition. 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 whether the condition is localized.

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

- 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 (estimated smallest of the three).
- 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 (use smallest number). Where the results indicate that different classes of concrete were employed, the degree of testing shall be increased to confirm class use.
- A minimum of three samples shall be removed for splitting tensile strength determination, if a lightweight aggregate concrete were used for primary components. Additional tests may be warranted, should the coefficient of variation in test results exceed 14%.

If a sample population greater than the minimum specified is used in the testing program and the coefficient of variation in test results is less than 14%, the mean strength derived may be used as the expected strength in the analysis. If the coefficient of variation from testing is greater than 14%, additional sampling and testing should be performed to improve the accuracy of testing or understanding of in situ material strength. The design professional (and subcontracted testing agency) shall 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. In general, the expected concrete strength shall not exceed the mean less one standard deviation in situations where variability is greater than 14%.

In addition to destructive sampling and testing, further quantification of concrete strength may be estimated via ultrasonics, or another nondestructive test method (see the *Commentary*). Because these methods do not yield accurate strength values directly, they should be used for confirmation and comparison only and shall not be substituted for core sampling and laboratory testing.

B. Conventional Reinforcing and Connector Steels

In terms of defining reinforcing and connector steel strength properties, the following guidelines shall be followed. Connector steel is defined as additional structural or bolting steel material used to secure precast and other concrete shapes to the building structure. Both yield and ultimate strengths shall be determined. 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.

- If original construction documents defining properties exist, and if an Enhanced Rehabilitation Objective (greater than the BSO) is desired, at least three strength coupons shall be randomly removed from each element or component type (e.g., slabs, walls, beams) and tested.
- If original construction documents defining properties do not exist, but the approximate date of construction is known and a common material grade is confirmed (e.g., all bars are Grade 60 steel), at least three strength coupons shall be randomly removed from each element or component type (e.g., beam, wall) for every three floors of the building. If the date of construction is unknown, at least six such samples/tests, for every three floors, shall be performed. This is required to satisfy the BSO.

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

C. Prestressing Steels

The sampling of prestressing steel tendons for laboratory testing shall be accomplished with extreme care; only those prestressed components that are a part of the lateral-force-resisting system shall be considered. Components in diaphragms should generally be excluded from testing. If limited information exists regarding original materials and the prestressing force applied, the design professional must attempt to quantify properties for analysis. Tendon removals shall be avoided if possible in prestressed members. Only a minimum number of tendon samples for laboratory testing shall be taken.

Determination of material properties may be possible, without tendon removal or prestress removal, by careful sampling of either the tendon grip or extension beyond the anchorage.

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

D. General

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 shall examine the particular need for this type of testing and establish an adequate protocol. In general, it is recommended that a minimum of three tests be conducted to determine any property. If outliers (results with coefficients of variation greater than 15%) are detected, additional tests shall be performed until an accurate representation of the property is gained.

6.3.2.5 Default Properties

Mechanical properties for materials and components shall be based on available historical data for the particular structure and tests on in-place conditions. Should extenuating circumstances prevent minimum material sampling and testing from being performed, default strength properties may be used. Default material and component properties have been established for concrete compressive strength and reinforcing steel tensile and yield strengths from published literature; these are presented in Tables 6-1 to 6-3. These default values are generally conservative, representing values reduced from mean strength in order to address variability. However, the selection of a default strength for concrete shall be made with care because of the multitude of mix designs and materials used in the construction industry.

For concrete default compressive strength, lower-bound values from Table 6-3 may be used. The default compressive strength shall be used to establish other strength and performance characteristics for the concrete as needed in the structural analysis.

Reinforcing steel tensile properties are presented in Tables 6-1 and 6-2. Because of lower variability, the lower-bound tabulated values may be used without further reduction. For Rehabilitation Objectives in which default values are assumed for existing reinforcing steel, no welding or mechanical coupling of new reinforcing to the existing reinforcing steel is permitted. For buildings constructed prior to 1950, the bond strength developed between reinforcing steel and concrete may be less than present-day strength. The tensile lap splices and development length of older plain reinforcing should be considered as 50% of the capacity of present-day tabulated values (such as in *ACI 318-95*

[ACI, 1995]) unless further justified through testing and assessment (CRSI, 1981).

For connector materials, the nominal strength from design and construction documents may be used. In the absence of this information, the default yield strength for steel connector material may be taken as 27,000 psi.

Default values for prestressing steel in prestressed concrete construction shall not be used, unless circumstances prevent material sampling/testing from being performed. In this case, it may be prudent to add a new lateral-force-resisting system to the building.

6.3.3 Condition Assessment

6.3.3.1 General

A condition assessment of the existing building and site conditions shall be performed as part of the seismic rehabilitation process. The goal of this assessment is threefold:

- To examine the physical condition of primary and secondary components and the presence of any degradation
- To verify the presence and configuration of components and their connections, and the continuity of load paths between components, elements, and systems
- To review other conditions that may influence existing building performance, such as neighboring party walls and buildings, nonstructural components that may contribute to resistance, and any limitations for rehabilitation

The physical condition of existing components and elements, and their connections, must be examined for presence of degradation. Degradation may include environmental effects (e.g., corrosion, fire damage, chemical attack), or past or current loading effects (e.g., overload, damage from past earthquakes, fatigue, fracture). The condition assessment shall also examine for configurational problems observed in recent earthquakes, including effects of discontinuous components, construction deficiencies, poor fit-up, and ductility problems.

Component orientation, plumbness, and physical dimensions should be confirmed during an assessment. Connections in concrete components, elements, and systems require special consideration and evaluation. The load path for the system must be determined, and each connection in the load path(s) must be evaluated. This includes diaphragm-to-component and component-to-component connections. Where the connection is attached to one or more components that are expected to experience significant inelastic response, the strength and deformation capacity of connections must be evaluated. The condition and detailing of at least one of each connection type should be investigated.

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

6.3.3.2 Scope and Procedures

The scope of the condition assessment should include all primary structural elements and components involved in gravity and lateral load resistance, as limited by accessibility. The knowledge and insight gained from the condition assessment is invaluable to the understanding of load paths and the ability of components to resist and transfer these loads. The degree of assessment performed also affects the κ factor that is used in the analysis, and the type of analysis (see Section 6.3.4).

A. Visual Inspection

Direct visual inspection provides the most valuable information, as it can be used to quickly identify any configurational issues, and it allows the measurement of component dimensions, and the determination whether degradation is present. The continuity of load paths may be established through viewing of components and connection condition. From visual inspection, the need for other test methods to quantify the presence and degree of degradation may be established. The dimensions of accessible primary components shall be measured and compared with available design information. Similarly, the configuration and condition of all connections (exposed surfaces) shall be verified with permanent deformations or other noted anomalies.

Industry-accepted procedures are cited in the *Commentary.*

Visual inspection of the specific building should include all elements and components constructed of concrete, including foundations, vertical and horizontal frame members, diaphragms (slabs), and connections. As a minimum, a representative sampling of at least 20% of the elements, components, and connections shall be visually inspected at each floor level. If significant damage or degradation is found, the assessment sample shall be increased to all critical components of similar type in the building. The damage should be quantified using supplemental methods cited in this chapter and the *Commentary.*

If coverings or other obstructions exist, indirect visual inspection through the obstruction may be conducted by using drilled holes and a fiberscope. If this method is not appropriate, then exposure will be necessary. 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 guidelines shall be used for assessing primary connections in the building.

- If detailed design drawings exist, exposure of at least three different primary connections shall occur, with the connection sample including different types (e.g., beam-column, column-foundation, beam-diaphragm). If no deviations from the drawings exist, the sample may be considered representative of installed conditions. If deviations are noted, then exposure of at least 25% of the specific connection type is necessary to identify the extent of deviation.

- In the absence of accurate drawings, exposure of at least three connections of each primary connection type shall occur for inspection. If common detailing is observed, this sample may be considered representative. If many different details of deviations are observed, increased connection inspection is warranted until an accurate understanding of building construction and behavior is gained.

B. Additional Testing

The physical condition of components and connectors may also dictate the need for certain destructive and nondestructive test methods. Such methods may be used to determine the degree of damage or presence of contamination, 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 the *Commentary.* 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 pachometer). Further assessment of suspected corrosion activity should utilize 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” (ASCE, 1990).

The *Commentary* provides general background and references for these methods.

6.3.3.3 Quantifying Results

The results of the condition assessment shall be used in the preparation of building system models in the evaluation of seismic performance. To aid in this effort, the results shall be quantified, with the following specific topics addressed:

- Component section properties and dimensions
- Component configuration and presence of any eccentricities or permanent deformation
- Connection configuration and presence of any eccentricities
- Presence and effect of alterations to the structural system since original construction (e.g., doorways cut into shear walls)
- Interaction of nonstructural components and their involvement in lateral load resistance

As previously noted, the acceptance criteria for existing components depend on the design professional's knowledge of the condition of the structural system and material properties. All deviations noted between available construction records and as-built conditions shall be accounted for and considered in the structural analysis. Again, some removal of cover concrete is required during this stage to confirm reinforcing steel configuration.

Gross component section properties in the absence of degradation have been found to be statistically close to nominal. 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 via direct measurement. The sectional properties shall then be reduced accordingly, using the principles of structural mechanics. If the degradation is significant, further analysis or rehabilitative measures shall be undertaken.

6.3.4 Knowledge (κ) Factor

As described in Section 2.7, computation of component capacities and allowable deformations involves the use of a knowledge (κ) factor. For cases where a Linear Static Procedure (LSP) will be used in the analysis, two possible values for κ exist (0.75 and 1.0). For nonlinear procedures, the design professional must obtain an in-depth understanding of the building structural system and condition to support the use of a κ factor of 1.0. This section further describes the requirements and selection criteria for a κ factor specific to concrete structural components.

If the concrete structural system is exposed and good access exists, significant knowledge regarding configuration and behavior may be gained through condition assessment. In general, a κ factor of 1.0 can be used when a thorough assessment is performed on the primary/secondary components and load paths, and the requirements of Sections 2.7 and 6.3.3 are met. This assessment should include exposure of at least one sample of each primary component connection type and comparison with construction documents. However, if original reinforcing steel shop drawings, material specifications, and field inspection or quality control records are available, this effort is not required.

If incomplete knowledge of as-built component or connection configuration exists because a smaller sampling is performed than that required for $\kappa = 1.0$, κ shall be reduced to 0.75. Rehabilitation requires that a minimum sampling be performed from which knowledge of as-built conditions can be surmised. Where a κ of 0.75 cannot be justified, no seismic resistance capacity may be used for existing components.

If all required testing for $\kappa = 1.0$ is done and the following situations prevail, κ shall be reduced to 0.75.

- Construction documents for the concrete structure are not available or are incomplete.
- Components are found degraded during assessment, for which further testing is required to qualify behavior and to use $\kappa = 1.0$.
- Components have high variability in mechanical properties (up to a coefficient of variation of 25%).
- Components shown in construction documents lack sufficient structural detail to allow proper analysis.
- Components contain archaic or proprietary material and their materials condition is uncertain.

6.3.5 Rehabilitation Issues

Upon determining that concrete elements in an existing building are deficient for the desired Rehabilitation Objective, the next step is to define rehabilitation or replacement alternatives. If replacement of the element is selected, design of the new element shall be in accordance with local building codes and the *NEHRP Recommended Provisions* for new buildings (BSSC, 1995).

6.3.6 Connections

Connections between existing concrete components and any components added to rehabilitate the original structure are critical to overall seismic performance. The design professional is strongly encouraged to examine as-built connections and perform any physical testing/inspection to assess their performance. All new connections shall be subject to the quality control provisions contained in these *Guidelines*. In addition, for connectors that are not cast-in-place, such as anchor bolts, a minimum of five samples from each connector type shall be tested after installation. Connectors that rely on ductility shall be tested according to Section 2.13. (See also Section 6.4.6.)

6.4 General Assumptions and Requirements

6.4.1 Modeling and Design

6.4.1.1 General Approach

Design approaches for an existing or rehabilitated building generally shall follow procedures of *ACI 318-95* (ACI, 1995), except as otherwise indicated in these *Guidelines*, and shall emphasize the following.

- Brittle or low-ductility failure modes shall be identified as part of the analysis. These 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 preferred that the stresses, forces, and moments acting to cause these failure modes be determined from consideration of the probable resistances at the locations for nonlinear action.

- Analysis of reinforced concrete components shall include an evaluation of demands and capacities at all sections along the length of the component. Particular attention shall be paid to locations 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.

6.4.1.2 Stiffness

Component stiffnesses shall be calculated according to accepted principles of mechanics. Sources of flexibility shall include flexure, shear, axial load, and reinforcement slip from adjacent connections and components. Stiffnesses should be selected to represent the stress and deformation levels to which the components will be subjected, considering volume change effects (temperature and shrinkage) combined with design earthquake and gravity load effects.

A. 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 for the component, except that higher stiffnesses may be used where it is demonstrated by analysis to be appropriate for the design loading. The effective stiffness values in Table 6-4 should be used, except where little nonlinear behavior is expected or detailed evaluation justifies different values. These same stiffnesses may be appropriate for the initial stiffness for use in the nonlinear procedures of Chapter 3.

B. 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, except that linear relations are acceptable where nonlinear response will not occur in the component. The nonlinear load-deformation relation shall be based on experimental evidence or may be taken from quantities specified in Sections 6.5 through 6.13. The nonlinear load-deformation relation for the Nonlinear Static Procedure (NSP) may be composed of line segments or curves defining behavior under monotonically increasing lateral deformation. The nonlinear load-deformation relation for the Nonlinear Dynamic Procedure (NDP) may be composed of line segments or curves, and shall define behavior under monotonically increasing lateral deformation and under multiple reversed deformation cycles.

