3. Modeling and Analysis (Systematic Rehabilitation)

3.1 Scope

This chapter presents Analysis Procedures and design requirements for seismic rehabilitation of existing buildings. Section 3.2 presents general requirements for analysis and design that are relevant to all four Analysis Procedures presented in this chapter. The four Analysis Procedures for seismic rehabilitation are presented in Section 3.3, namely: Linear Static Procedure, Linear Dynamic Procedure, Nonlinear Static Procedure, and Nonlinear Dynamic Procedure. Modeling and analysis assumptions, and procedures for determination of design actions and design deformations, are also presented in Section 3.3. Acceptance criteria for elements and components analyzed using any one of the four procedures presented in Section 3.3 are provided in Section 3.4. Section 3.5 provides definitions for key terms used in this chapter, and Section 3.6 defines the symbols used in this chapter. Section 3.7 contains a list of references.

The relationship of the Analysis Procedures described in this chapter with specifications in other chapters in the *Guidelines* is as follows.

- Information on Rehabilitation Objectives to be used for design, including hazard levels (that is, earthquake shaking) and on Performance Levels, is provided in Chapter 2.

- The provisions set forth in this chapter are intended for Systematic Rehabilitation only. Provisions for Simplified Rehabilitation are presented in Chapter 10.

- Guidelines for selecting an appropriate Analysis Procedure are provided in Chapter 2. Chapter 3 describes the loading requirements, mathematical model, and detailed analytical procedures required to estimate seismic force and deformation demands on elements and components of a building. Information on the calculation of appropriate stiffness and strength characteristics for components and elements is provided in Chapters 4 through 9.

- General requirements for analysis and design, including requirements for multidirectional excitation effects, P-Δ effects, torsion, and overturning; basic analysis requirements for the linear and nonlinear procedures; and basic design requirements for diaphragms, walls, continuity of the framing system, building separation, structures sharing common components, and nonstructural components are given in Section 2.11.

- Component strength and deformation demands obtained from analysis using procedures described in this chapter, based on component acceptance criteria outlined in this chapter, are compared with permissible values provided in Chapters 4 through 9 for the desired Performance Level.

- Design methods for walls subjected to out-of-plane seismic forces are addressed in Chapter 2. Analysis and design methods for nonstructural components, and mechanical and electrical equipment, are presented in Chapter 11.

- Specific analysis and design requirements for buildings incorporating seismic isolation and/or supplemental damping hardware are given in Chapter 9.

3.2 General Requirements

Modeling, analysis, and evaluation for Systematic Rehabilitation shall follow the guidelines of this chapter.

3.2.1 Analysis Procedure Selection

Four procedures are presented for seismic analysis of buildings: two linear procedures, and two nonlinear procedures. The two linear procedures are termed the Linear Static Procedure (LSP) and the Linear Dynamic Procedure (LDP). The two nonlinear procedures are termed the Nonlinear Static Procedure (NSP) and Nonlinear Dynamic Procedure (NDP).

Either the linear procedures of Section 3.3.1 and Section 3.3.2, or the nonlinear procedures of Sections 3.3.3 and 3.3.4, may be used to analyze a building, subject to the limitations set forth in Section 2.9.

3.2.2 Mathematical Modeling

3.2.2.1 Basic Assumptions

In general, a building should be modeled, analyzed, and evaluated as a three-dimensional assembly of elements and components. Three-dimensional mathematical models shall be used for analysis and evaluation of buildings with plan irregularity (see Section 3.2.3).

Two-dimensional modeling, analysis, and evaluation of buildings with stiff or rigid diaphragms (see Section 3.2.4) is acceptable if torsional effects are either sufficiently small to be ignored, or indirectly captured (see Section 3.2.2.2).

Vertical lines of seismic framing in buildings with flexible diaphragms (see Section 3.2.4) may be individually modeled, analyzed, and evaluated as two-dimensional assemblies of components and elements, or a three-dimensional model may be used with the diaphragms modeled as flexible elements.

