CHAPTER 6
ACCEPTANCE CRITERIA

6-1. General.

This chapter prescribes the acceptance criteria for the various performance objectives described in Chapter 4. The applicable acceptance criteria for each performance objective are provided for each of the analytical procedures described in Chapter 5. Numerical values of the criteria for specific structural systems are provided in Chapter 7.

6-2. Performance Objective 1A.

a. General. This is the basic Life Safety performance objective for all buildings, and is the only performance objective for Seismic Use Group I buildings, which constitute the vast majority of military construction. The design is based on the FEMA 302 seismic provisions with an applicable Response Modification Factor, *R*, and drift limits based on elastic analysis. The designer should not lose sight of the fact that an elastic analysis with an *R* factor greater than unity implies energy dissipation capacity in the structural system, and structural detailing that results in brittle or nonductile response could preclude the assumed energy dissipation, and lead to the development of a premature failure mechanism.

b. Analytical Procedure. As indicated in Table 4-4, the minimum analytical procedures for this performance objective are the linear elastic static or dynamic procedures with ELF or modal analysis in accordance with FEMA 302. More rigorous analytical procedures, as described in subsequent paragraphs for enhanced performance objectives, may be necessary or desirable for highly irregular or complex structural systems.

c. Design Coefficients and Factors for Basic Seismic-Force Resisting Systems. Table 7-1 (Table 5.2.2 of FEMA 302) lists the basic seismic-force-resisting systems, and for each system, provides detailing references; the applicable response modification factor, *R*; the systems overstrength factor, S_o; the deflection amplification factor, C_d; and system restrictions and building height limitations by Seismic Design Category.

d. Deflection and Drift Limits. Table 6-1 (Table 5.2.8 in FEMA 302) provides the allowable story drift applicable to each performance level for representative structural systems. The story drift,), is computed as the difference of the deflections $*_x$, of the center of mass at the top and bottom of the story under consideration. The story deflections are equal to the deflections $*_{xe}$, determined from the elastic analysis multiplied by the deflection amplification factor, C_d.

e. Acceptance Criteria. The acceptance criteria for Performance Objective 1A consists in confirming that the capacity of the structural components and elements satisfies the combined demand of the gravity and design loads in accordance with the LRFD procedures referenced for the various structural material in FEMA 302. Additionally, compliance with the drift and detailing requirements prescribed in FEMA 302, or incorporated by reference, must be met.

Table 6-1
Allowable Story Drift, ?$_a$ (in. or mm)

Structure	**Performance Level**		
	1	**2**	**3**
Structures, other than masonry shear wall or masonry wall frame structures, four stories or less in height with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	0.025 h_{sx} [b]	0.020 h_{sx}	0.015 h_{sx}
Masonry cantilever shear wall structures [c]	0.010 h_{sx}	0.010 h_{sx}	0.010 h_{sx}
Other masonry shear wall structures	0.007 h_{sx}	0.007 h_{sx}	0.007 h_{sx}
Masonry wall frame structures	0.013 h_{sx}	0.013 h_{sx}	0.010 h_{sx}
All other structures	0.020 h_{sx}	0.015 h_{sx}	0.010 h_{sx}

[a] h_{sx} is the story height below Level x.

[b] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts.

[c] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

6-3. Enhanced Performance Objectives.

The minimum analytical procedure for enhanced performance objectives is the linear elastic static or dynamic procedure using the modification factors, *m* (refer to Paragraph 5-2b(2) for the exception applicable to buildings with enhanced performance objectives in Seismic Design Categories A and B). The dynamic procedure shall be employed when the limitations described in Paragraph 5-2b preclude the use of the static procedure. The acceptance criteria for all performance objectives analyzed by the dynamic procedure are the same as for the Linear Elastic Static Procedure, except that the seismic design actions, Q_E in Equations 6-2, 6-4a, and 6-4b, for the individual structural components, are obtained by either square root of the sum of the squares (SRSS), or by the complete quadratic combination (CQC) of the modal values for each action. The nonlinear static procedure shall be used in lieu of the linear procedures when the conditions described in Paragraph 5-4b are present. Alternative analytical procedures and applicable acceptance criteria not prescribed by this document will require specific authorization by the cognizant design authority.

a. Linear Elastic Static Procedure. For those structures with a linear elastic static procedure permitted in accordance with Paragraph 5-2b, compliance with enhanced Performance Objectives 2A, 2B, and 3B shall be achieved by evaluation of the demand on individual structural components in accordance with the following procedures adopted from FEMA 273. Structural components or elements are classified as being either primary or secondary. Primary components and elements are those that provide the structure's overall ability to resist collapse under earthquake-induced ground motion. Although damage to these components, and some degradation of their strength and stiffness, may be permitted to occur, the overall function of these components in resisting structural collapse should not be compromised. Other components and elements are designated as secondary. For some structural performance levels, substantial degradation of the lateral-force-resisting strength and stiffness of secondary components and elements is permissible; however, the ability of these secondary components and elements to support gravity loads under the maximum deformations induced by the design ground motion, must be preserved.