Figure 6-1 illustrates a generalized load-deformation relation that may be applicable for most concrete components evaluated using the NSP. The relation is

Table 6-4 Effective Stiffness Values

Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams—nonprestressed	$0.5E_cI_g$	$0.4E_cA_w$	—
Beams—prestressed	E_cI_g	$0.4E_cA_w$	—
Columns in compression	$0.7E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns in tension	$0.5E_cI_g$	$0.4E_cA_w$	E_sA_s
Walls—uncracked (on inspection)	$0.8E_cI_g$	$0.4E_cA_w$	E_cA_g
Walls—cracked	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
Flat Slabs—nonprestressed	See Section 6.5.4.2	$0.4E_cA_g$	—
Flat Slabs—prestressed	See Section 6.5.4.2	$0.4E_cA_g$	—

Note: I_g for T-beams may be taken as twice the value of I_g of the web alone, or may be based on the effective width as defined in Section 6.4.1.3. For shear stiffness, the quantity $0.4E_c$ has been used to represent the shear modulus G.

described by linear response from *A* (unloaded component) to an effective yield *B*. Subsequently, there is linear response, at reduced stiffness, from *B* to *C*, with sudden reduction in lateral load resistance to *D*, response at reduced resistance to *E*, and final loss of resistance thereafter. The slope from *A* to *B* shall be according to Section 6.4.1.2A. The slope from *B* to *C*, ignoring effects of gravity loads acting through lateral displacements, typically may be taken as equal to between zero and 10% of the initial slope. *C* has an ordinate equal to the strength of the component and an abscissa equal to the deformation at which significant strength degradation begins. It is permissible to represent the load-deformation relation by lines connecting points *A*, *B*, and *C*, provided that the calculated response is not beyond *C*. It is also acceptable to use more refined relations where they are justified by experimental evidence. Sections 6.5 through 6.13 recommend numerical values for the points identified in Figure 6-1.

Typically, the responses shown in Figure 6-1 are associated with flexural response or tension response. In this case, the resistance at $Q/Q_{CE} = 1.0$ is the yield value, and subsequent strain hardening accommodates strain hardening in the load-deformation relation as the member is deformed toward the expected strength. When the response shown in Figure 6-1 is associated with compression, the resistance at $Q/Q_{CE} = 1.0$ typically is the value at which concrete begins to spall, and strain hardening in well-confined sections may be associated with strain hardening of the longitudinal reinforcement and the confined concrete. When the

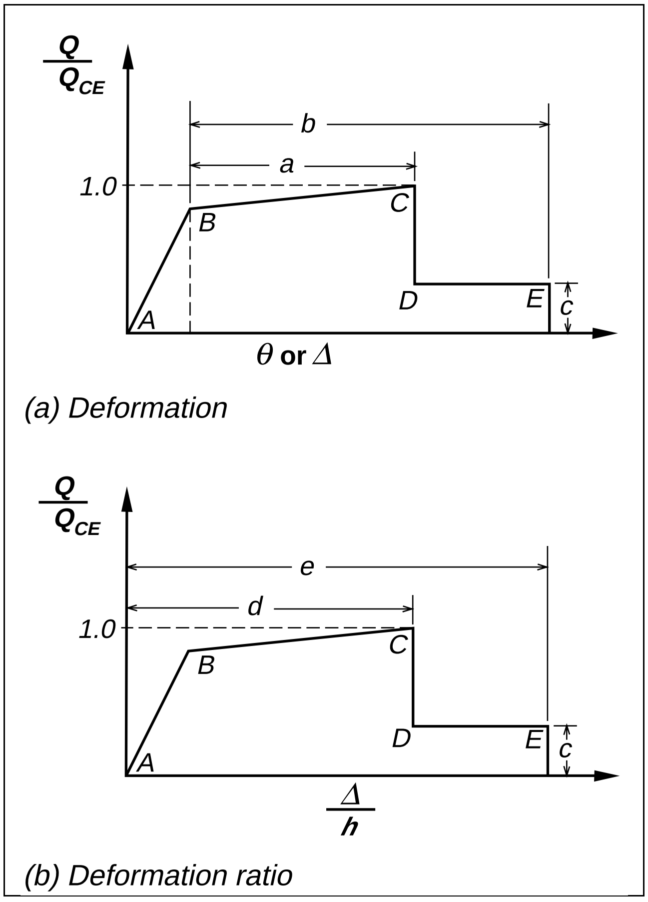

Figure 6-1 Generalized Load-Deformation Relation

response shown in Figure 6-1 is associated with shear, the resistance at Q/Q_{CE} = 1.0 typically is the value at which the design shear strength is reached, and no strain hardening follows.

Figure 6-1 shows two different ways to define the deformations, 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* 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.

(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.

6.4.1.3 Flanged Construction

In components and elements 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, and (4) one-fifth of the span for beams or one-half the total height for walls. When the flange is in compression, both the concrete and reinforcement within the effective width shall be considered effective in resisting flexure and axial load. When the flange is in tension, longitudinal reinforcement within the effective width shall be considered fully effective for resisting flexure and axial loads, provided that proper splice lengths in the reinforcement can be verified. The portion of the flange extending beyond the width of the web shall be assumed ineffective in resisting shear.

6.4.2 Design Strengths and Deformabilities

6.4.2.1 General

Actions in a structure shall be classified as being either deformation-controlled or force-controlled, as defined in Chapter 3. General procedures for calculating design strengths for deformation-controlled and force-controlled actions shall be according to Sections 6.4.2.2 and 6.4.2.3.

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

Where strength and deformation capacities are derived from test data, the tests shall be representative of proportions, details, and stress levels for the component. General requirements for testing are specified in Section 2.13.1.

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 more numerous 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 more numerous deformation cycles should be considered in design. Large earthquakes will cause more numerous cycles.

6.4.2.2 Deformation-Controlled Actions

Deformation-controlled actions are defined in Section 3.2.2.4. Strengths used in design for deformation-controlled actions generally are denoted Q_{CE} and shall be taken as equal to expected strengths obtained experimentally or calculated using accepted mechanics principles. Expected strength is defined as the mean maximum resistance expected over the range of deformations to which the component is likely to be subjected. When calculations are used to define mean expected strength, expected material strength—including strain hardening—is to be taken into account. The tensile stress in yielding longitudinal reinforcement shall be assumed to be at least 1.25 times the nominal yield stress. Procedures specified in ACI 318 may be used to calculate strengths used in design, except that the strength reduction factor, ϕ, shall be taken as equal to unity, and other procedures specified in these *Guidelines* shall govern where applicable.

6.4.2.3 Force-Controlled Actions

Force-controlled actions are defined in Chapter 3. Strengths used in design for force-controlled actions generally are denoted Q_{CL} and shall be taken as equal to lower bound strengths obtained experimentally or calculated using established mechanics principles. Lower bound strength is defined generally as the lower five percentile of strengths expected. Where the strength degrades with continued cycling or increased lateral deformations, the lower bound strength is defined as the expected minimum value within the range of deformations and loading cycles to which the component is likely to be subjected. When calculations are used to define lower bound strengths, lower bound estimates of material properties are to be assumed. Procedures specified in ACI 318 may be used to calculate strengths used in design, except other procedures specified in the *Guidelines* shall govern where applicable (see Section 6.3.2.5).

6.4.2.4 Component Ductility Demand Classification

Some strength calculation procedures in this chapter require definition of component ductility demand classification. For this purpose, components shall also be classified as having low, moderate, or high ductility demands, based on the maximum value of the demand capacity ratio (DCR; see Section 2.9.1) from the linear procedures of Chapter 3, or the calculated displacement ductility from the nonlinear procedures of Chapter 3. Table 6-5 defines the relation.

Table 6-5 Component Ductility Demand Classification

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

6.4.3 Flexure and Axial Loads

Flexural strength and deformability of members with and without axial loads shall be calculated according to accepted procedures. Strengths and deformabilities of components with monolithic flanges shall be calculated considering concrete and developed longitudinal reinforcement within the effective flange width defined in Section 6.4.1.3. Strengths and deformabilities shall be determined considering available development of longitudinal reinforcement.

Without confining transverse reinforcement, maximum usable strain at extreme concrete compression fiber shall not exceed 0.002 for components in nearly pure compression and 0.005 for other components. Larger strains are permitted where transverse reinforcement provides confinement. Maximum allowable compression 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 compression strain shall not exceed 0.02, and maximum longitudinal reinforcement tension strain shall not exceed 0.05.

Where longitudinal reinforcement has embedment or development length into adjacent components that is insufficient for development of reinforcement strength—as in beams with bottom bars embedded a short distance into beam-column joints—flexural strength shall be calculated based on limiting stress capacity of the embedded bar as defined in Section 6.4.5.

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

6.4.4 Shear and Torsion

Strengths in shear and torsion shall be calculated according to ACI 318 (ACI, 1995), except as noted below and in Sections 6.5 and 6.9.

Within yielding regions of components with moderate or high ductility demands, shear and torsion strength shall be calculated according to accepted procedures for ductile components (for example, the provisions of Chapter 21 of *ACI 318-95*). Within yielding regions of components with low ductility demands, and outside yielding regions, shear strength may be calculated using accepted procedures normally used for elastic response (for example, the provisions of Chapter 11 of ACI 318).

Within yielding regions of components with moderate or high ductility demands, transverse reinforcement shall be assumed ineffective in resisting shear or torsion where: (1) longitudinal spacing of transverse reinforcement exceeds half the component effective depth measured in the direction of shear, or (2) perimeter hoops are either lap spliced or have hooks that are not adequately anchored in the concrete core.

Within yielding regions of components with low ductility demands, and outside yielding regions, transverse reinforcement shall be assumed ineffective in resisting shear or torsion where the longitudinal spacing of transverse reinforcement exceeds the component effective depth measured in the direction of shear.

Shear friction strength shall be calculated according to *ACI 318-95*, taking into consideration the expected axial load due to gravity and earthquake effects. Where rehabilitation involves addition of concrete requiring overhead work with dry-pack, the shear friction coefficient μ shall be taken as equal to 70% of the value specified by *ACI 318-95*.

6.4.5 Development and Splices of Reinforcement

Development strength of straight bars, hooked bars, and lap splices shall be calculated according to the general provisions of *ACI 318-95*, with the following modifications:

Within yielding regions of components with moderate or high ductility demands, details and strength provisions for new straight developed bars, hooked bars, and lap spliced bars shall be according to Chapter 21 of *ACI 318-95*. Within yielding regions of components with low ductility demands, and outside yielding regions, details and strength provisions for new construction shall be according to Chapter 12 of *ACI 318-95*, except requirements and strength provisions for lap splices may be taken as equal to those for straight development of bars in tension without consideration of lap splice classifications.

Where existing development, hook, and lap splice length and detailing requirements are not according to the requirements of the preceding paragraph, maximum stress capacity of reinforcement shall be calculated according to Equation 6-1.

$$f_s = \frac{l_b}{l_d} f_y \qquad (6\text{-}1)$$

where f_s = bar stress capacity for the development, hook, or lap splice length l_b provided; l_d = length required by Chapter 12 or Chapter 21 (as appropriate) of *ACI 318-95* for development, hook, or lap splice length, except splices may be assumed to be equivalent to straight bar development in tension; and f_y = yield strength of reinforcement. Where transverse reinforcement is distributed along the development length with spacing not exceeding one-third of the effective depth, the developed reinforcement may be assumed to retain the calculated stress capacity to large ductility levels. For larger spacings of transverse reinforcement, the developed stress shall be assumed to degrade from f_s to $0.2f_s$ at ductility demand or DCR equal to 2.0.

Strength of straight, discontinuous bars embedded in concrete sections (including beam-column joints) with clear cover over the embedded bar not less than $3d_b$ may be calculated according to Equation 6-2.

$$f_s = \frac{2500}{d_b} l_e \le f_y \qquad (6\text{-}2)$$

where f_s = maximum stress (in psi) that can be developed in an embedded bar having embedment length l_e (in inches), d_b = diameter of embedded bar (in inches), and f_y = bar yield stress (in psi). When the expected stress equals or exceeds f_s as calculated above, and f_s is less than f_y, the developed stress shall be assumed to degrade from f_s to $0.2f_s$ at 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 6-2, and modeling parameters and acceptance criteria shall be according to Section 6.5.2.

Doweled bars added in seismic rehabilitation may be assumed to develop yield stress when all the following are satisfied: (1) drilled holes for dowel bars are cleaned with a stiff brush that extends the length of the hole; (2) embedment length l_e is not less than $10d_b$; and (3) minimum spacing of dowel bars is not less than $4l_e$, and minimum edge distance is not less than $2l_e$. Other design values for dowel bars shall be verified by test data. Field samples shall be obtained to ensure design strengths are developed per Section 6.3.3.

6.4.6 Connections to Existing Concrete

Connections used to connect two or more components may be classified according to their anchoring systems as cast-in-place systems or as post-installed systems.

6.4.6.1 Cast-In-Place Systems

The capacity of the connection should be not less than 1.25 times the smaller of (1) the force corresponding to development of the minimum probable strength of the two interconnected components, and (2) the component actions at the connection. Shear forces, tension forces, bending moments, and prying actions shall be considered. Design values for connection anchorages shall be ultimate values, and shall be taken as suggested in ACI Report 355.1R-91, or as specified in the latest version of the locally adopted strength design building code.

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

6.4.6.2 Post-Installed Systems

The capacity should be calculated according to Section 6.4.6.1. See the *Commentary* for exceptions.

6.4.6.3 Quality Control

See *Commentary* for this section.

6.5 Concrete Moment Frames

6.5.1 Types of Concrete Moment Frames

Concrete moment frames are those elements composed primarily of horizontal framing components (beams and/or slabs) and vertical framing components (columns) that develop lateral load resistance through bending of horizontal and vertical framing components. These elements may act alone to resist lateral loads, or they may act in conjunction with shear walls, braced frames, or other elements to form a dual system.