Explicit modeling of a connection is required for nonlinear procedures if the connection is weaker than the connected components, and/or the flexibility of the connection results in a significant increase in the relative deformation between the connected components.

3.2.2.2 Horizontal Torsion

The effects of horizontal torsion must be considered. The total torsional moment at a given floor level shall be set equal to the sum of the following two torsional moments:

- The actual torsion; that is, the moment resulting from the eccentricity between the centers of mass at all floors above and including the given floor, and the center of rigidity of the vertical seismic elements in the story below the given floor, and

- The accidental torsion; that is, an accidental torsional moment produced by horizontal offset in the centers of mass, at all floors above and including the given floor, equal to a minimum of 5% of the horizontal dimension at the given floor level measured perpendicular to the direction of the applied load.

In buildings with rigid diaphragms the effect of actual torsion shall be considered if the maximum lateral displacement from this effect at any point on any floor diaphragm exceeds the average displacement by more than 10%. The effect of accidental torsion shall be considered if the maximum lateral displacement due to this effect at any point on any floor diaphragm exceeds the average displacement by more than 10%. This effect shall be calculated independent of the effect of actual torsion.

If the effects of torsion are required to be investigated, the increased forces and displacements resulting from horizontal torsion shall be evaluated and considered for design. The effects of torsion cannot be used to reduce force and deformation demands on components and elements.

For linear analysis of buildings with rigid diaphragms, when the ratio $\delta_{max} / \delta_{avg}$ due to total torsional moment exceeds 1.2, the effect of accidental torsion shall be amplified by a factor, A_x:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \qquad (3\text{-}1)$$

where:

δ_{max} = Maximum displacement at any point of the diaphragm at level x

δ_{avg} = Average of displacements at the extreme points of the diaphragm at level x

A_x need not exceed 3.0.

If the ratio η of (1) the maximum displacement at any point on any floor diaphragm (including torsional amplification), to (2) the average displacement, calculated by rational analysis methods, exceeds 1.50, three-dimensional models that account for the spatial distribution of mass and stiffness shall be used for analysis and evaluation. Subject to this limitation, the effects of torsion may be indirectly captured for analysis of two-dimensional models as follows.

- For the LSP (Section 3.3.1) and the LDP (Section 3.3.2), the design forces and displacements shall be increased by multiplying by the maximum value of η calculated for the building.

- For the NSP (Section 3.3.3), the target displacement shall be increased by multiplying by the maximum value of η calculated for the building.
- For the NDP (Section 3.3.4), the amplitude of the ground acceleration record shall be increased by multiplying by the maximum value of η calculated for the building.

3.2.2.3 Primary and Secondary Actions, Components, and Elements

Components, elements, and component actions shall be classified as either primary or secondary. Primary actions, components, and elements are key parts of the seismic framing system required in the design to resist earthquake effects. These shall be evaluated, and rehabilitated as necessary, to sustain earthquake-induced forces and deformations while simultaneously supporting gravity loads. Secondary actions, components, and elements are not designated as part of the lateral-force-resisting system, but nevertheless shall be evaluated, and rehabilitated as necessary, to ensure that such actions, components, and elements can simultaneously sustain earthquake-induced deformations and gravity loads. (See the *Commentary* on this section.)

For linear procedures (Sections 3.3.1 and 3.3.2), only the stiffness of primary components and elements shall be included in the mathematical model. Secondary components and elements shall be checked for the displacements estimated by such analysis. For linear procedures, the total lateral stiffness of the secondary components and elements shall be no greater than 25% of the total stiffness of the primary components and elements, calculated at each level of the building. If this limit is exceeded, some secondary components shall be reclassified as primary components.

For nonlinear procedures (Sections 3.3.3 and 3.3.4), the stiffness and resistance of all primary and secondary components (including strength loss of secondary components) shall be included in the mathematical model. Additionally, if the total stiffness of the nonstructural components—such as precast exterior panels—exceeds 10% of the total lateral stiffness of a story, the nonstructural components shall be included in the mathematical model.