(1) General. The analysis procedure indicates the structure's response to the design earthquake and the forces and deformations imposed on the various components, as well as the global drift demands on the structure. Acceptability of component behavior is evaluated for each of the component's various actions using Equation 6-2 for ductile (deformation-controlled) actions, and Equations 6-4a and 6-4b for nonductile (force-controlled) actions.

(a) Figure 6-1 indicates typical idealized force-deformation curves for various types of component actions. The Type 1 curve is representative of typical ductile behavior. It is characterized by an elastic range (point 0 to point 1 on the curve), a plastic range (points 1 to 2) that may include strain hardening or softening, and a strength-degraded range (points 2 to 3), in which the residual force that can be resisted is significantly less than the peak strength, but still substantial. Acceptance

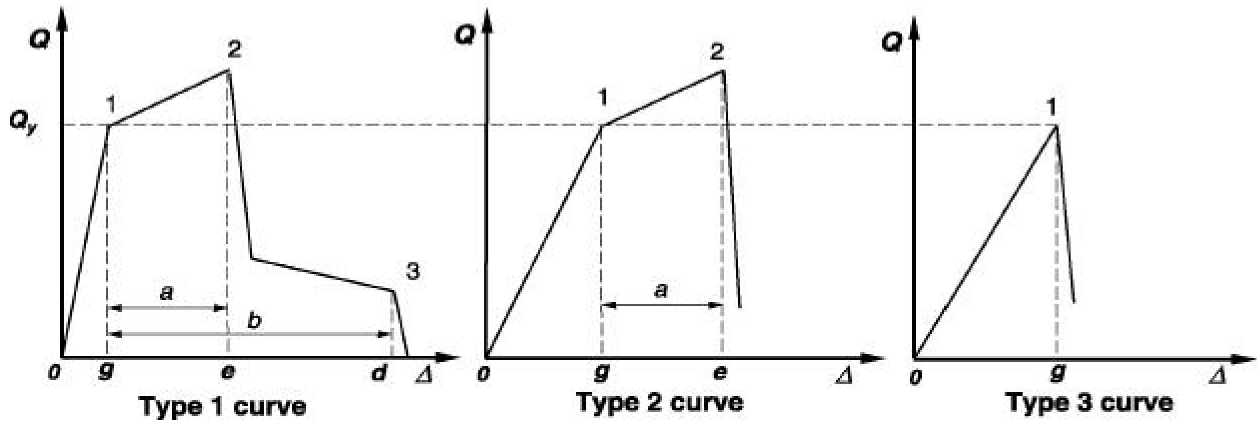

Figure 6-1 General Component Behavior Curves

criteria for primary elements that exhibit this behavior are typically within the elastic or plastic ranges, depending on the performance level. Acceptance criteria for secondary elements can be within any of the ranges. Primary component actions exhibiting this behavior are considered deformation-controlled if the plastic range is sufficiently large ($b \geq 2a$); otherwise, they are considered force-controlled. Structural steel and reinforced concrete members in flexural response are typical examples of deformation-controlled elements. Secondary component actions exhibiting this behavior are typically considered to be deformation-controlled.

(b) The Type 2 curve is representative of semi-ductile behavior. It is characterized by an elastic range and a plastic range, followed by a rapid and complete loss of strength if the behavior is categorized as deformation-controlled. Otherwise it is categorized as force-controlled. A reinforced concrete wall in shear response is a typical example of a deformation-controlled element with semi-ductile behavior. Acceptance criteria for primary and secondary components exhibiting this behavior will be within the elastic or plastic ranges, depending on the performance level.

(c) The Type 3 curve is representative of a brittle or non-ductile behavior. It is characterized by an elastic range, followed by a rapid and complete loss of strength. Component actions resulting in this behavior are always categorized as force-controlled. Shear critical (i.e., shear capacity is attained prior to flexural capacity) beams and columns in reinforced concrete frames are typical examples of force-controlled elements. Acceptance criteria for primary and secondary components exhibiting this behavior are always within the elastic range.