The provisions in Section 6.5 are applicable to frames that are cast monolithically, including monolithic concrete frames rehabilitated or created by the addition of new material. Frames covered under this section include reinforced concrete beam-column moment frames, prestressed concrete beam-column moment frames, and slab-column moment frames. Sections 6.6, 6.7, and 6.10 apply to precast concrete frames, infilled concrete frames, and concrete braced frames, respectively.

6.5.1.1 Reinforced Concrete Beam-Column Moment Frames

Reinforced concrete beam-column moment frames are those frames that satisfy the following conditions:

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

The frames include Special Moment Frames, Intermediate Moment Frames, and Ordinary Moment Frames as defined in the 1994 *NEHRP Recommended Provisions* (BSSC, 1995), as well as frames not satisfying the requirements of these *Provisions*. This classification includes existing construction, new construction, and existing construction that has been rehabilitated.

6.5.1.2 Post-Tensioned Concrete Beam-Column Moment Frames

Post-tensioned concrete beam-column moment frames are those frames that satisfy the following conditions:

1. Framing components are beams (with or without slabs) and columns.
2. Beams and columns are of monolithic construction that provides for moment transfer between beams and columns.
3. Primary reinforcement in beams contributing to lateral load resistance includes post-tensioned reinforcement with or without nonprestressed reinforcement.

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

6.5.1.3 Slab-Column Moment Frames

Slab-column moment frames are those frames that satisfy the following conditions:

1. Framing components are slabs (with or without beams in the transverse direction) and columns.

2. Slabs and columns are of monolithic construction that provides for moment transfer between slabs and columns.

3. Primary reinforcement in slabs contributing to lateral load resistance includes nonprestressed reinforcement, prestressed reinforcement, or both.

The slab-column frame may or may not have been intended in the original design to be part of the lateral-load-resisting system. This classification includes existing construction, new construction, and existing construction that has been rehabilitated.

6.5.2 Reinforced Concrete Beam-Column Moment Frames

6.5.2.1 General Considerations

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

The analytical model generally can represent a beam-column frame using line elements with properties concentrated at component centerlines. Where beam and column centerlines do not coincide, the effects on framing shall be considered. Where minor eccentricities occur (i.e., the centerline of the narrower component falls within the middle third of the adjacent framing component measured transverse to the framing direction), the effect of the eccentricity can be ignored. Where larger eccentricities occur, the effect shall be represented either by reductions in effective stiffnesses, strengths, and deformation capacities, or by direct modeling of the eccentricity.

The beam-column joint in monolithic construction generally 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 may be assumed where the beam is wider than the column and where justified by experimental evidence. The model of the connection between the columns and foundation shall be selected based on the details of the column-foundation connection and rigidity of the foundation-soil system.

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

Inelastic deformations in primary components shall be restricted to flexure in beams (plus slabs, if present) and columns. Other inelastic deformations are permitted in secondary components. Acceptance criteria are provided in Section 6.5.2.4.

6.5.2.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

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

B. Nonlinear Static Procedure

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

Beams and columns may be modeled using concentrated plastic hinge models, distributed plastic hinge models, or other models whose behavior has been demonstrated to adequately represent important characteristics of reinforced concrete beam 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. Where nonlinear response is expected in a mode other than flexure, the model shall be established to represent these effects.

Monotonic load-deformation relations shall be according to the generalized relation shown in Figure 6-1, except that different relations are permitted where verified by experiments. In that figure, point B corresponds to significant yielding, C corresponds to the point where significant lateral load resistance can be assumed to be lost, and E corresponds to the point where gravity load resistance can be assumed to be lost.

The overall load-deformation relation shall be established so that the maximum resistance is consistent with the design strength specifications of Sections 6.4.2 and 6.5.2.3.

For beams and columns, the generalized deformation in Figure 6-1 may be either the chord rotation or the plastic hinge rotation. For beam-column joints, an acceptable measure of the generalized deformation is shear strain. Values of the generalized deformation at points B, C, and D may be derived from experiments or rational analyses, and shall take into account the interactions between flexure, axial load, and shear. Alternately, 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 6-6 and 6-7. Where the generalized deformation is shear distortion of the beam-column joint, shear angle capacities shall be as defined by Table 6-8.

C. Nonlinear Dynamic Procedure

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

6.5.2.3 Design Strengths

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

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

For columns, the contribution of concrete to shear strength, V_c, may be calculated according to Equation 6-3.

$$V_c = 3.5\lambda\left(k + \frac{N_u}{2000A_g}\right)\sqrt{f'_c}\, b_w d \qquad (6\text{-}3)$$

in which k = 1.0 in regions of low ductility demand and 0 in regions of moderate and high ductility demand, λ = 0.75 for lightweight aggregate concrete and 1.0 for normal weight aggregate concrete, and N_u = axial compression force in pounds (= 0 for tension force). All units are expressed in pounds and inches. Where axial force is calculated from the linear procedures of Chapter 3, compressive axial load for use in Equation 6-3 should be taken as equal to the value calculated considering design gravity load only, and tensile axial load should be taken as equal to the value calculated from the analysis considering design load combinations, including gravity and earthquake loading according to Section 3.2.8.

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

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

$$Q_{CL} = V_n = \lambda\gamma\sqrt{f'_c}\,A_j,\ psi \qquad (6\text{-}4)$$

in which λ = 0.75 for lightweight aggregate concrete and 1.0 for normal weight aggregate concrete, A_j is the effective horizontal joint area with dimension as defined above, and γ is as defined in Table 6-9.

6.5.2.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

All actions shall be classified as being either deformation-controlled or force-controlled, as defined in Chapter 3. 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 6-10 through 6-12.

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

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

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic region, closed stirrups are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (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 is permitted.

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 actions preferably 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

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

			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Component Type				
						Primary			Secondary	
			Plastic Rotation Angle, radians		Residual Strength Ratio	Performance Level				
Conditions			**a**	**b**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.015	0.03
≤ 0.1	C	≥ 6	0.015	0.025	0.2	0.005	0.01	0.015	0.01	0.025
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.0	0.005	0.015	0.010	0.025
≥ 0.4	C	≥ 6	0.01	0.015	0.2	0.0	0.005	0.01	0.01	0.015
≤ 0.1	NC	≤ 3	0.01	0.015	0.2	0.005	0.005	0.01	0.005	0.015
≤ 0.1	NC	≥ 6	0.005	0.005	–	0.005	0.005	0.005	0.005	0.005
≥ 0.4	NC	≤ 3	0.005	0.005	–	0.0	0.0	0.005	0.0	0.005
≥ 0.4	NC	≥ 6	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0
ii. Columns controlled by shear[1,3]										
Hoop spacing ≤ $d/2$, or $\frac{P}{A_g f'_c} \leq 0.1$			0.0	0.015	0.2	0.0	0.0	0.0	0.01	0.015
Other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]										
Hoop spacing ≤ $d/2$			0.01	0.02	0.4	1	1	1	0.01	0.02
Hoop spacing > $d/2$			0.0	0.01	0.2	1	1	1	0.005	0.01
iv. Columns with axial loads exceeding 0.70P_o[1,3]										
Conforming reinforcement over the entire length			0.015	0.025	0.02	0.0	0.005	0.001	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading “Transverse Reinforcement,” “C” and “NC” are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic hinge region, closed hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, hoops must not be lap spliced in the cover concrete, and hoops must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
4. Linear interpolation between values listed in the table is permitted.

Table 6-8 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beam-Column Joints

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

1. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A joint is conforming if closed hoops are spaced at ≤ $h_c/3$ within the joint. Otherwise, the component is considered nonconforming. Also, to qualify as conforming details under ii, hoops must not be lap spliced in the cover concrete, and must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
2. This is the ratio of the design axial force on the column above the joint to the product of the gross cross-sectional area of the joint and the concrete compressive strength. The design axial force is to be calculated using limit analysis procedures, as described in Chapter 3.
3. This is the ratio of the design shear force to the shear strength for the joint. The design shear force is to be calculated according to Section 6.5.2.3.
4. Linear interpolation between values listed in the table is permitted.

Table 6-9 Values of γ for Joint Strength Calculation

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

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

columns; (2) joint shears corresponding to development of strength in adjacent beams and/or 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 to determine which components develop their design strengths. Those components that satisfy Equations 3-18 and 3-19 may be assumed to satisfy the performance criteria for those components. Components that reach their design strengths shall be further evaluated according to Section 6.5.2.4A to determine performance acceptability.

Where the average DCR of columns at a level exceeds the average value of beams at the same level, and exceeds the greater of 1.0 and $m/2$ for columns, the element is 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 is repeated, considering all elements in the building system. If the average of the DCR values for vertical components exceeds the average value for horizontal components at the level, and exceeds 2.0, the structure shall be reanalyzed using a nonlinear procedure, or the structure shall be rehabilitated to remove this deficiency.

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

3. The structure shall be rehabilitated to remove this deficiency.

Calculated component actions shall satisfy the requirements of Chapter 3. Tables 6-10 through 6-12 present *m* values for use in Equation 3-18. Alternative approaches or values are permitted where justified by experimental evidence and analysis.

B. Nonlinear Static and Dynamic Procedures

Inelastic response shall be restricted to those components and actions listed in Tables 6-6 through 6-8, except where it is demonstrated that other inelastic action can be tolerated considering the selected Performance Levels.

Calculated component actions shall satisfy the requirements of Chapter 3. Maximum permissible inelastic deformations are listed in Tables 6-6 through 6-8. 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 are permitted where justified by experimental evidence and analysis.

6.5.2.5 Rehabilitation Measures

Rehabilitation measures include the following general approaches, plus other approaches based on rational procedures.

- **Jacketing existing beams, columns, or joints with new reinforced concrete, steel, or fiber wrap overlays**. The new materials shall be designed and constructed to act compositely with the existing concrete. Where reinforced concrete jackets are used, the design shall provide detailing to enhance ductility. Component strength shall be taken to not exceed any limiting strength of connections with adjacent components. Jackets designed to provide increased connection strength and improved continuity between adjacent components are permitted.

Table 6-10 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beams

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

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic region, closed stirrups are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (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 is permitted.

- **Post-tensioning existing beams, columns, or joints using external post-tensioned reinforcement.** Post-tensioned reinforcement shall be unbonded within a distance equal to twice the effective depth from sections where inelastic action is expected. Anchorages shall be located away from regions where inelastic action is anticipated, and shall be designed considering possible force variations due to earthquake loading.
- **Modification of the element by selective material removal from the existing element.** Examples include: (1) where nonstructural elements or components interfere with the frame, removing or separating the nonstructural elements or components to eliminate the interference; (2) weakening, usually by removal of concrete or severing of longitudinal reinforcement, to change response mode from a nonductile mode to a more ductile mode (e.g.,

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

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

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic hinge region, closed hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, hoops must not be lap spliced in the cover concrete, and must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
4. Linear interpolation between values listed in the table is permitted.

weakening of beams to promote formation of a strong-column, weak-beam system); and (3) segmenting walls to change stiffness and strength.

Table 6-12 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beam-Column Joints

Conditions			*m* factors[4]				
			Component Type				
			Primary[5]			Secondary	
			Performance Level				
			IO	LS	CP	LS	CP
i. Interior joints							
$\frac{P}{A_g f'_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]					
≤ 0.1	C	≤ 1.2	–	–	–	3	4
≤ 0.1	C	≥ 1.5	–	–	–	2	3
≥ 0.4	C	≤ 1.2	–	–	–	3	4
≥ 0.4	C	≥ 1.5	–	–	–	2	3
≤ 0.1	NC	≤ 1.2	–	–	–	2	3
≤ 0.1	NC	≥ 1.5	–	–	–	2	3
≥ 0.4	NC	≤ 1.2	–	–	–	2	3
≥ 0.4	NC	≥ 1.5	–	–	–	2	3
ii. Other joints							
$\frac{P}{A_g f'_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]					
≤ 0.1	C	≤ 1.2	–	–	–	3	4
≤ 0.1	C	≥ 1.5	–	–	–	2	3
≥ 0.4	C	≤ 1.2	–	–	–	3	4
≥ 0.4	C	≥ 1.5	–	–	–	2	3
≤ 0.1	NC	≤ 1.2	–	–	–	2	3
≤ 0.1	NC	≥ 1.5	–	–	–	2	3
≥ 0.4	NC	≤ 1.2	–	–	–	1	1
≥ 0.4	NC	≥ 1.5	–	–	–	1	1

1. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A joint is conforming if closed hoops are spaced at ≤ $h_c/3$ within the joint. Otherwise, the component is considered nonconforming. Also, to qualify as conforming details under ii, hoops must not be lap spliced in the cover concrete, and must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
2. This is the ratio of the design axial force on the column above the joint to the product of the gross cross-sectional area of the joint and the concrete compressive strength. The design axial force is to be calculated using limit analysis procedures as described in Chapter 3.
3. This is the ratio of the design shear force to the shear strength for the joint. The design shear force is to be calculated according to Section 6.5.2.3.
4. Linear interpolation between values listed in the table is permitted.
5. All interior joints are force-controlled, and no *m* factors apply.

- **Improvement of deficient existing reinforcement details.** This approach involves removal of cover concrete, modification of existing reinforcement details, and casting of new cover concrete. Concrete removal shall avoid unintended damage to core concrete and the bond between existing reinforcement and core concrete. New cover concrete shall be designed and constructed to achieve fully composite action with the existing materials.

- **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.

- **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 shall be designed to transfer the forces anticipated for the design load combinations. Where the existing concrete frame columns and beams act as boundary elements and collectors for the new shear wall or braced frame, these shall be checked for adequacy, considering strength, reinforcement development, and deformability. Diaphragms, including drag struts and collectors, shall be evaluated and, if necessary, rehabilitated to ensure a complete load path to the new shear wall or braced frame element.

Rehabilitated frames shall be evaluated according to the general principles and requirements of this chapter. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in an analytical model of the rehabilitated structure. Connections required between existing and new elements shall satisfy the requirements of Section 6.4.6 and other requirements of the *Guidelines*. An existing frame rehabilitated according to procedures listed above shall satisfy the relevant specific requirements of Chapter 6.