The classification of components and elements shall not result in a change in the classification of a building's configuration (see Section 3.2.3); that is, components and elements shall not be selectively assigned as either primary or secondary to change the configuration of a building from *irregular* to *regular.*

3.2.2.4 Deformation- and Force-Controlled Actions

Actions shall be classified as either deformation-controlled or force-controlled. A deformation-controlled action is one that has an associated deformation that is allowed to exceed the yield value; the maximum associated deformation is limited by the ductility capacity of the component. A force-controlled action is one that has an associated deformation that is not allowed to exceed the yield value. Actions with limited ductility (such as allowing $a < g$ in Figure 2-4) may also be considered force-controlled. Guidance on these classifications may be found in Chapters 5 through 8.

3.2.2.5 Stiffness and Strength Assumptions

Element and component stiffness properties and strength estimates for both linear and nonlinear procedures shall be determined from information given in Chapters 4 through 9, and 11. Guidelines for modeling structural components are given in Chapters 5 through 8. Similar guidelines for modeling foundations and nonstructural components are given in Chapters 4 and 11, respectively.

3.2.2.6 Foundation Modeling

The foundation system may be included in the mathematical model for analysis with stiffness and damping properties as defined in Chapter 4. Otherwise, unless specifically prohibited, the foundation may be assumed to be rigid and not included in the mathematical model.

3.2.3 Configuration

Building irregularities are discussed in Section 2.9. Such classification shall be based on the plan and vertical configuration of the framing system, using a mathematical model that considers both primary and secondary components.

One objective of seismic rehabilitation should be the improvement of the regularity of a building through the judicious placement of new framing elements.

3.2.4 Floor Diaphragms

Floor diaphragms transfer earthquake-induced inertial forces to vertical elements of the seismic framing system. Roof diaphragms are considered to be floor diaphragms. Connections between floor diaphragms and vertical seismic framing elements must have sufficient strength to transfer the maximum calculated diaphragm shear forces to the vertical framing elements. Requirements for design and detailing of diaphragm components are given in Section 2.11.6.

Floor diaphragms shall be classified as either flexible, stiff, or rigid. (See Chapter 10 for classification of diaphragms to be used for determining whether Simplified Rehabilitation Methods are applicable.) Diaphragms shall be considered flexible when the maximum lateral deformation of the diaphragm along its length is more than twice the average interstory drift of the story immediately below the diaphragm. For diaphragms supported by basement walls, the average interstory drift of the story above the diaphragm may be used in lieu of the basement story. Diaphragms shall be considered rigid when the maximum lateral deformation of the diaphragm is less than half the average interstory drift of the associated story. Diaphragms that are neither flexible nor rigid shall be classified as stiff. The interstory drift and diaphragm deformations shall be estimated using the seismic lateral forces (Equation 3-6). The in-plane deflection of the floor diaphragm shall be calculated for an in-plane distribution of lateral force consistent with the distribution of mass, as well as all in-plane lateral forces associated with offsets in the vertical seismic framing at that floor.

Mathematical models of buildings with stiff or flexible diaphragms should be developed considering the effects of diaphragm flexibility. For buildings with flexible diaphragms at each floor level, the vertical lines of seismic framing may be designed independently, with seismic masses assigned on the basis of tributary area.

3.2.5 P-Δ Effects

Two types of P-Δ (second-order) effects are addressed in the *Guidelines*: (1) static P-Δ and (2) dynamic P-Δ.

3.2.5.1 Static P-Δ Effects

For linear procedures, the stability coefficient θ should be evaluated for each story in the building using Equation 2-14. This process is iterative. The story drifts calculated by linear analysis, δ_i in Equation 2-14, shall be increased by $1/(1 - \theta_i)$ for evaluation of the stability coefficient. If the coefficient is less than 0.1 in all stories, static P-Δ effects will be small and may be ignored. If the coefficient exceeds 0.33, the building may be unstable and redesign is necessary (Section 2.11.2). If the coefficient lies between 0.1 and 0.33, the seismic force effects in story *i* shall be increased by the factor $1/(1 - \theta_i)$.