(d) Figure 6-2 shows an idealized force versus deformation curve that is used throughout this procedure to specify acceptance criteria for deformation-controlled components and element actions for any of the four basic types of materials. Linear response is depicted between point A (unloaded component) and an effective yield point B. The slope from B to C is typically a small percentage (0 to 10 percent) of the elastic slope, and is included to represent phenomena such as strain hardening. C has an ordinate that represents the strength of the component, and an abscissa value equal to the deformation at which significant strength degradation begins (line CD). Beyond point D, the component responds with substantially reduced strength to point E. At deformations greater than point E, the component strength is essentially zero. In Figure 6-1, Q_{CE} is the expected strength of a component or element at the deformation level under consideration for deformation-controlled actions. Expected strength is defined as the mean value of resistance at the deformation level anticipated, and includes phenomena such as strain hardening and plastic section development. Q_{CL} is the lower-bound strength of a component or element at the deformation level under consideration for force-controlled actions. Lower-bound strength is typically established by the lower five percentile of yield, buckling, or brittle failure strength. Q_{CE} and Q_{CL} are further defined in Paragraph 6-3a(3).

(e) For some components it is convenient to prescribe acceptance criteria in terms of deformation (e.g., 2 or)), while for others it is more convenient to give criteria in terms of deformation ratios. To accommodate this, two types of idealized force versus deformation curves are used in this procedure as illustrated in Figures 6-2a and 6-2b. Figure 6-2a shows normalized force (Q/Q_{CE}) versus deformation (or)) and the parameters a, b, and c. Figure 6-2b shows normalized force (Q/Q_{CE}) versus deformation ratio ($2/2_y$, $)/)_y$ or)/h) and the parameters d, e, and c. Elastic stiffness and values for the parameters a, b, c, d, and e that can be used for modeling components for various structural systems are given in Chapter 7. Figure 6-2c graphically shows the approximate deformation or deformation ratio, in relation to the idealized force versus deformation curve, that is deemed acceptable in this procedure for structural components for Immediate Occupancy (IO), Safe Egress (SE), and Life Safety (LS), Performance Levels. The Collapse Prevention (CP) performance level indicated in Figure 6-2c is not an acceptable performance level, and is indicated here and in the acceptance criteria tables in Chapter 7 as a limit state for ductile response. Numerical values of the acceptable deformations or deformation ratios are given in Chapter 7 for components and elements in various structural systems. Additional guidelines on the calculation of individual component force and deformation capacities may be found in the following chapters.

- Base isolation systems and energy dissipation systems - Chapter 8.

- Foundations - Chapter 9.

- Nonstructural Systems and Components – Chapter 10.

Acceptance criteria for elements and components for which criteria are not presented in this document shall be determined by an approved qualification-testing program.

(2) Pseudo-lateral load, *V*, in a given horizontal direction of a building, is given by Equation 6-1. This load shall be used for the design of the vertical seismic framing system when the linear elastic analysis procedures are used with the *m* values.

$$V = C_1\, C_2\, C_3\, S_a\, W \qquad (6\text{-}1)$$

where:

V = pseudo-lateral load. This force, when distributed over the height of the linearly elastic model of the building, is intended to produce calculated lateral displacements approximately equal to those that are expected in the real structure during the design event. If it is expected that the actual structure will yield during the design event, the force given by Equation 6-1 may be significantly larger than the actual strength of the structure to resist that force. The acceptance criteria in the following paragraph are developed to take this aspect into account.

C_1, C_2, C_3, and S_a are defined in Paragraph 5-4f(2).

W = Total dead and applicable live loads as defined in FEMA 302.