6.5.3 Post-Tensioned Concrete Beam-Column Moment Frames

6.5.3.1 General Considerations

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

The linear procedures and the NSP described in Chapter 3 apply directly to frames with post-tensioned beams in which the following conditions are satisfied:

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 350 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 have been demonstrated to perform satisfactorily for seismic loadings. These anchorages must occur outside hinging areas or joints.

Alternative procedures are required where these conditions are not satisfied.

6.5.3.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

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

B. Nonlinear Static Procedure

Nonlinear load-deformation relations shall follow the general guidelines of Section 6.4.1.2 and the reinforced concrete frame guidelines of Section 6.5.2.2B.

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

C. Nonlinear Dynamic Procedure

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

6.5.3.3 Design Strengths

Component strengths shall be computed according to the general requirements of Section 6.4.2 and the additional requirements of Section 6.5.2.3. Effects of prestressing on strength shall be considered. For deformation-controlled actions, prestress shall be assumed to be effective for the purpose of determining the maximum actions that may be developed associated with nonlinear response of the frame. For force-controlled actions, the effects on strength of prestress loss shall also be considered as a design condition, where these losses are possible under design load combinations including inelastic deformation reversals.

6.5.3.4 Acceptance Criteria

Acceptance criteria shall follow the criteria for reinforced concrete beam-column frames, as specified in Section 6.5.2.4.

Tables 6-6, 6-7, 6-8, 6-10, 6-11, and 6-12 present acceptability values for use in the four procedures of Chapter 3. The values in these tables for beams apply only if the beams satisfy the three conditions of Section 6.5.3.1.

6.5.3.5 Rehabilitation Measures

Rehabilitation measures include the general approaches listed in Section 6.5.2.5, as well as other approaches based on rational procedures.

Rehabilitated frames shall be evaluated according to the general principles and requirements of this chapter. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in an analytical model of the rehabilitated building. Connections required between existing and new elements shall satisfy the requirements of Section 6.4.6 and other requirements of the *Guidelines*.

6.5.4 Slab-Column Moment Frames

6.5.4.1 General Considerations

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

The analytical model can represent the slab-column frame, using 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). Three approaches are specifically recognized.

- **Effective beam width model.** Columns and slabs are represented by frame elements that are rigidly interconnected at the slab-column joint.
- **Equivalent frame model.** Columns and slabs are represented by frame elements that are interconnected by connection springs.
- **Finite element model.** The columns are represented by frame 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 model of the connection between the columns and foundation shall be selected based on the details of the column-foundation connection and rigidity of the foundation-soil system.

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

Inelastic deformations in primary components shall be restricted to flexure in slabs and columns, plus limited nonlinear response in slab-column connections. Other inelastic deformations are permitted in secondary components. Acceptance criteria are in Section 6.5.4.4.

6.5.4.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

Slabs shall be modeled considering flexural, shear, and tension (in the slab adjacent to the column) stiffnesses. Columns shall be modeled considering flexural, shear, and axial stiffnesses. Joints shall be modeled as stiff components, and may in most cases be considered rigid. The effective stiffnesses of components shall be adjusted on the basis of experimental evidence to represent effective stiffnesses according to the general principles of Section 6.4.1.2.

B. Nonlinear Static Procedure

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

Slabs and columns may be modeled using concentrated plastic hinge models, distributed plastic hinge models, or other models whose behavior has been demonstrated to adequately represent important characteristics 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 preferably will be modeled separately from the slab and column components, so that potential failure in shear and moment transfer can be identified. Where nonlinear response is expected in a mode other than flexure, the model shall be established to represent these effects.

Monotonic load-deformation relations shall be according to the generalized relation shown in Figure 6-1, with definitions according to Section 6.5.2.2B. The overall load-deformation relation shall be established so that the maximum resistance is consistent with the design strength specifications of Sections 6.4.2 and 6.5.4.3. Where the generalized deformation shown in Figure 6-1 is taken as the flexural plastic hinge rotation for the column, the plastic hinge rotation capacities shall be as defined by Table 6-7. Where the generalized deformation shown in Figure 6-1 is taken as the rotation of the slab-column connection, the plastic rotation capacities shall be as defined by Table 6-13.

C. Nonlinear Dynamic Procedure

The general approach shall be according to the specification of Section 6.5.2.2C.

6.5.4.3 Design Strengths

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

The maximum component strength shall be determined considering potential failure in flexure, axial load, shear, torsion, development, and other actions at all points along the length of the component under the actions of design gravity and lateral load combinations. The strength of slab-column connections shall also 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} is to be calculated according to the procedures of *ACI 318-95* (ACI, 1995) for the design gravity load specified in Chapter 3.

For columns, the shear strength may be evaluated according to Section 6.5.2.3.

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. An acceptable procedure is to calculate the shear and moment transfer strength as described below.

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 may be taken as equal to the minimum of two strengths: (1) the strength calculated considering eccentricity of shear on a slab critical section due to combined shear and moment, as prescribed in *ACI 318-95*; and (2) the moment transfer strength equal to $\Sigma M_n/\gamma_f$, where ΣM_n = the sum of positive and negative flexural strengths of a section of slab between lines that are two and one-half slab or drop panel thicknesses ($2.5h$) outside opposite faces of the column or capital; γ_f = the fraction of the moment resisted by flexure per *ACI 318-95*; 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

Table 6-13 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Two-Way Slabs and Slab-Column Connections

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

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

does not exceed 0.5 V_c, the moment transfer strength may 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 318-95*.

6.5.4.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

All component actions shall be classified as being either deformation-controlled or force-controlled, as defined in Chapter 3. 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 6-14. 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 actions preferably 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

Table 6-14 ***Numerical Acceptance Criteria for Linear Procedures—Two-Way Slabs and Slab-Column Connections***

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

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

likely plastic action in components above the level in question.

Design actions shall be compared with design strengths to determine which components develop their design strengths. Those components that do not reach their design strengths may be assumed to satisfy the performance criteria for those components. Components that reach their design strengths shall be further evaluated according to this section to determine performance acceptability.

Where the average of the DCRs of columns at a level exceeds the average value of slabs at the same level, and exceeds the greater of 1.0 and $m/2$, the element is defined as a weak story element. In this case, follow the procedure for weak story elements described in Section 6.5.2.4A.

Calculated component actions shall satisfy the requirements of Chapter 3. Tables 6-11 and 6-14 present *m* values.

B. Nonlinear Static and Dynamic Procedures

Inelastic response shall be restricted to those components and actions listed in Tables 6-7 and 6-13, except where it is demonstrated that other inelastic action can be tolerated considering the selected Performance Levels.

Calculated component actions shall satisfy the requirements of Chapter 3. Maximum permissible inelastic deformations are listed in Tables 6-7 and 6-13. 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 are permitted where justified by experimental evidence and analysis.

6.5.4.5 Rehabilitation Measures

Rehabilitation measures include the general approaches listed in Section 6.5.2.5, plus other approaches based on rational procedures.

Rehabilitated frames shall be evaluated according to the general principles and requirements of this chapter. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in an analytical model of the rehabilitated building. Connections required between existing and new elements shall satisfy requirements of Section 6.4.6 and other requirements of the *Guidelines*.

6.6 Precast Concrete Frames

6.6.1 Types of Precast Concrete Frames

Precast concrete frames are those elements that are constructed from individually made beams and columns, that are assembled to create gravity-load-carrying systems. These systems are sometimes expected to directly resist lateral loads, and are always required to deform in a manner that is compatible with the structure as a whole.

The provisions of this section are applicable to precast concrete frames that emulate cast-in-place moment frames, precast concrete beam-column moment frames other than emulated cast-in-place moment frames, and precast concrete frames not expected to directly resist lateral loads.

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

Emulated moment frames of precast concrete are those precast beam-column systems that are interconnected using reinforcing and wet concrete in such a way as to create a system that will act to resist lateral loads in a manner similar to cast-in-place concrete systems. These systems are recognized and accepted by the 1994 *NEHRP Recommended Provisions* (BSSC, 1995), and are based on ACI 318, which requires safety and serviceability levels expected from monolithic construction. There are insufficient research and testing data at this time to qualify systems assembled using dry joints as emulated moment frames.

6.6.1.2 Precast Concrete Beam-Column Moment Frames other than Emulated Cast-in-Place Moment Frames

Frames of this classification are assembled using dry joints; that is, connections are made by bolting, welding, post-tensioning, or other similar means. Frames of this nature may act alone to resist lateral loads, or they may act in conjunction with shear walls, braced frames, or other elements to form a dual system. The appendix to Chapter 6 of the 1994 *NEHRP Recommended Provisions* (BSSC, 1995) contains a trial version of code provisions for new construction of this nature, but it was felt to be premature in 1994 to base actual provisions on the material in the appendix.

6.6.1.3 Precast Concrete Frames Not Expected to Resist Lateral Loads Directly

Frames of this classification are assembled using dry joints similar to those of Section 6.6.1.2, but are not expected to participate in resisting the lateral loads directly or significantly. Shear walls, braced frames, or steel moment frames are expected to provide the entire lateral load resistance, but the precast concrete "gravity" frame system must be able to deform in a manner that is compatible with the structure as a whole. Conservative assumptions shall be made concerning the relative fixity of joints.

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

6.6.2.1 General Considerations

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

6.6.2.2 Stiffness for Analysis

Stiffness for analysis shall be as defined in Section 6.5.2.2. The effects of prestressing shall be considered when computing the effective stiffness values using Table 6-4.

6.6.2.3 Design Strengths

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

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 that the moment connections may have been constructed after dead load had been applied to portions of the structure

3. Effects of restraint that may be present due to interaction with interconnected wall or brace components

6.6.2.4 Acceptance Criteria

Acceptance criteria for precast concrete frames that emulate cast-in-place moment frames are as described in Section 6.5.2.4, except that the factors defined in Section 6.6.2.3 shall also be considered.

6.6.2.5 Rehabilitation Measures

Rehabilitation measures for emulated cast-in-place moment frames are given in Section 6.5.2.5. Special consideration shall be given to the presence of prestressing strand when installing new elements and when adding new rigid elements to the existing system.

6.6.3 Precast Concrete Beam-Column Moment Frames other than Emulated Cast-in-Place Moment Frames

6.6.3.1 General Considerations

The analysis model for precast concrete beam-column moment frames other than emulated moment frames shall be established following Section 6.5.2.1 for reinforced concrete beam-column moment frames, with additional consideration of the special nature of the dry joints used in assembling the precast system. The requirements given in the appendix to Chapter 6 of the 1994 *NEHRP Recommended Provisions* for this type of structural system should be adhered to where possible, and the philosophy and approach should be employed when designing new connections for existing components. See also Section 6.4.6.

6.6.3.2 Stiffness for Analysis

Stiffness for analysis shall be as defined in Sections 6.5.2.2 and 6.6.2.2. Flexibilities associated with connections should be included in the analytical model. See also Section 6.4.6.

6.6.3.3 Design Strengths

Component strength shall be computed according to the requirements of Sections 6.5.2.3 and 6.6.2.3, with the additional requirements that the connections comply with the appendix to Chapter 6 of the 1994 *NEHRP Recommended Provisions*, and connection strength shall be represented. See also Section 6.4.6.

6.6.3.4 Acceptance Criteria

Acceptance criteria for precast concrete beam-column moment frames other than emulated cast-in-place moment frames are given in Sections 6.5.2.4 and 6.6.2.4, with the additional requirement that the connections meet the requirements of Section 6.A.4 of the appendix to Chapter 6 of the 1994 *NEHRP Recommended Provisions*. See also Section 6.4.6.

6.6.3.5 Rehabilitation Measures

Rehabilitation measures for the frames of this section shall meet the requirements of Section 6.6.2.5. Special consideration shall be given to connections that are stressed beyond their elastic limit.

6.6.4 Precast Concrete Frames Not Expected to Resist Lateral Loads Directly

6.6.4.1 General Considerations

The analysis model for precast concrete frames that are not expected to resist significant lateral loads directly shall include the effects of deformations that the lateral-load-resisting system will experience. The general considerations of Sections 6.5.2.1 and 6.6.3.1 shall be included.

6.6.4.2 Stiffness for Analysis

The stiffness for analysis considers 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 practically no situations where the precast column can be considered to be completely pinned top and bottom, and as a consequence, not resisting any shear from building drift. Several parking structures collapsed as a result of this defect. Conservative assumptions should be made.

6.6.4.3 Design Strengths

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

6.6.4.4 Acceptance Criteria

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

6.6.4.5 Rehabilitation Measures

Rehabilitation measures for the frames discussed in this section shall meet the requirements of Section 6.6.3.5.

6.7 Concrete Frames with Infills

6.7.1 Types of Concrete Frames with Infills

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

Infills may be considered to be isolated infills if they are isolated from the surrounding frame according to the minimum gap requirements described in Section 7.5.1. If all infills in a frame are isolated infills, the frame should 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 are applicable to frames with existing infills, frames that are rehabilitated by addition or removal of material, and concrete frames that are rehabilitated by the addition of new infills.

6.7.1.1 Types of Frames

The provisions are applicable to frames that are cast monolithically and frames that are precast. Types of concrete frames are described in Sections 6.5, 6.6, and 6.10.

6.7.1.2 Masonry Infills

Types of masonry infills are described in Chapter 7.

6.7.1.3 Concrete Infills

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

6.7.2 Concrete Frames with Masonry Infills

6.7.2.1 General Considerations

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

Behavior of a concrete frame with masonry infill resisting lateral forces within its plane may be calculated based on linear elastic behavior if it can be demonstrated that the wall will not crack when subjected to design lateral forces. In this case, the assemblage of frame and infill should be considered to be a homogeneous medium for stiffness computations.

Behavior of cracked concrete frames with masonry infills may be represented by a diagonally braced frame model in which the columns act as vertical chords, the beams act as horizontal ties, and the infill is modeled using the equivalent compression strut analogy. Requirements for the equivalent compression strut analogy are described 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 in the noninfilled portion of the bay.