For nonlinear procedures, second-order effects shall be considered directly in the analysis; the geometric stiffness of all elements and components subjected to axial forces shall be included in the mathematical model.

3.2.5.2 Dynamic P-Δ Effects

Dynamic P-Δ effects may increase component actions and deformations, and story drifts. Such effects are indirectly evaluated for the linear procedures and the NSP using the coefficient C_3. Refer to Sections 3.3.1.3A and 3.3.3.3A for additional information.

Second-order effects shall be considered directly for nonlinear procedures; the geometric stiffness of all elements and components subjected to axial forces shall be included in the mathematical model.

3.2.6 Soil-Structure Interaction

Soil-structure interaction (SSI) may modify the seismic demand on a building. Two procedures for computing the effects of SSI are provided below. Other rational methods of modeling SSI may also be used.

For those rare cases (such as for near-field and soft soil sites) in which the increase in fundamental period due to SSI increases spectral accelerations, the effects of SSI on building response must be evaluated; the increase in fundamental period may be calculated using the simplified procedures referred to in Section 3.2.6.1. Otherwise, the effects of SSI may be ignored. In addition, SSI effects need not be considered for any building permitted to be rehabilitated using the Simplified Rehabilitation Method (Table 10-1).

The simplified procedures referred to in Section 3.2.6.1 can be used with the LSP of Section 3.3.1. Consideration of SSI effects with the LDP of Section 3.3.2, the NSP of Section 3.3.3, and the NDP of

Section 3.3.4 shall include explicit modeling of foundation stiffness as in Section 3.2.6.2. Modal damping ratios may be calculated using the method referred to in Section 3.2.6.1.

Soil-structure interaction effects shall not be used to reduce component and element actions by more than 25%.

3.2.6.1 Procedures for Period and Damping

The simplified procedures presented in Chapter 2 of the 1997 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (BSSC, 1997) may be used to calculate seismic demands using the effective fundamental period $\tilde{T}$ and effective fundamental damping ratio $\tilde{\beta}$ of the foundation-structure system.

3.2.6.2 Explicit Modeling of SSI

Soil-structure interaction may be modeled explicitly by modeling the stiffness and damping for individual foundation elements. Guidance on the selection of spring characteristics to represent foundation stiffness is presented in Section 4.4.2. Unless otherwise determined, the damping ratio for individual foundation elements shall be set equal to that value of the damping ratio used for the elastic superstructure. For the NSP, the damping ratio of the foundation-structure system $\tilde{\beta}$ shall be used to calculate the spectral demands.

3.2.7 Multidirectional Excitation Effects

Buildings shall be designed for seismic forces in any horizontal direction. For regular buildings, seismic displacements and forces may be assumed to act nonconcurrently in the direction of each principal axis of a building. For buildings with plan irregularity (Section 3.2.3) and buildings in which one or more components form part of two or more intersecting elements, multidirectional excitation effects shall be considered. Multidirectional effects on components shall include both torsional and translational effects.

The requirement that multidirectional (orthogonal) excitation effects be considered may be satisfied by designing elements or components for the forces and deformations associated with 100% of the seismic displacements in one horizontal direction plus the forces associated with 30% of the seismic displacements in the perpendicular horizontal direction. Alternatively, it is acceptable to use SRSS to combine multidirectional effects where appropriate.

The effects of vertical excitation on horizontal cantilevers and prestressed elements shall be considered by static or dynamic response methods. Vertical earthquake shaking may be characterized by a spectrum with ordinates equal to 67% of those of the horizontal spectrum (Section 2.6.1.5) unless alternative vertical response spectra are developed using site-specific analysis.