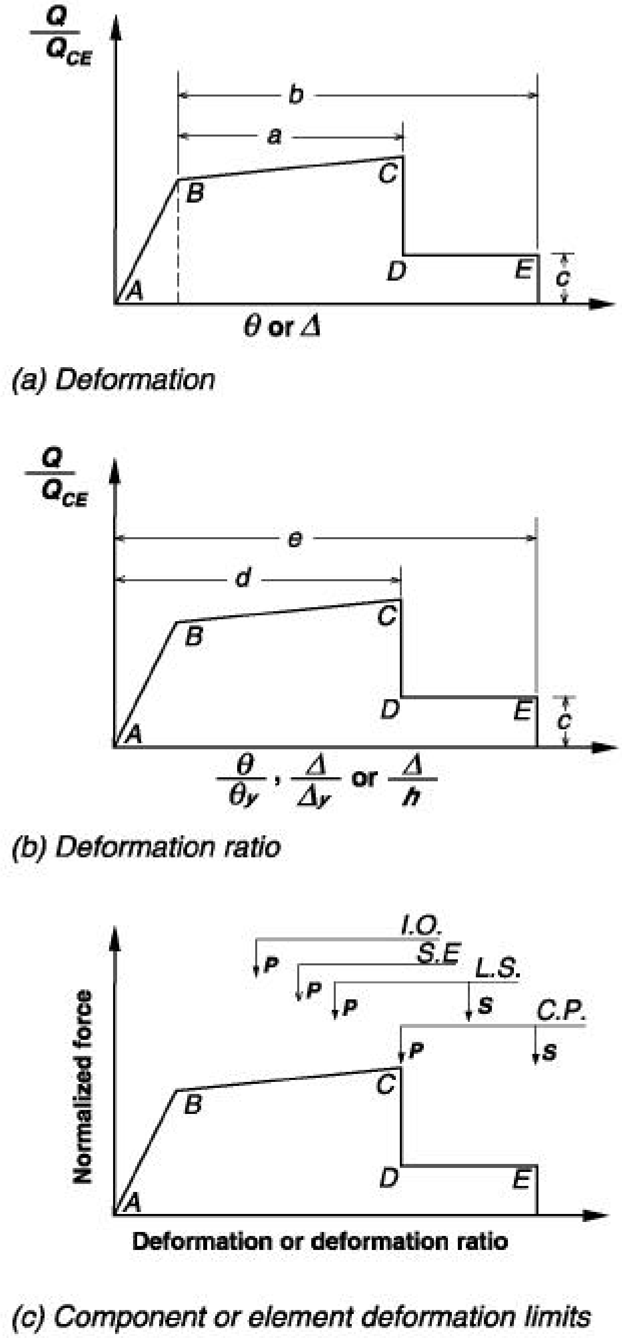

Figure 6-2 Idealized Component Load Versus Deformation Curves for Depicting Component Modeling and Acceptability

(3) Design actions.

(a) Deformation-controlled actions shall be calculated according to Equation 6-2:

$$Q_{UD} = Q_G \pm Q_E \qquad (6\text{-}2)$$

where:

Q_{UD} = design action due to gravity loads and earthquake loads.

Q_G = action due to design gravity loads as defined in ASCE 7.

Q_E = action due to design earthquake loads.

Deformation-controlled actions in structural components shall satisfy Equation 6-3:

$$mQ_{CE} \geq Q_{UD} \qquad (6\text{-}3)$$

where:

m = component or element demand modifier to account for expected ductility of the deformation associated with this action at selected Performance Level (see Chapters 7, 8, 9 and 10).

Q_{CE} = expected strength of the component or element at the deformation level under consideration for deformation-controlled actions.

For Q_{CE}, the expected strength shall be determined considering all co-existing actions acting on the component under the design loading condition. In this document, Q_{CE} is defined as the nominal strength, Q_N, multiplied by 1.25, unless otherwise noted in Chapters 7 through 10.

(b) Force-controlled actions. Force-controlled actions in structural or nonstructural components or elements are those responses generally characterized by the Type 3 curve and in some cases by the Type 2 curve in Figure 6-1. Acceptance criteria for the capacity of these components or elements are provided in Chapter 7, and the components or elements shall be evaluated in accordance with the provisions of this paragraph. The value of force-controlled design action, Q_{UF}, need not exceed the maximum action that can be developed in a component considering the nonlinear behavior of the structure. In lieu of more rational analysis, design actions may be calculated according to Equation 6-4a or 6-4b. Note that Q_E has been determined from the pseudo-lateral load, V, defined in Paragraph (2) above as the basic spectral response force, S_aW, modified by C_1, C_2, and C_3 to represent the expected deformation in the building. In Equation 6-4b, Q_E is divided by the modification factors to restore Q_E to a force-controlled action. The force delivery factor, J, in Equation 6-4a, represents an approximation of the additional reduction in the force delivered to a force-controlled component or element by a yielding component of the seismic framing system.

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3 J} \qquad (6\text{-}4a)$$

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3} \qquad (6\text{-}4b)$$

where:

Q_{UF} = design actions due to gravity loads and earthquake loads.

$C_1C_2C_3$ = coefficients as defined in Paragraph 5-4f.

J = a force-delivery reduction factor given by Equation 6-5.

The coefficient J shall be established using Equation 6-5:

$$J = 1.0 + S_{DS}, \text{ not to exceed } 2 \qquad (6\text{-}5)$$

where:

S_{DS} = spectral acceleration, described in Chapter 3.