In frames having infills in some bays and no infill in other bays, the restraint of the infill shall be represented as described above, and the noninfilled bays shall be modeled as frames according to the specifications of this chapter. Where infills create a discontinuous wall, the effects on overall building performance shall be considered.

6.7.2.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

General aspects of modeling are described in Section 6.7.2.1. Beams and columns in infilled portions may be modeled considering axial tension and compression flexibilities only. Noninfilled portions shall be modeled according to procedures described for noninfilled frames. Effective stiffnesses shall be according to Section 6.4.1.2.

B. Nonlinear Static Procedure

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

Beams and columns in infilled portions may be modeled using nonlinear truss elements. Beams and columns in noninfilled portions may be modeled using procedures described in this chapter. The model shall be capable of representing inelastic response along the component lengths.

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

1. For beams and columns in noninfilled portions of frames, where the generalized deformation is taken as rotation in the flexural plastic hinge zone, the plastic hinge rotation capacities shall be as defined by Table 6-17.

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 6-15.

C. Nonlinear Dynamic Procedure

For the NDP, the complete hysteresis behavior of each component shall be modeled using properties verified by tests. Unloading and reloading properties shall represent significant stiffness and strength degradation characteristics.

6.7.2.3 Design Strengths

Strengths of reinforced concrete components shall be according to the general requirements of Section 6.4.2, as modified by other specifications of this chapter. Strengths of masonry infills shall be according to the requirements of Chapter 7. Strengths shall consider limitations imposed by beams, columns, and joints in unfilled portions of frames; tensile and compressive capacity of columns acting as boundary elements of infilled frames; local forces applied from the infill to the frame; strength of the infill; and connections with adjacent elements. .

Table 6-15 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Infilled Frames

<table>
<tr><th rowspan="6">Conditions</th><th colspan="3">Modeling Parameters[4]</th><th colspan="5">Acceptance Criteria</th></tr>
<tr><th colspan="2" rowspan="4">Total Strain</th><th rowspan="4">Residual Strength Ratio</th><th colspan="5">Total Strain</th></tr>
<tr><th colspan="5">Component Type</th></tr>
<tr><th colspan="3">Primary</th><th colspan="2">Secondary</th></tr>
<tr><th colspan="5">Performance Level</th></tr>
<tr><th>d</th><th>e</th><th>c</th><th>IO</th><th>LS</th><th>CP</th><th>LS</th><th>CP</th></tr>
<tr><td colspan="9">i. Columns modeled as compression chords[3]</td></tr>
<tr><td>Columns confined along entire length[2]</td><td>0.02</td><td>0.04</td><td>0.4</td><td>0.003</td><td>0.015</td><td>0.020</td><td>0.03</td><td>0.04</td></tr>
<tr><td>All other cases</td><td>0.003</td><td>0.01</td><td>0.2</td><td>0.002</td><td>0.002</td><td>0.003</td><td>0.01</td><td>0.01</td></tr>
<tr><td colspan="9">ii. Columns modeled as tension chords[3]</td></tr>
<tr><td>Columns with well-confined splices, or no splices</td><td>0.05</td><td>0.05</td><td>0.0</td><td>0.01</td><td>0.03</td><td>0.04</td><td>0.04</td><td>0.05</td></tr>
<tr><td>All other cases</td><td>See note 1</td><td>0.03</td><td>0.2</td><td colspan="3">See note 1</td><td>0.02</td><td>0.03</td></tr>
</table>

1. Splice failure in a primary component can result in loss of lateral load resistance. For these cases, refer to the generalized procedure of Section 6.4.2. For primary actions, Collapse Prevention Performance Level shall be defined as the deformation at which strength degradation begins. Life Safety Performance Level shall be taken as three-quarters of that value.

2 A column may be considered to be confined along its entire length when the quantity of transverse reinforcement along the entire story height including the joint is equal to three-quarters of that required by ACI 318 for boundary elements of concrete shear walls. The maximum longitudinal spacing of sets of hoops shall not exceed $h/3$ nor $8d_b$.

3. In most infilled walls, load reversals will result in both conditions i and ii applying to a single column, but for different loading directions.

4. Interpolation is not permitted.

6.7.2.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

All component actions shall be classified as either deformation-controlled or force-controlled, as defined in Chapter 3. 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 this chapter and for the masonry infill in Chapter 7.

Design actions shall be determined as prescribed in Chapter 3. Where calculated DCR values exceed unity, the following actions preferably 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 to determine which components develop their design strengths. Those components that have design actions less than design strengths may be assumed to satisfy the performance criteria for those components. Components that reach their design strengths shall be further evaluated according to this section to determine performance acceptability.

Calculated component actions shall satisfy the requirements of Chapter 3. Refer to Section 7.5.2.2 for *m* values for masonry infills. Refer to other sections of this chapter for *m* values for concrete frames; *m* values for columns modeled as tension and compression chords are in Table 6-16.

Table 6-16 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Infilled Frames

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

1. A column may be considered to be confined along its entire length when the quantity of transverse reinforcement along the entire story height including the joint is equal to three-quarters of that required by ACI 318 for boundary elements of concrete shear walls. The maximum longitudinal spacing of sets of hoops shall not exceed $h/3$ nor $8d_b$.
2. In most infilled walls, load reversals will result in both conditions i and ii applying to a single column, but for different loading directions.
3. Interpolation is not permitted.

B. Nonlinear Static and Dynamic Procedures

Inelastic response shall be restricted to those components and actions that are permitted for isolated frames in this chapter and for masonry infills in Chapter 7.

Design actions shall be compared with design strengths to determine which components develop their design strengths. Those components that have design actions less than design strengths may be assumed to satisfy the performance criteria for those components. Components that reach their design strengths shall be further evaluated, according to Section 6.5.2.4B, to determine performance acceptability.

Calculated component actions shall not exceed the numerical values listed in Table 6-15, the relevant tables for isolated frames given in this chapter, and the relevant tables for masonry infills given in Chapter 7. Where inelastic action is indicated for a component or action not listed in Tables 6-10 through 6-12, the performance shall be deemed unacceptable. Alternative approaches or values are permitted where justified by experimental evidence and analysis.

6.7.2.5 Rehabilitation Measures

Rehabilitation measures include the general approaches listed for isolated frames in this chapter, the measures listed for masonry infills in Section 7.5, and other approaches based on rational procedures. Both in-plane and out-of-plane loading shall be considered.

The following methods should be considered.

- **Jacketing existing beams, columns, or joints with new reinforced concrete, steel, or fiber wrap overlays.** The new materials shall be designed and constructed to act compositely with the existing concrete. Where reinforced concrete jackets are used, the design shall provide detailing to enhance ductility. Component strength shall be taken to not exceed any limiting strength of connections with adjacent components. Jackets designed to provide increased connection strength and improved continuity between adjacent components are permitted.

- **Post-tensioning existing beams, columns, or joints using external post-tensioned reinforcement.** Vertical post-tensioning may be useful for increasing tensile capacity of columns acting as boundary zones. Anchorages shall be located away from regions where inelastic action is anticipated, and shall be designed considering possible force variations due to earthquake loading.

- **Modification of the element by selective material removal from the existing element.** Either the infill can be completely removed from the frame, or gaps can be provided between the frame and the infill. In the latter case, the gap requirements of Chapter 7 shall be satisfied.

- **Improvement of deficient existing reinforcement details.** This approach involves removal of cover concrete, modification of existing reinforcement details, and casting of new cover concrete. Concrete removal shall avoid unintended damage to core concrete and the bond between existing reinforcement and core concrete. New cover concrete shall be designed and constructed to achieve fully composite action with the existing materials.

- **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.

Rehabilitated frames shall be evaluated according to the general principles and requirements of this chapter. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in the analytical model. Connections required between existing and new elements shall satisfy the requirements of Section 6.4.6 and other requirements of the *Guidelines*.

6.7.3 Concrete Frames with Concrete Infills

6.7.3.1 General Considerations

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

The numerical model should 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, it may be suitable to model the infilled frame as a solid shear wall, although openings should be considered where they occur. In other cases, it may be more suitable to model the frame-infill system using a braced-frame analogy such as that described for concrete frames with masonry infills in Section 6.7.2. Some judgment is necessary to determine the appropriate type and complexity of the analytical model.

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 in the noninfilled 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 above, and the noninfilled bays shall be modeled as frames according to the specifications of this chapter. Where infills create a discontinuous wall, the effects on overall building performance shall be considered.

6.7.3.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

General aspects of modeling are described in Section 6.7.3.1. Effective stiffnesses shall be according to the general principles of Section 6.4.1.2.

B. Nonlinear Static Procedure

General aspects of modeling are described in Section 6.7.3.1. Nonlinear load-deformation relations shall follow the general guidelines of Section 6.4.1.2.

Monotonic load-deformation relations shall be according to the generalized relation shown in Figure 6-1, except different relations are permitted where verified by tests. Numerical quantities in Figure 6-1 may be derived from tests or rational analyses following the general guidelines of Section 2.13, and shall take into account the interactions between frame and infill components. The guidelines of Section 6.7.2.2 may be used to guide development of modeling parameters for concrete frames with concrete infills.

C. Nonlinear Dynamic Procedure

For the NDP, the complete hysteresis behavior of each component shall be modeled using properties verified by tests. Unloading and reloading properties shall represent significant stiffness and strength degradation characteristics.

6.7.3.3 Design Strengths

Strengths of reinforced concrete components shall be according to the general requirements of Section 6.4.2, as modified by other specifications of this chapter. Strengths shall consider limitations imposed by beams, columns, and joints in unfilled portions of frames; tensile and compressive capacity of columns acting as boundary elements of infilled frames; local forces applied from the infill to the frame; strength of the infill; and connections with adjacent elements.

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

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

6.7.3.4 Acceptance Criteria

The acceptance criteria for concrete frames with concrete infills should be guided by relevant acceptance criteria of Sections 6.7.2.4, 6.8, and 6.9.

6.7.3.5 Rehabilitation Measures

Rehabilitation measures include the general approaches listed for masonry infilled frames in Section 6.7.2.5.

Strengthening of the existing infill may be considered as an option for rehabilitation. Shotcrete can be applied to the face of an existing wall to increase the thickness and shear strength. 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.

Rehabilitated frames shall be evaluated according to the general principles and requirements of this chapter. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in the analytical model. Connections required between existing and new elements shall satisfy the requirements of Section 6.4.6 and other requirements of the *Guidelines*.

6.8 Concrete Shear Walls

6.8.1 Types of Concrete Shear Walls and Associated Components

Concrete shear walls consist of planar vertical elements that normally serve as the primary lateral-load-resisting elements when they are used in concrete structures. In general, shear walls (or wall segments) are considered to be slender if their aspect ratio (height/length) is Š 3.0, and they are considered to be short 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.

The provisions given here are applicable to all shear walls in all types of structural systems that incorporate shear walls. This includes isolated shear walls, shear walls used in dual (wall-frame) systems, coupled shear walls, and discontinuous shear walls. Shear walls are considered to be solid walls if they have small openings that do not significantly influence the strength or inelastic behavior of the wall. Perforated shear walls are characterized by a regular pattern of large openings in both horizontal and vertical directions that create a series of pier and deep beam elements. In the discussions and tables that appear in the following sections, these vertical piers and horizontal beams will both be referred to as wall segments.

Provisions are also included for coupling beams and columns that support discontinuous shear walls. These are special frame components that are associated more with shear walls than with the normal frame elements covered in Section 6.5.

6.8.1.1 Monolithic Reinforced Concrete Shear Walls and Wall Segments

Monolithic reinforced concrete (RC) shear walls consist of vertical cast-in-place elements, usually with a constant cross section, that typically form open or closed shapes around vertical building shafts. Shear walls are also used frequently along portions of the perimeter of the building. 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. 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.

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

6.8.1.2 Reinforced Concrete Columns Supporting Discontinuous Shear Walls

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

6.8.1.3 Reinforced Concrete Coupling Beams

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

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

6.8.2.1 General Modeling Considerations

The analysis model for an RC shear wall element shall be sufficiently detailed to represent the stiffness, strength, and deformation capacity of the overall shear wall. Potential failure in flexure, shear, and reinforcement development at any point in the shear wall shall be considered. Interaction with other structural and nonstructural elements shall be included.

In most cases, shear walls and wall elements may be modeled analytically 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 will need to be long enough to properly represent the distance from the wall centroid—where the beam-column element is placed in the computer model—to the edge of the wall. Unsymmetrical wall sections shall model the different bending capacities for the two loading directions.

For rectangular shear walls and wall segments with $h_w / l_w \leq 2.5$, and flanged wall sections with $h_w / l_w \leq 3.5$, shear deformations become more significant. For such cases, either a modified beam-column analogy or a multiple-node, multiple-spring approach should be used (references are given in the *Commentary*). Because shear walls usually respond in single curvature over a story height, the use of one multiple-spring element per story is recommended for modeling shear walls. For wall segments, which typically deform into a double curvature pattern, the beam-column element is usually preferred. If a multiple-spring model is used for a wall segment, then it is recommended that two elements be used over the length of the wall segment.

A beam element that incorporates both bending and shear deformations shall be used to model coupling beams. It is recommended that the element inelastic response should account for the loss of shear strength and stiffness during reversed cyclic loading to large deformations. Coupling beams that have diagonal reinforcement satisfying the 1994 *NEHRP Recommended Provisions* (BSSC, 1995) 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.

Columns supporting discontinuous shear walls may be modeled with beam-column elements typically used in frame analysis. This element should also account for shear deformations, and care must be taken to ensure that the model properly reflects the potentially rapid reduction in shear stiffness and strength these columns may experience after the onset of flexural yielding.

The diaphragm action of concrete slabs that interconnect shear walls and frame columns shall be properly represented.