3.2.8 Component Gravity Loads and Load Combinations

The following component gravity forces, Q_G, shall be considered for combination with seismic loads.

When the effects of gravity and seismic loads are additive,

$$Q_G = 1.1(Q_D + Q_L + Q_S) \tag{3-2}$$

When the effects of gravity counteract seismic loads,

$$Q_G = 0.9Q_D \tag{3-3}$$

where:

Q_D = Dead load effect (action)

Q_L = Effective live load effect (action), equal to 25% of the unreduced design live load but not less than the measured live load

Q_S = Effective snow load effect (action), equal to either 70% of the full design snow load or, where conditions warrant and approved by the regulatory agency, not less than 20% of the full design snow load, except that where the design snow load is 30 pounds per square foot or less, Q_S = 0.0

Evaluation of components for gravity and wind forces, in the absence of earthquake forces, is beyond the scope of this document.

3.2.9 Verification of Design Assumptions

Each component shall be evaluated to determine that assumed locations of inelastic deformations are consistent with strength and equilibrium requirements at all locations along the component length. Further, each component should be evaluated by rational analysis for adequate post-earthquake residual gravity load capacity, considering reduction of stiffness caused by earthquake damage to the structure.

Where moments in horizontally-spanning primary components, due to the gravity load combinations of Equations 3-2 and 3-3, exceed 50% of the expected moment strength at any location, the possibility for inelastic flexural action at locations other than component ends shall be specifically investigated by comparing flexural actions with expected component strengths, and the post-earthquake gravity load capacity should be investigated. Sample checking procedures are presented in the *Commentary*. Formation of flexural plastic hinges away from component ends is not permitted unless it is explicitly accounted for in modeling and analysis.

3.3 Analysis Procedures

3.3.1 Linear Static Procedure (LSP)

3.3.1.1 Basis of the Procedure

Under the Linear Static Procedure (LSP), design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using a linearly-elastic, static analysis. Restrictions on the applicability of this procedure are given in Section 2.9.

In the LSP, the building is modeled with linearly-elastic stiffness and equivalent viscous damping that approximate values expected for loading to near the yield point. Design earthquake demands for the LSP are represented by static lateral forces whose sum is equal to the pseudo lateral load defined by Equation 3-6. The magnitude of the pseudo lateral load has been selected with the intention that when it is applied to the linearly elastic model of the building it will result in design displacement amplitudes approximating maximum displacements that are expected during the design earthquake. If the building responds essentially elastically to the design earthquake, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. If the building responds inelastically to the design earthquake, as will commonly be the case, the internal forces that would develop in the yielding building will be less than the internal forces calculated on an elastic basis.

Results of the LSP are to be checked using the applicable acceptance criteria of Section 3.4. Calculated internal forces typically will exceed those that the building can develop, because of anticipated inelastic response of components and elements. These obtained design forces are evaluated through the acceptance criteria of Section 3.4.2, which include modification factors and alternative Analysis Procedures to account for anticipated inelastic response demands and capacities.

3.3.1.2 Modeling and Analysis Considerations

Period Determination. The fundamental period of a building, in the direction under consideration, shall be calculated by one of the following three methods. (Method 1 is preferred.)

Method 1. Eigenvalue (dynamic) analysis of the mathematical model of the building. The model for buildings with flexible diaphragms shall consider representation of diaphragm flexibility unless it can be shown that the effects of omission will not be significant.

Method 2. Evaluation of the following equation:

$$T = C_t h_n^{3/4} \tag{3-4}$$

where:

T = Fundamental period (in seconds) in the direction under consideration

C_t = 0.035 for moment-resisting frame systems of steel

= 0.030 for moment-resisting frames of reinforced concrete

= 0.030 for eccentrically-braced steel frames

= 0.020 for all other framing systems

= 0.060 for wood buildings (types 1 and 2 in Table 10-2)

h_n = Height (in feet) above the base to the roof level

Method 2 is not applicable to unreinforced masonry buildings with flexible diaphragms.