Equation 6-4b can be used in all cases. Equation 6-4a can only be used if the forces contributing to Q_{UF} are delivered by yielding components of the seismic framing system. Force-controlled actions in primary and secondary components and elements shall satisfy Equation 6-6.

$$Q_{CL} \geq Q_U \qquad (6\text{-}6)$$

where:

Q_{CL} = lower-bound strength of a component or element at the deformation level under consideration for force-controlled actions.

For Q_{CL}, the lower-bound strength shall be determined considering all co-existing actions acting on the component under the design loading condition. In this document, Q_{CL} is defined as the nominal strength Q_N multiplied by the appropriate capacity reduction factor, ϕ, unless otherwise noted in Chapters 7 through 10.

(3) Nonstructural components. As indicated in Paragraph 4-12, the minimum performance objective for all nonstructural components will be similar to structural Performance Objective 1A, and the acceptance criteria are satisfied by compliance with Chapter 6 of FEMA 301 with a component importance factor of 1.0. Selected nonstructural components shall be assigned component importance factors, in accordance with Paragraph 10-1d, regardless of the structural performance objectives of the building.

b. Nonlinear Static Procedure.

(1) General. This procedure shall be used for the evaluation of structures in Seismic Use Groups II and III, with the characteristics described in Paragraph 5-4b. Acceptance criteria are also provided for this procedure to satisfy Performance Objective 1A, but the use of this procedure for that performance objective requires specific authorization from the cognizant design authority.

(2) Actions and Deformations. With the procedures as described in Paragraph 5-4, compliance with the performance objective requires compliance with the global displacement criteria for the structure as a whole, and the local deformation criteria for individual structural elements.

(a) Global displacement. The displacement for the control node of the structure in the force/displacement plot (i.e., pushover analysis) must equal or exceed the target displacement, δ_t, described in Paragraph 5-4f. Story drifts shall not exceed the values indicated in Table 6-1.

(b) Deformation-controlled actions. Primary and secondary components shall have expected deformation capacities not less than the deformations derived from the pushover analysis when the target displacement, δ_t is attained. Modeling parameters and numerical acceptance criteria are provided for each performance objective for the structural systems described in Chapters 7 through 10. The acceptance criteria are provided in terms of rotations, θ, in radians; rotation ratios, θ/θ_y; or deformation ratios Δ/Δ_y, as depicted in Figure 6-2.

1. Steel moment and braced frames. Acceptance criteria are provided in terms of either plastic rotations of ratios or plastic rotations to yield rotations. Figure 6-3 illustrates the definition of chord rotation for frame beams and columns. If it is assumed that the total chord rotation, θ, (elastic plus plastic rotation) is defined by the interstory drift, Δ/h, then the interstory drift ratio becomes a convenient parameter to monitor the inelastic deformations by subtraction of the yield deformations, θ_y.

i. For beams:

$$\theta_y = \frac{ZF_{ye}\ell_b}{6EI_b} \qquad (6\text{-}7)$$

ii. For columns:

$$\theta_y = \frac{ZF_{ye}\ell_c}{6EI_c}\left(1-\frac{P}{P_{ye}}\right) \qquad (6\text{-}8)$$

where:

Z = Plastic section modulus, in^3 (mm^3).

F_{ye} = Expected yield strength, psi (kPa), as defined in the AISC Seismic provisions.

I_b = Moment of inertia of beams, in^4 (mm^4).

I_c = Moment of inertia of columns, in^4 (mm^4).

ℓ_b = Beam length, in (mm).

ℓ_c = Column length, in (mm).

P = Axial force in the columns, kips (kN).

P_{ye} = Expected axial yield strength, $A_g F_{ye}$ kips (kN).

iii. For beams in partially restrained moment frames, EI_b in Equation 6-7 is modified to:

$$EI_b\ (\text{adjusted}) = \frac{1}{\dfrac{6h}{\ell_b^2 K_\Theta} + \dfrac{1}{EI_b}} \qquad (6\text{-}9)$$

where:

h = Average story height of the columns, in. (mm) K_θ = Rotational spring stiffness, estimated as $M_{CE}/0.005$, kip-in per rad ($M_{CE}/0.044$, kN-m per rad.).

M_{CE} = Expected moment capacity of the connection, kip-in. (kN-m)

iv. For link beams in eccentric braced frames:

$$\theta_y = Q_{CE}/eK_e \qquad (6\text{-}10)$$

where:

θ_y = Yield deformation of the link, rad.