6.8.2.2 Stiffness for Analysis

The stiffness of all the elements discussed in this section depends on the material properties, component dimensions, reinforcement quantities, boundary conditions, and current state of the member with respect to cracking and stress levels. All of these aspects should be considered when defining the effective stiffness of an element. General values for effective stiffness are given in Table 6-4. 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 should be considered to be cracked.

For coupling beams, the values given in Table 6-4 for nonprestressed beams should be used. Columns supporting discontinuous shear walls will experience significant changes in axial load during lateral loading of the shear wall they support. Thus, the stiffness values for these column elements will need to 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.

A. 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 on each side of the web shall be taken as the smaller of: (1) one-fifth of the wall height, (2) half the distance to the next web, or (3) the provided width of the flange. The calculated stiffnesses to be used in analysis shall be in accordance with the requirements of Section 6.4.1.2.

Joints between shear walls and frame elements shall be modeled as stiff components and shall be considered rigid in most cases.

B. Nonlinear Static and Dynamic Procedures

Nonlinear load-deformation relations shall follow the general procedures described in Section 6.4.1.2.

Monotonic load-deformation relationships for analytical models that represent shear walls, wall elements, coupling beams, and RC columns that support discontinuous shear walls shall be of the general shapes defined in Figure 6-1. For both of the load-deformation relationships in Figure 6-1, point *B* corresponds to significant yielding, point *C* corresponds to the point where significant lateral resistance is assumed to be lost, and point *E* corresponds to the point where gravity load resistance is assumed to be lost.

The load-deformation relationship in Figure 6-1(a) should be referred to 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. For all of these members, the x-axis of Figure 6-1(a) should be taken as the rotation over the plastic hinging region at the end of the member (Figure 6-2). The hinge rotation at point *B* corresponds to the yield point, θ_y, and is given by the following expression:

$$\theta_y = \left(\frac{M_y}{E_c I}\right) l_p \tag{6-5}$$

where:

- M_y = Yield moment capacity of the shear wall or wall segment
- E_c = Concrete modulus
- I = Member moment of inertia, as discussed above
- 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 RC columns supporting discontinuous shear walls, l_p shall be set equal to 0.5 times the flexural depth of the component.

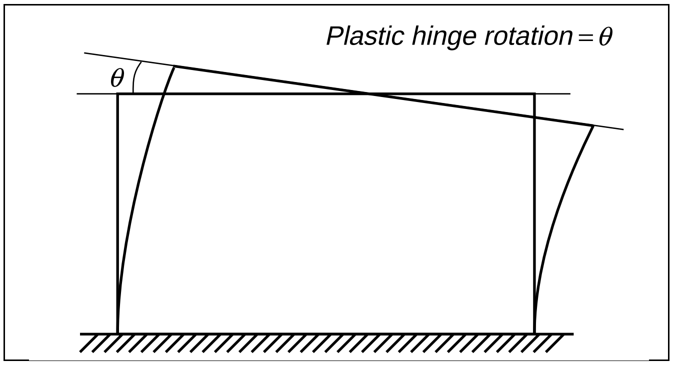

Figure 6-2 ***Plastic Hinge Rotation in Shear Wall where Flexure Dominates Inelastic Response***

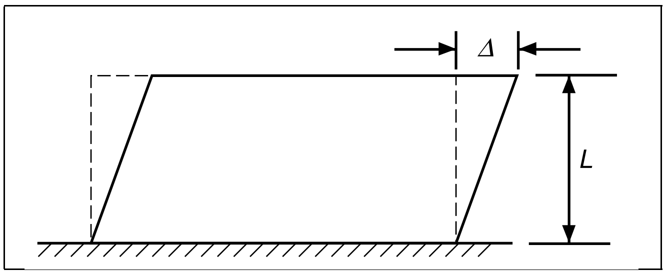

Figure 6-3 ***Story Drift in Shear Wall where Shear Dominates Inelastic Response***

Values for the variables *a*, *b*, and *c*, which are required to define the location of points *C*, *D*, and *E* in Figure 6-1(a), are given in Table 6-17.

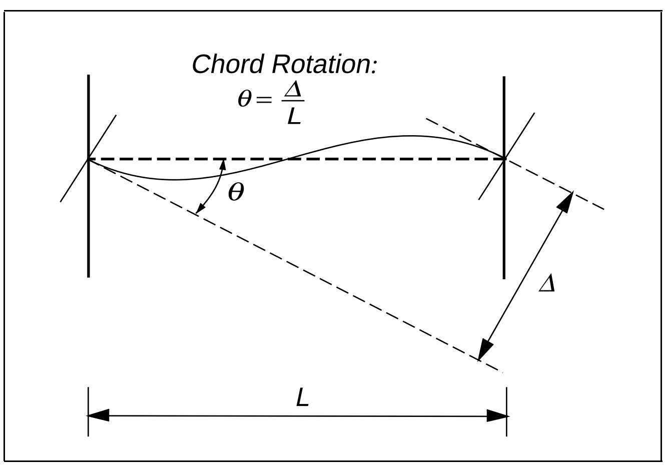

Figure 6-4 ***Chord Rotation for Shear Wall Coupling Beams***

For shear walls and wall segments whose inelastic response is controlled by shear, it is more appropriate to use drift as the deformation value in Figure 6-1(b). For shear walls, this drift is actually the story drift as shown in Figure 6-3. For wall segments, Figure 6-3 essentially represents the member drift.

For coupling beams, the deformation measure to be used in Figure 6-1(b) is the chord rotation for the member, as defined in Figure 6-4. Chord rotation is the most representative measure of the deformed state of a coupling beam, whether its inelastic response is governed by flexure or by shear.

Values for the variables *d*, *e*, and *c*, which are required to find the points *C*, *D*, and *E* in Figure 6-1(b), are given in Tables 6-17 and 6-18 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.

For the NDP, the complete hysteresis behavior of each component shall be modeled using properties verified by experimental evidence. The relationships in Figure 6-1 may 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 Design Strengths

The discussions in the following paragraphs shall apply to shear walls, wall segments, coupling beams, and columns supporting discontinuous shear walls. In general, component strengths shall be computed according to the general requirements of Section 6.4.2, except as modified here. The yield and maximum component 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 shall be determined using the fundamental principles given in Chapter 10 of *Building Code Requirements for Structural Concrete, ACI 318-95* (ACI, 1995). For calculation of nominal flexural strength, the effective compression and tension flange widths defined in Section 6.8.2.2A shall be used, except that the first limit shall be changed to one-tenth of the wall height. When determining the flexural yield strength of a shear wall, as represented by point *B* in

Table 6-17 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Members Controlled by Flexure

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

1. Requirements for a confined boundary are the same as those given in *ACI 318-95*.
2. Requirements for conforming transverse reinforcement are: (a) closed stirrups over the entire length of the column at a spacing ≤ $d/2$, and (b) strength of closed stirrups V_s ≥ required shear strength of column.
3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups V_s ≥ 3/4 of required shear strength of beam.

Figure 6-1(a), only the longitudinal steel in the boundary of the wall should be included. If the wall does not have a boundary member, then only the longitudinal steel in the outer 25% of the wall section

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

Conditions		Drift Ratio (%), or Chord Rotation (radians)[1]		Residual Strength Ratio	Acceptable Drift (%) or Chord Rotation (radians)[1]				
					Component Type				
					Primary			Secondary	
					Performance Level				
		d	*e*	*c*	IO	LS	CP	LS	CP
i. Shear walls and wall segments									
All shear walls and wall segments[2]		0.75	2.0	0.40	0.40	0.60	0.75	0.75	1.5
ii. Shear wall coupling beams									
Longitudinal reinforcement and transverse reinforcement[3]	$\frac{\text{Shear}}{t_w l_w \sqrt{f_c'}}$								
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	0.018	0.030	0.60	0.006	0.012	0.015	0.015	0.024
	≥ 6	0.012	0.020	0.30	0.004	0.008	0.010	0.010	0.016
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤ 3	0.012	0.025	0.40	0.006	0.008	0.010	0.010	0.020
	≥ 6	0.008	0.014	0.20	0.004	0.006	0.007	0.007	0.012

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

shall be included in the calculation of the yield strength. When calculating the nominal flexural strength of the wall, as represented by point *C* in Figure 6-1(a), all longitudinal steel (including web reinforcement) shall be included in the calculation. For both of the moment calculations described here, the yield strength of the longitudinal reinforcement should be taken as 125% of the specified yield strength to account for material overstrength and strain hardening. For all moment strength calculations, the axial load acting on the wall shall include gravity loads as defined in Chapter 3.

The nominal flexural strength of a shear wall or wall segment shall be used to determine the maximum shear force likely to act in shear walls, wall segments, and columns supporting discontinuous shear walls. For cantilever shear walls and columns supporting discontinuous shear walls, the design shear force is 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 shear force is equal to the shear corresponding to the development of the positive and negative nominal moment strengths at opposite ends of the wall segment.

The nominal shear strength of a shear wall or wall segment shall be determined based on the principles and equations given in Section 21.6 of *ACI 318-95*. The nominal shear strength of RC columns supporting discontinuous shear walls shall be determined based on the principles and equations given in Section 21.3 of *ACI 318-95*. For all shear strength calculations, 1.0 times the specified reinforcement yield strength should be used. There should be no difference between the yield and nominal shear strengths, as represented by points *B* and *C* in Figure 6-1.

When a shear wall or wall segment has a transverse reinforcement percentage, ρ_n, less than the minimum value of 0.0025 but greater than 0.0015, the shear strength of the wall shall be analyzed using the *ACI 318-95* 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 $\rho_n = 0.0015$ (Wood, 1990).

Splice lengths for primary longitudinal reinforcement shall be evaluated using the procedures given in Section 6.4.5. Reduced flexural strengths shall be evaluated at locations where splices govern the usable stress in the reinforcement. The need for confinement reinforcement in shear wall boundary members shall be evaluated by the procedure in the *Uniform Building Code* (ICBO, 1994), or 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.

The nominal flexural and shear strengths of coupling beams reinforced with conventional reinforcement shall be evaluated using the principles and equations contained in Chapter 21 of *ACI 318-95*. The nominal flexural and shear strengths of coupling beams reinforced with diagonal reinforcement shall be evaluated using the procedure defined in the 1994 *NEHRP Recommended Provisions*. In both cases, 125% of the specified yield strength for the longitudinal and diagonal reinforcement should be used.

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

6.8.2.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

All shear walls, wall segments, coupling beams, and columns supporting discontinuous shear walls shall be classified as either deformation- or force-controlled, as defined in Chapter 3. For columns supporting discontinuous shear walls, deformation-controlled actions shall be restricted to flexure. In the other components or elements noted here, deformation-controlled actions shall be restricted to flexure or shear. All other actions shall be defined as being force-controlled actions.

Design actions (flexure, shear, or force transfer at rebar anchorages and splices) on components shall be determined as prescribed in Chapter 3. When determining the appropriate value for the design actions, proper consideration should be given to gravity loads and to the maximum forces that can be transmitted considering nonlinear action in adjacent components. For example, the maximum shear at the base of a shear wall cannot exceed the shear required to develop the nominal flexural strength of the wall. Tables 6-19 and 6-20 present *m* values for use in Equation 3-18. Alternate *m* values are permitted where justified by experimental evidence and analysis.

B. Nonlinear Static and Dynamic Procedures

Inelastic response shall be restricted to those elements and actions listed in Tables 6-17 and 6-18, except where it is demonstrated that other inelastic actions can be tolerated considering 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.

For members experiencing inelastic response, the maximum plastic hinge rotations, drifts, or chord rotation angles shall not exceed the values given in Tables 6-17 and 6-18, for the particular Performance Level being evaluated. Linear interpolation between tabulated values shall be used if the member under analysis has conditions that are between the limits given in the tables. If the maximum plastic hinge rotation, drift, or chord rotation angle exceeds the corresponding value obtained either directly from the tables or by interpolation, the member shall be considered to be deficient, and either the member or the structure will need to be rehabilitated.

6.8.2.5 Rehabilitation Measures

All of the rehabilitation measures listed here for shear walls assume that a proper evaluation will be made of the wall foundation, diaphragms, and connections between existing structural elements and any elements added for rehabilitation purposes. Connection requirements are given in Section 6.4.6.

Table 6-19 ***Numerical Acceptance Criteria for Linear Procedures—Members Controlled by Flexure***

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

1. Requirements for a confined boundary are the same as those given in *ACI 318-95*.
2. Requirements for conforming transverse reinforcement are: (a) closed stirrups over the entire length of the column at a spacing ≤ $d/2$, and (b) strength of closed stirrups $V_s \geq$ required shear strength of column.
3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of beam.

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

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

1. For shear walls and wall segments where inelastic behavior is governed by shear, the axial load on the member must be $\leq 0.15\, A_g f_c'$, the longitudinal reinforcement must be symmetrical, and the maximum shear stress must be $\leq 6\sqrt{f_c'}$, otherwise the shear shall be considered to be a force-controlled action.
2. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing $\leq d/3$, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of beam.

- **Addition of wall boundary members.** Shear walls or wall segments that have insufficient flexural strength may be strengthened by the addition of boundary members. These members could be cast-in-place reinforced concrete elements or steel sections. In both cases, proper connections must be made between the existing wall and the added members. Also, the shear capacity of the rehabilitated wall will need to be reevaluated.

- **Addition of confinement jackets at wall boundaries.** The flexural deformation capacity of a shear wall can be improved by increasing the confinement at the wall boundaries. This is most easily achieved by the addition of a steel or reinforced concrete jacket. 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 shall be three inches. Carbon fiber wrap may also be an effective method for improving the confinement of concrete in compression.

- **Reduction of flexural strength.** In some cases it may be desirable to reduce the flexural capacity of a shear wall to change the governing failure mode from shear to flexure. This is most easily accomplished by saw-cutting a specified number of longitudinal bars near the edges of the shear wall.

- **Increased shear strength of wall.** The shear strength provided by the web of a shear wall can be increased by casting additional reinforced concrete adjacent to the wall web. The new concrete should be at least four inches thick and should contain horizontal and vertical reinforcement. The new concrete will need to be properly bonded to the existing web of the shear wall. The use of carbon fiber sheets, epoxied to the concrete surface, can also increase the shear capacity of a shear wall.

- **Confinement jackets to improve deformation capacity of coupling beams and columns supporting discontinuous shear walls.** The use of confinement jackets has been discussed above for wall boundaries and in Section 6.5 for frame elements. The same procedures can be used to increase both the shear capacity and the deformation capacity of coupling beams and columns supporting discontinuous shear walls.

- **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, the opening between these columns may be infilled to make the wall continuous. The infill and existing columns should be designed to satisfy all the requirements for new wall construction. This may require strengthening of the existing columns by adding a concrete or steel jacket for strength and increased confinement. The opening below a discontinuous shear wall could also 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.

6.9 Precast Concrete Shear Walls

6.9.1 Types of Precast Shear Walls

Precast concrete shear walls typically 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, and, usually a cast-in-place connection strip. Connections between precast segments are typically made along both the horizontal and vertical edges of a wall segment. Tilt-up construction should be considered to be a special technique for precast wall construction. 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.

If the reinforcement connections are made to be stronger than the adjacent precast panels, the lateral load response behavior of the precast wall system will be comparable to that for monolithic shear walls. This design approach is known as *cast-in-place emulation*.

An alternate design approach is to allow the inelastic action to occur at the connections between precast panels, an approach known as *jointed construction*. The provisions given here are intended for use with all types of precast wall systems.

6.9.1.1 Cast-In-Place Emulation

For this design approach, the connections between precast wall elements are designed and detailed to be stronger than the panels they connect. Thus, when the precast shear wall is subjected to lateral loading, any yielding and inelastic behavior should take place in the panel elements away from the connections. If the reinforcement detailing in the panel is similar to that for cast-in-place shear walls, then the inelastic response of a precast shear wall should be very similar to that for a cast-in-place wall.

Modern building codes permit the use of precast shear wall construction in high seismic zones if it satisfies the criteria for cast-in-place emulation. For such structures, the shear walls and wall segments can be evaluated by the criteria defined in Section 6.8.

6.9.1.2 Jointed Construction

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

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

6.9.1.3 Tilt-up Construction

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

6.9.2 Precast Concrete Shear Walls and Wall Segments

6.9.2.1 General Modeling Considerations

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

In most cases, precast concrete shear walls and wall segments within the precast panels may be modeled analytically as equivalent beam-columns that include both flexural and shear deformations. The rigid connection zone at beam connections to these equivalent beam-columns must properly represent the distance from the wall centroid—where the beam-column is placed—to the edge of the wall or wall segment. Unsymmetrical precast wall sections shall model the different bending capacities for the two loading directions.

For precast shear walls and wall segments where shear deformations will have a more significant effect on behavior, a multiple spring model should be used.

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

6.9.2.2 Stiffness for Analysis

The modeling assumptions defined in Section 6.8.2.2 for monolithic concrete shear walls and wall segments shall also be used for precast concrete walls. In addition, the analytical model shall adequately model the stiffness of the connections between the precast components that compose the wall. This may be accomplished by softening the model used to represent the precast panels to account for flexibility in the connections. An alternative procedure would be to add spring elements to simulate axial, shear, and rotational deformations within the connections between panels.

A. Linear Static and Dynamic Procedures

The modeling procedures given in Section 6.8.2.2A, combined with a procedure for including connection deformations as noted above, shall be used.

B. Nonlinear Static and Dynamic Procedures

Nonlinear load-deformation relations shall follow the general guidelines of Section 6.4.1.2. The monotonic load-deformation relationships for analytical models that represent precast shear walls and wall elements within precast panels shall be represented by one of the general shapes defined in Figure 6-1. Values for plastic hinge rotations or drifts at points *B*, *C*, and *E* for the two general shapes are defined below. The strength levels at points *B* and *C* should correspond to the yield strength and nominal strength, as defined in Section 6.8.2.3. The residual strength for the line segment *D–E* is defined below.

For precast shear walls and wall segments whose inelastic behavior under lateral loading is governed by flexure, the general load-deformation relationship in Figure 6-1(a) will be referred to. For these members, the x-axis of Figure 6-1(a) should be taken as the rotation over the plastic hinging region at the end of the member (Figure 6-2). If the requirements for cast-in-place emulation are satisfied, the value of the hinge rotation at point *B* corresponds to the yield rotation, θ_y, and is given by Equation 6-5. The same expression should also be used for wall segments within a precast panel if flexure controls the inelastic response of the segment.

If the precast wall is of jointed construction and flexure governs the inelastic response of the member, then the value of θ_y will need to 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 in

Figure 6-1(b) will be referred to. For these members, the x-axis of Figure 6-1(b) should be taken as the story drift for shear walls, and as the element drift for wall segments (Figure 6-3).

For construction classified as cast-in-place emulation, the values for the variables *a*, *b*, and *c*, which are required to define the location of points *C*, *D*, and *E* in Figure 6-1(a), are given in Table 6-17. For construction classified as jointed construction, the values of *a*, *b*, and *c* given in Table 6-17 shall be reduced to 50% of the given values, unless there is experimental evidence available to justify higher values. In no case, however, shall values larger than those given in Table 6-17 be used.

For construction classified as cast-in-place emulation, values for the variables *d*, *e*, and *c*, which are required to find the points *C*, *D*, and *E* in Figure 6-1(b), are given in Tables 6-17 and 6-18 for the appropriate member conditions. For construction classified as jointed construction, the values of *d*, *e*, and *c* given in Tables 6-17 and 6-18 shall be reduced to 50% of the given values unless there is experimental evidence available to justify higher values. In no case, however, shall values larger than those given in Tables 6-17 and 6-18 be used.

For Tables 6-17 and 6-18, linear interpolation between tabulated values shall be used if the member under analysis has conditions that are between the limits given in the tables.

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

6.9.2.3 Design Strengths

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

For jointed construction, calculations of axial, shear, and flexural strength of the connections between panels shall be based on known or assumed material properties and the fundamental principles of structural mechanics. Yield strength for steel reinforcement of connection hardware used in the connections shall be increased to 125% of its specified yield value when calculating the axial and flexural strength of the connection region. The unmodified specified yield strength of the reinforcement and connection hardware shall be used when calculating the shear strength of the connection region.

In older construction, particular attention must be given to the technique used for splicing reinforcement extending from adjacent panels into the connection. These connections may be insufficient and can often govern the strength of the precast shear wall system. If sufficient detail is not given on the design drawings, concrete should be removed in some connections to expose the splicing details for the reinforcement.

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. Thus, the values for strength represented by the points *B* and *C* in Figure 6-1 shall be computed following the procedures given in Section 6.8.2.3.

6.9.2.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

For precast shear wall construction that emulates cast-in-place construction and for wall segments within a precast panel, the acceptance criteria defined in Section 6.8.2.4A shall be followed. For precast shear wall construction defined as jointed construction, the acceptance criteria procedure given in Section 6.8.2.4A shall be followed. However, the *m* values given in Tables 6-19 and 6-20 shall be reduced by 50%, unless experimental evidence justifies the use of a larger value. In no case shall an *m* value be taken as less than 1.0.

B. Nonlinear Static and Dynamic Procedures

Inelastic response shall be restricted to those shear walls (and wall segments) and actions listed in Tables 6-17 and 6-18, except where it is demonstrated that other inelastic action can be tolerated considering the selected Performance Levels. For members experiencing inelastic behavior, the magnitude of the other actions (forces, moments, or torques) in the member 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 of the cast-in-place emulation type of construction, and for wall segments within a precast panel, the maximum plastic hinge rotation angles or drifts during inelastic response shall not exceed the values given in Tables 6-17 and 6-18. 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 given in Tables 6-17 and 6-18, unless experimental evidence is available to justify a higher value. However, in no case shall deformation values larger than those given 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 will need to be rehabilitated.

6.9.2.5 Rehabilitation Measures

Precast concrete shear wall systems may suffer from some of the same deficiencies as cast-in-place walls. These may include inadequate flexural capacity, inadequate shear capacity with respect to flexural capacity, lack of confinement at wall boundaries, and inadequate splice lengths for longitudinal reinforcement in wall boundaries. All of these deficiencies can be rehabilitated by the use of one of the measures described in Section 6.8.2.5. A few deficiencies unique to precast wall construction are inadequate connections between panels, to the foundation, and to floor or roof diaphragms.

- **Enhancement of connections between adjacent or intersecting precast wall panels.** A combination of mechanical and cast-in-place details may be used to strengthen connections between precast panels. Mechanical connectors may include steel shapes and various types of drilled-in anchors. Cast-in-place strengthening methods generally involve exposing the reinforcing steel at the edges of adjacent panels, adding vertical and transverse (tie) reinforcement, and placing new concrete.

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

- **Enhancement of connections between precast wall panels and floor or roof diaphragms.** These connections can be strengthened by using either supplemental mechanical devices or cast-in-place connectors. Both in-plane shear and out-of-plane forces will need to be considered when strengthening these connections.

6.10 Concrete Braced Frames

6.10.1 Types of Concrete Braced Frames

Reinforced concrete braced frames are those frames with monolithic reinforced concrete beams, columns, and diagonal braces that are coincident at beam-column joints. Components are nonprestressed. Under lateral loading, the braced frame resists loads primarily through truss action.

Masonry infills may be present in braced frames. Where masonry infills are present, requirements for masonry infilled frames as specified in Section 6.7 also apply.

The provisions are applicable to existing reinforced concrete braced frames, and existing reinforced concrete braced frames rehabilitated by addition or removal of material.

6.10.2 General Considerations in Analysis and Modeling

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

The analytical model generally can represent the framing, using line elements with properties concentrated at component centerlines. General considerations relative to the analytical model are summarized in Section 6.5.2.1.

In frames having braces in some bays and no braces in other bays, the restraint of the brace shall be represented as described above, and the nonbraced bays shall be modeled as frames according to the specifications of this chapter. Where braces create a vertically discontinuous frame, the effects 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 are permitted in secondary components. Acceptance criteria are presented in Section 6.10.5.

6.10.3 Stiffness for Analysis

6.10.3.1 Linear Static and Dynamic Procedures

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

6.10.3.2 Nonlinear Static Procedure

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

Beams, columns, and braces in braced portions may be modeled using nonlinear truss components. Beams and columns in nonbraced portions may be modeled using procedures described elsewhere in this chapter. The model shall be capable of representing inelastic response along the component lengths, as well as within connections.

Numerical quantities in Figure 6-1 may be derived from tests or rational analyses. Alternately, the guidelines of Section 6.7.2.2B may be used, with braces modeled as columns per Table 6-15.

6.10.3.3 Nonlinear Dynamic Procedure

For the NDP, the complete hysteresis behavior of each component shall be modeled using properties verified by tests. Unloading and reloading properties shall represent significant stiffness and strength degradation characteristics.

6.10.4 Design Strengths

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

6.10.5 Acceptance Criteria

6.10.5.1 Linear Static and Dynamic Procedures

All component actions shall be classified as being either deformation-controlled or force-controlled, as defined in Chapter 3. 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 Chapter 3. Refer to other sections of this chapter for *m* values for concrete frames, except that *m* values for beams, columns, and braces modeled as tension and compression components may be taken as equal to values specified for columns in Table 6-16. Values of *m* shall be reduced from values in that table where component buckling is a consideration. Alternate approaches or values are permitted where justified by experimental evidence and analysis.

6.10.5.2 Nonlinear Static and Dynamic Procedures

Calculated component actions shall not exceed the numerical values listed in Table 6-15 or the relevant tables for isolated frames given elsewhere in 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 are permitted where justified by experimental evidence and analysis.

6.10.6 Rehabilitation Measures

Rehabilitation measures include the general approaches listed for other elements in this chapter, plus other approaches based on rational procedures.

Rehabilitated frames shall be evaluated according to the general principles and requirements of this chapter. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in the analytical model. Connections required between existing and new elements shall satisfy requirements of Section 6.4.6 and other requirements of the *Guidelines*.

6.11 Concrete Diaphragms

6.11.1 Components of Concrete Diaphragms

Cast-in-place concrete diaphragms transmit inertial forces from one location in a structure to a vertical lateral-force-resisting element. A concrete diaphragm is generally a floor or roof slab, but can be a structural truss in the horizontal plane.

Diaphragms are made up of slabs that transmit shear forces, struts that provide continuity around openings, collectors that gather force and distribute it, and chords that are located at the edges of diaphragms and that resist tension and compression forces.

6.11.1.1 Slabs

The primary function of any slab that is part of a floor or roof system is to support gravity loads. A slab must also function as part of the diaphragm to transmit the shear forces associated with the load transfer. These internal shear forces are generated when the slab is the load path for forces that are being transmitted from one vertical lateral-force-resisting system to another, or when the slab is functioning to provide bracing to other portions of the building that are being loaded out of plane. Included in this section are all versions of cast-in-place concrete floor systems, and concrete-on-metal deck systems.

6.11.1.2 Struts and Collectors

Struts and collectors are built into diaphragms in locations where there are defined stress demands that exceed the typical stress capacity of the diaphragm. These locations occur around openings in the diaphragms, along defined load paths between lateral-load-resisting elements, and at intersections of portions of floors that have plan irregularities. Struts and collectors may occur within the slab thickness or may have the form of cast-in-place beams that are monolithic with the slabs. The forces that they resist are primarily axial in nature, but may also include shear and bending forces.

6.11.1.3 Diaphragm Chords

Diaphragm chords generally occur at the edges of a horizontal diaphragm and function to resist bending stresses in the diaphragm. Tensile forces typically are most critical, but compressive forces in thin slabs could be a problem. Exterior walls can serve this function if there is adequate horizontal shear capacity between the slab and wall. When evaluating an existing building, special care should be taken to evaluate the condition of the lap splices. Where the splices are not confined by closely-spaced transverse reinforcement, splice failure is possible if stress levels reach critical values. In rehabilitation construction, new laps should be confined by closely-spaced transverse reinforcement.