Q_{CE} = Expected shear strength of link beam, kips (kN) = 0.6 $F_{ye}\,A_w$

F_{ye} = Expected yield strength, ksi (kPa)

A_w = Area of link beam $(d_b\text{-}2t_f)t_w$, in^2 (mm^2)

d_b = Depth of link beam, in (mm).

t_f = Thickness of link beam flanges, in. (mm).

t_w = Thickness of link beam web, in. (mm)

A_w = Area of link beam web, in^2 (mm^2)

e = Length of link beam, in. (mm)

K_e = Stiffness of link beam, kip/in (kN/mm) = $\dfrac{K_s K_b}{K_s + K_b}$

K_s = Shear stiffness of link beam, kip/in (kN/mm) = $\dfrac{GA_w}{e}$

G = Shear modulus, kips/in^2 (kPa)

K_b = Flexural stiffness of link beam, kips/in (kN/mm) = $12EI_b/e^3$.

2. Concrete moment frames. Acceptance criteria for reinforced concrete beams, columns, and beam/column joints in moment frames are tabulated in Chapter 7. The numerical values are given as the plastic rotation angles in radians as defined in Figure 6-2. As described in Paragraph 6-3b(2)(b)1 above, the total chord rotation may be assumed to be equal to the interstory drift ratio, Δ/h, and the yield chord rotation, θ_y, for beams and columns is assumed to be:

$$\theta_y = \frac{M_{CE}\,d}{E_c I_g} \qquad (6\text{-}11)$$

where:

M_{CE} = Expected moment capacity of the beam or column with the design axial load and F_y for the reinforcement equal to F_{ye}.

d = Depth of the beam or column, in. (mm).

E_c = Elastic modulus of the concrete, ksi (kPa).

I_g = Gross moment of inertia of the beam of column, in^4 (mm^4).

(Note that in Equation 6-11, the yield curvature, N_y, is calculated with $I_g/2$ and the plastic hinge length is assumed to be $d/2$.)

3. Reinforced concrete shear walls.

i. Controlled by flexure. For shear walls in which the vertical reinforcement is expected to yield in flexure prior to the wall exceeding its shear capacity, the acceptance criteria in Table 7-4 are provided in terms of the plastic rotation, 2, as indicated in Figure 6-4 and are similar to that for concrete moment-resisting frames in Paragraph 2 above with the depth, d, in Equation 6-11 to be replaced by the length of the wall.

ii. Controlled by shear. For shear walls when the shear capacity is attained prior to flexural yielding of the reinforcement, the tabulated acceptance criteria values in Table 7-5 represent allowable values of the interstory drift ratio,)/h, with reference to Figure 6-2b, and it is not necessary to determine) y.

iii. Coupling beams. The acceptance criteria in Table 7-4, for coupling beams controlled by flexure, are evaluated as shown in Figure 6-5 for beams in moment frames. Coupling beams controlled by shear are evaluated as indicated above for walls controlled by shear, and the acceptance criteria are tabulated in Table 7-5.

4. Reinforced masonry shear walls. The acceptance criteria for these shear walls, tabulated in Table 7-9, are provided in terms of drift ratios,)/h, as indicated in Figure 6-2b. Acceptance criteria for coupling beams for reinforced masonry walls are similar to criteria for coupling beams in reinforced concrete shear walls described in Paragraph iii above.

(c) Force-controlled actions. Structural components shall have lower-bound strengths, Q_{CL}, not less than the required strength, Q_{UF}, from the appropriate combinations of seismic and gravity load effects. Lower-bound strengths, Q_{CL}, are defined in Paragraph 6-3a(3)b and in Chapters 7 through 10.

(d). Reanalysis. The results of the analysis must be carefully monitored to determine whether any of the structural components have exceeded the deformation limits indicated in Chapters 7 through 10 for the desired performance objectives. Minor exceedance (i.e., 10 to 15 percent) of the deformation limits in a limited number of components may be acceptable if it can be demonstrated that the additional deformation does not have an adverse effect on the performance of the structure. All other components with excessive deformations should be strengthened to meet the acceptance criteria. If the revised member sizes for the components are significant, a reanalysis may be required to confirm an acceptable response. Similarly, if the results of the analysis indicate that a number of the components or elements are overdesigned by a factor of 10 to 15 percent, the overdesigned components or elements shall be redesigned, and the analysis reported, unless it can be demonstrated that the overdesign is cost-effective, or otherwise beneficial. If the structural members are required to be substantially stronger or stiffer, as compared to the design for gravity loads, the designer should consider the use of a supplementary structural system; such as the use of shear walls or braced frames to stiffen a flexible moment frame system.

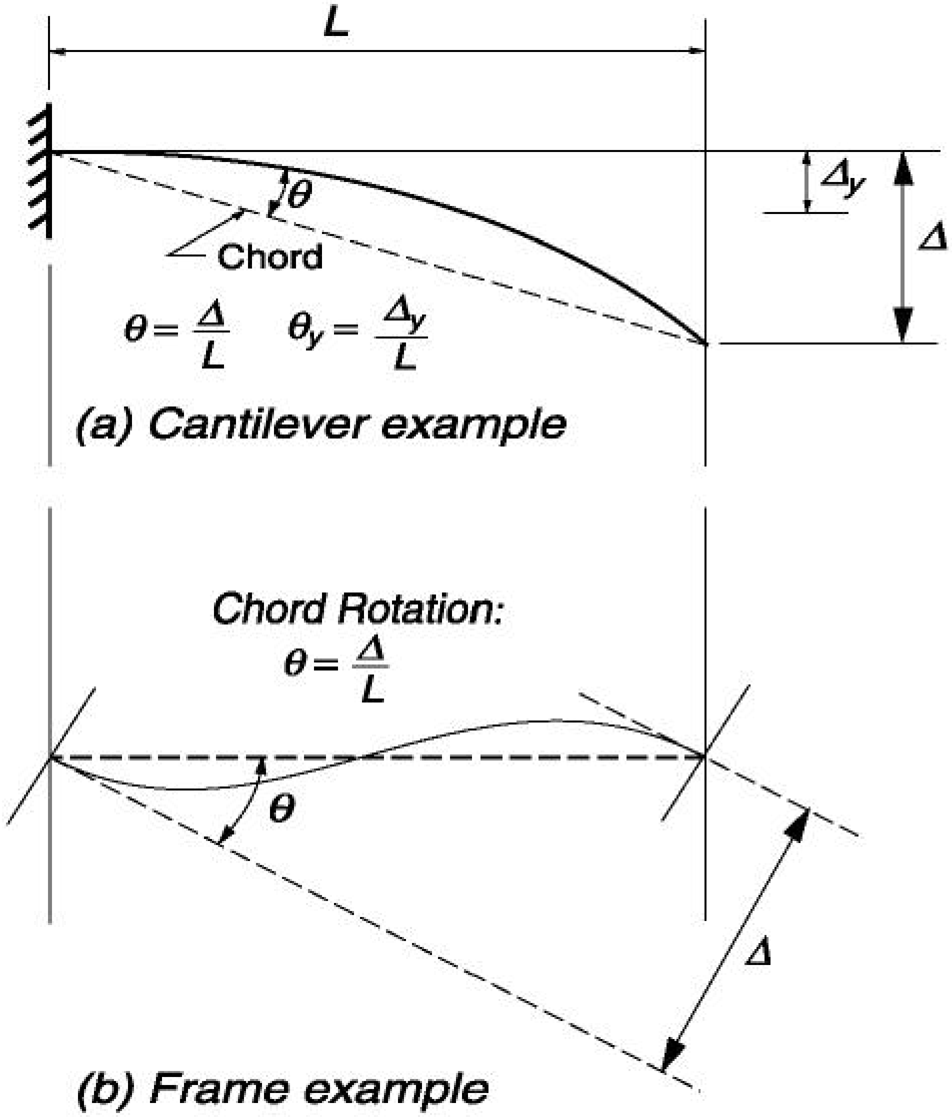

Figure 6-3 Definition of Chord Rotation

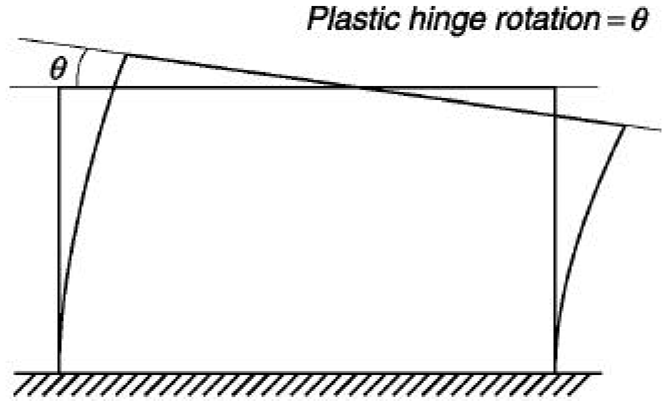

Figure 6-4 Plastic Hinge Rotation in Shear Wall where Flexure Dominates Inelastic Response