6.11.2 Analysis, Modeling, and Acceptance Criteria

6.11.2.1 General Considerations

The analysis 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 at any point in the diaphragm shall be considered.

The analytical model of the diaphragm can typically be taken as a continuous or simple span horizontal beam that is supported by elements of varying stiffness. The beam may be rigid or semi-rigid. Most computer models assume a rigid diaphragm. Few cast-in-place diaphragms would be considered flexible, whereas a thin concrete slab on a metal deck might be semi-rigid depending on the length-to-width ratio of the diaphragm.

6.11.2.2 Stiffness for Analysis

Diaphragm stiffness shall be modeled according to Section 6.11.2.1 and shall be determined using a linear elastic model and gross section properties. The modulus of elasticity used shall be that of the concrete as specified in Section 8.5.1 of *ACI 318-95*. When the length-to-width ratio of the diaphragm exceeds 2.0 (where the length is the distance between vertical elements), the effects of diaphragm deflection shall be considered when assigning lateral forces to the resisting vertical elements. 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 due to the same diaphragm displacement.

6.11.2.3 Design Strengths

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

The maximum component strength shall be determined considering potential failure in flexure, axial load, shear, torsion, development, and other actions at all points in the component under the actions of design gravity and lateral load combinations. The shear strength shall be as specified in Section 21.6.4 of *ACI 318-95*. Strut, collector, and chord strengths shall be determined according to Section 6.5.2.3 of these *Guidelines*.

6.11.2.4 Acceptance Criteria

All component actions shall be classified as either deformation-controlled or force-controlled, as defined in Chapter 3. Diaphragm shear shall be considered as being a force-controlled component and shall have a DCR not greater than 1.25. Acceptance criteria for all other component actions shall be as defined in Section 6.5.2.4A, with *m* values taken according to similar components in Tables 6-10 and 6-11 for use in Equation 3-18. Analysis shall be restricted to linear procedures.

6.11.3 Rehabilitation Measures

Cast-in-place concrete diaphragms can have a wide variety of deficiencies; see Chapter 10 and FEMA 178 (BSSC, 1992a). Two general alternatives may be used to correct deficiencies: either improving the strength and ductility, or reducing the demand in accordance with FEMA 172 (BSSC, 1992b). Individual components can be strengthened or improved by adding additional reinforcement and encasement. Diaphragm thickness may be increased, but the added weight may overload the footings and increase the seismic load. Demand can be lowered by adding additional lateral-force-resisting elements, introducing additional damping, or base isolating the structure. All corrective measures taken shall be based upon engineering mechanics, taking into account load paths and deformation compatibility requirements of the structure.

6.12 Precast Concrete Diaphragms

6.12.1 Components of Precast Concrete Diaphragms

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

Precast concrete diaphragms can be classified as topped or untopped. A topped diaphragm is one that has had a concrete topping slab poured over the completed horizontal system. Most floor systems have a topping system, but some hollow core floor systems do not. The topping slab generally bonds to the top of the precast elements, but may have an inadequate thickness at the center of the span, or may be inadequately reinforced. Also, extensive cracking of joints may be present along the panel joints. Shear transfer at the edges of precast concrete diaphragms is especially critical.

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

6.12.2 Analysis, Modeling, and Acceptance Criteria

Analysis and modeling of precast concrete diaphragms shall conform to Section 6.11.2.2, with the added requirement that special attention be paid to considering the segmental nature of the individual components.

Component strengths shall be determined according to Section 6.11.2.3, with the following exception. Welded connection strength shall be determined using the latest version of the Precast Concrete Institute (PCI) Handbook, assuming that the connections have little ductility unless test data are available to document the assumed ductility.

Acceptance criteria shall be as defined in Section 6.11.2.4; the criteria of Section 6.4.6.2, where applicable, shall also be included.

6.12.3 Rehabilitation Measures

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

6.13 Concrete Foundation Elements

6.13.1 Types of Concrete Foundations

Foundations 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 are classified as either shallow or deep foundations. Shallow foundations include spread or isolated footings; strip or line footings; combination footings; and concrete mat footings. Deep foundations include pile foundations and cast-in-place piers. Concrete grade beams may be present in both shallow and deep foundation systems.

These provisions are applicable to existing foundation elements and to new materials or elements that are required to rehabilitate an existing building.

6.13.1.1 Shallow Foundations

Existing spread footings, strip footings, and combination footings may be reinforced or unreinforced. Vertical loads are transmitted to the soil by direct bearing; 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 must 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 are resisted primarily by friction between the soil and the bottom of the footing, and by passive pressure developed against foundation walls that are part of the system.

6.13.1.2 Deep Foundations

A. Driven Pile Foundations

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

B. Cast-in-Place Pile Foundations

Cast-in-place concrete pile foundations consist of reinforced concrete placed in a drilled or excavated shaft. The shaft may be formed or bare. 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. Cast-in-place pile or pier foundations resist vertical and lateral loads in a manner similar to that of driven pile foundations.

6.13.2 Analysis of Existing Foundations

The analytical model for concrete buildings, with columns or walls cast monolithically with the foundation, is sometimes assumed to have the vertical structural elements fixed at the top of the foundation. When this is assumed, the foundations and supporting soil must be capable of resisting the induced moments. When columns are not monolithic with their foundations, or are designed so as to not resist flexural moments, they may be modeled with pinned ends. In such cases, the column base must 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 must be taken into account at the point of maximum displacement of the superstructure.

Overturning moments and economics may dictate the use of more rigorous Analysis Procedures. When this is the case, 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 based on the material in Chapter 4, and on the recommendations of the geotechnical consultant. Rigorous analysis of structures with deep foundations in soft soils will require 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 is required. Buildings designed for gravity loads only may have a nominal (about six inches) embedment of the piles without any dowels into the pile cap. These piles must be modeled as being "pinned" to the cap. Unless the connection can be identified from the available construction documents, the "pinned" connection should be assumed in any analytical model.

When the foundations are included in the analytical model, the responses of the foundation components may be derived by any of the analytical methods prescribed in Chapter 3, as modified by the requirements of Section 6.4. When the structural elements of the analytical model are assumed to be pinned or fixed at the foundation level, the reactions (axial loads, shears, and moments) of those elements shall be used to evaluate the individual components of the foundation system.

6.13.3 Evaluation of Existing Condition

Allowable soil capacities (subgrade modulus, bearing pressure, passive pressure) are a function of the chosen Performance Level, and will be as prescribed in Chapter 4 or as established with project-specific data by a geotechnical consultant. All components of existing foundation elements, and all new material, components, or elements required for rehabilitation, will be considered to be force-controlled ($m = 1.0$) based on the mechanical and analytical properties in Section 6.3.3. However, the capacity of the foundation components need not exceed 1.25 times the capacity of the supported vertical structural component or element (column or wall). The amount of foundation displacement that is acceptable for the given structure should be determined by the design engineer, and is a function of the desired Performance Level.

6.13.4 Rehabilitation Measures

The following general rehabilitation measures are applicable to existing foundation elements. Other approaches, based on rational procedures, may also be utilized.

6.13.4.1 Rehabilitation Measures for Shallow Foundations

- **Enlarging the existing footing by lateral additions**. The existing footing will continue to resist the loads and moment acting at the time of rehabilitation (unless temporarily removed). The enlarged footing is to resist subsequent loads and moments produced by earthquakes if the lateral additions are properly tied into the existing footing. Shear transfer and moment development must be accomplished in the additions.

- **Underpinning the footing.** Underpinning involves the removal of unsuitable soil under an existing footing, coupled with replacement using concrete, soil cement, suitable soil, or other material, and must be properly staged in small increments so as not to endanger the stability of the structure. This technique also serves to enlarge an existing footing or to extend it to a more competent soil stratum.

- **Providing tension hold-downs.** Tension ties (soil and rock anchors—prestressed and unstressed) are drilled and grouted into competent soils and anchored in the existing footing to resist uplift. Increased soil bearing pressures produced by the ties must be checked against values associated with the desired Performance Level. Piles or drilled piers may also be utilized.

- **Increasing effective depth of footing.** This method involves pouring new concrete to increase shear and moment capacity of existing footing. New horizontal reinforcement can be provided, if required, to resist increased moments.

- **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. The practicality must be checked against possible severe architectural restrictions.

- **Providing pile supports for concrete footings or mat foundations.** Addition of piles requires careful design of pile length and spacing to avoid overstressing the existing foundations. The technique may only be feasible in a limited number of cases for driven piles, but special augered systems have been developed and are used regularly.

- **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. The addition of new shear walls or braces will generally reduce the demand on existing foundations.

- **Adding new grade beams.** Grade beams may be used to tie existing footings together when poor soil exists, to provide fixity to column bases, and to distribute lateral loads between individual footings, pile caps, or foundation walls.

- **Improving existing soil.** Grouting techniques may be used to improve existing soil.

6.13.4.2 Rehabilitation Measures for Deep Foundations

- **Providing additional piles or piers.** The addition of piles or piers may require extension and additional reinforcement of existing pile caps. See the comments in previous sections for extending an existing footing.

- **Increasing the effective depth of the pile cap.** Addition of new concrete and reinforcement to the top of the cap is done to increase shear and moment capacity.

- **Improving soil adjacent to existing pile cap.** See Section 4.6.1.

- **Increasing passive pressure bearing area of pile cap.** Addition of new reinforced concrete extensions to the existing pile cap provides more vertical foundation faces and greater load transferability.

- **Changing the building system to reduce the demands on the existing elements.** Introduction of new lateral-load-resisting elements may reduce demand.

- **Adding batter piles or piers.** Batter piles or piers may be used to resist lateral loads. It should be noted that batter piles have performed poorly in recent earthquakes when liquefiable soils were present. This is especially important to consider around wharf structures and in areas that have a high water table. See Sections 4.2.2.2, 4.3.2, and 4.4.2.2B.

- **Increasing tension tie capacity from pile or pier to superstructure.**

6.14 Definitions

The definitions used in this chapter generally follow those of BSSC (1995) as well as those published in ACI 318. Many of the definitions that are independent of material type are provided in Chapter 2.

6.15 Symbols

A_g	Gross area of column, in.^{2}
A_j	Effective cross-sectional area within a joint, in.^{2}, in a plane parallel to plane of reinforcement generating shear in the joint. The joint depth shall be the overall depth of the column. Where a beam frames into a support of larger width, the effective width of the joint shall not exceed the smaller of: (1) beam width plus the joint depth, and (2) twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side.
A_s	Area of nonprestressed tension reinforcement, in.^{2}
A'_s	Area of compression reinforcement, in.^{2}
A_w	Area of the web cross section, = $b_w d$
E_c	Modulus of elasticity of concrete, psi
E_s	Modulus of elasticity of reinforcement, psi
I	Moment of inertia
I_g	Moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement
L	Length of member along which deformations are assumed to occur
M_{gCS}	Moment acting on the slab column strip according to ACI 318

M_n	Nominal moment strength at section
M_{nCS}	Nominal moment strength of the slab column strip
M_y	Yield moment strength at section
N_u	Factored axial load normal to cross section occurring simultaneously with V_u. To be taken as positive for compression, negative for tension, and to include effects of tension due to creep and shrinkage.
P	Axial force in a member, lbs
P_o	Nominal axial load strength at zero eccentricity
Q	Generalized load
Q_{CE}	Expected strength of a component or element at the deformation level under consideration for deformation-controlled actions
Q_{CL}	Lower-bound estimate of the strength of a component or element at the deformation level under consideration for force-controlled actions
V	Design shear force at section
V_c	Nominal shear strength provided by concrete
V_g	Shear acting on slab critical section due to gravity loads
V_n	Nominal shear strength at section
V_o	Shear strength of slab at critical section
V_s	Nominal shear strength provided by shear reinforcement
V_u	Factored shear force at section
a	Parameter used to measure deformation capacity
b	Parameter used to measure deformation capacity
b_w	Web width, in.
c	Parameter used to measure residual strength
c_1	Size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, in.
d	Parameter used to measure deformation capacity
d	Distance from extreme compression fiber to centroid of tension reinforcement, in.
d_b	Nominal diameter of bar, in.
e	Parameter used to measure deformation capacity
f'_c	Compressive strength of concrete, psi
f_{pc}	Average compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses)
f_s	Stress in reinforcement, psi
f_y	Yield strength of tension reinforcement
h	Height of member along which deformations are measured
h	Overall thickness of member, in.
h_c	Gross cross-sectional dimension of column core measured in the direction of joint shear, in.
h_w	Total height of wall from base to top, in.
k	Coefficient used for calculation of column shear strength
l_b	Provided length of straight development, lap splice, or standard hook, in.
l_d	Development length for a straight bar, in.
l_e	Length of embedment of reinforcement, in.
l_p	Length of plastic hinge used for calculation of inelastic deformation capacity, in.
l_w	Length of entire wall or a segment of wall considered in the direction of shear force, in.
m	Modification factor used in the acceptance criteria of deformation-controlled components or elements, indicating the available ductility of a component action
t_w	Thickness of wall web, in.
Δ	Generalized deformation, consistent units
γ	Coefficient for calculation of joint shear strength
γ_f	Fraction of unbalanced moment transferred by flexure at slab-column connections
θ	Generalized deformation, radians
θ_y	Yield rotation, radians
κ	A reliability coefficient used to reduce component strength values for existing components, based on the quality of knowledge about the components' properties (see Section 2.7.2)

λ Correction factor related to unit weight of concrete

μ Coefficient of friction

ρ Ratio of nonprestressed tension reinforcement

ρ' Ratio of nonprestressed compression reinforcement

ρ'' Reinforcement ratio for transverse joint reinforcement

ρ_{bal} Reinforcement ratio producing balanced strain conditions

ρ_n Ratio of distributed shear reinforcement in a plane perpendicular to the direction of the applied shear

6.16 References

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