3. Reinforced concrete shear walls.

i. Controlled by flexure. For shear walls in which the vertical reinforcement is expected to yield in flexure prior to the wall exceeding its shear capacity, the acceptance criteria in Table 7-4 are provided in terms of the plastic rotation, θ, as indicated in Figure 6-4 and are similar to that for concrete moment-resisting frames in Paragraph 2 above with the depth, *d*, in Equation 6-11 to be replaced by the length of the wall.

ii. Controlled by shear. For shear walls when the shear capacity is attained prior to flexural yielding of the reinforcement, the tabulated acceptance criteria values in Table 7-5 represent allowable values of the interstory drift ratio, Δ/h, with reference to Figure 6-2b, and it is not necessary to determine Δy.

iii. Coupling beams. The acceptance criteria in Table 7-4, for coupling beams controlled by flexure, are evaluated as shown in Figure 6-5 for beams in moment frames. Coupling beams controlled by shear are evaluated as indicated above for walls controlled by shear, and the acceptance criteria are tabulated in Table 7-5.

4. Reinforced masonry shear walls. The acceptance criteria for these shear walls, tabulated in Table 7-9, are provided in terms of drift ratios, Δ/h, as indicated in Figure 6-2b. Acceptance criteria for coupling beams for reinforced masonry walls are similar to criteria for coupling beams in reinforced concrete shear walls described in Paragraph iii above.

(c) Force-controlled actions. Structural components shall have lower-bound strengths, Q_{CL}, not less than the required strength, Q_{UF}, from the appropriate combinations of seismic and gravity load effects. Lower-bound strengths, Q_{CL}, are defined in Paragraph 6-3a(3)b and in Chapters 7 through 10.

(d). Reanalysis. The results of the analysis must be carefully monitored to determine whether any of the structural components have exceeded the deformation limits indicated in Chapters 7 through 10 for the desired performance objectives. Minor exceedance (i.e., 10 to 15 percent) of the deformation limits in a limited number of components may be acceptable if it can be demonstrated that the additional deformation does not have an adverse effect on the performance of the structure. All other components with excessive deformations should be strengthened to meet the acceptance criteria. If the revised member sizes for the components are significant, a reanalysis may be required to confirm an acceptable response. Similarly, if the results of the analysis indicate that a number of the components or elements are overdesigned by a factor of 10 to 15 percent, the overdesigned components or elements shall be redesigned, and the analysis reported, unless it can be demonstrated that the overdesign is cost-effective, or otherwise beneficial. If the structural members are required to be substantially stronger or stiffer, as compared to the design for gravity loads, the designer should consider the use of a supplementary structural system; such as the use of shear walls or braced frames to stiffen a flexible moment frame system.

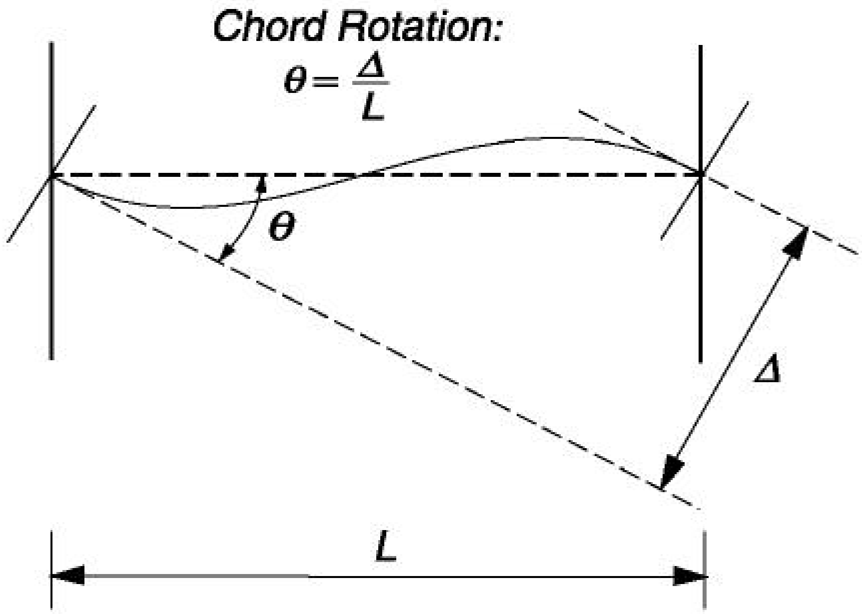

Figure 6-5   Chord Rotation for Shear Wall Coupling Beams