Method 3. The fundamental period of a one-story building with a single span flexible diaphragm may be calculated as:

$$T = (0.1\Delta_w + 0.078\Delta_d)^{0.5} \quad (3\text{-}5)$$

where Δ_w and Δ_d are in-plane wall and diaphragm displacements in inches, due to a lateral load, in the direction under consideration, equal to the weight tributary to the diaphragm (see *Commentary*, Figure C3-2). For multiple-span diaphragms, a lateral load equal to the gravity weight tributary to the diaphragm span under consideration shall be applied to each diaphragm span to calculate a separate period for each diaphragm span. The period so calculated that maximizes the pseudo lateral load (see Equation 3-6) shall be used for design of all walls and diaphragm spans in the building.

3.3.1.3 Determination of Actions and Deformations

A. Pseudo Lateral Load

The pseudo lateral load in a given horizontal direction of a building is determined using Equation 3-6. This load, increased as necessary to account for the effects of torsion (see Section 3.2.2.2), shall be used for the design of the vertical seismic framing system.

$$V = C_1 C_2 C_3 S_a W \quad (3\text{-}6)$$

where:

V = Pseudo lateral load

This force, when distributed over the height of the linearly-elastic analysis model of the structure, 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 3-6 may be significantly larger than the actual strength of the structure to resist this force. The acceptance criteria in Section 3.4.2 are developed to take this aspect into account.

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response. C_1 may be calculated using the procedure indicated in Section 3.3.3.3. with the elastic base shear capacity substituted for V_y. Alternatively, C_1 may be calculated as follows:

$C_1 = 1.5$ for $T < 0.10$ second

$C_1 = 1.0$ for $T \geq T_0$ second

Linear interpolation shall be used to calculate C_1 for intermediate values of T.

T = Fundamental period of the building in the direction under consideration. If soil-structure interaction is considered, the effective fundamental period $\tilde{T}$ shall be substituted for T.

T_0 = Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum. (See Sections 2.6.1.5 and 2.6.2.1.)

C_2 = Modification factor to represent the effect of stiffness degradation and strength deterioration on maximum displacement response. Values of C_2 for different framing systems and Performance Levels are listed in Table 3-1. Linear interpolation shall be used to estimate values for C_2 for intermediate values of T.

C_3 = Modification factor to represent increased displacements due to dynamic P-Δ effects. This effect is in addition to the consideration of static P-D effects as defined in Section 3.2.5.1. For values of the stability coefficient θ (see Equation 2-14) less than 0.1, C_3 may be set equal to 1.0. For values of θ greater than 0.1, C_3 shall be calculated as $1 + 5(\theta - 0.1)/T$. The maximum value of θ for all stories in the building shall be used to calculate C_3.

S_a = Response spectrum acceleration, at the fundamental period and damping ratio of the building in the direction under consideration. The value of S_a shall be obtained from the procedure in Section 2.6.1.5.

W = Total dead load and anticipated live load as indicated below:

- In storage and warehouse occupancies, a minimum of 25% of the floor live load
- The actual partition weight or minimum weight of 10 psf of floor area, whichever is greater
- The applicable snow load—see the *NEHRP Recommended Provisions* (BSSC, 1995)
- The total weight of permanent equipment and furnishings

B. Vertical Distribution of Seismic Forces

The lateral load F_x applied at any floor level *x* shall be determined from the following equations:

$$F_x = C_{vx}V \tag{3-7}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{3-8}$$

where:

k = 1.0 for $T \le 0.5$ second

= 2.0 for $T \ge 2.5$ seconds

Linear interpolation shall be used to estimate values of k for intermediate values of T.

C_{vx} = Vertical distribution factor

V = Pseudo lateral load from Equation 3-6

w_i = Portion of the total building weight W located on or assigned to floor level i

w_x = Portion of the total building weight W located on or assigned to floor level x

h_i = Height (in ft) from the base to floor level i

h_x = Height (in ft) from the base to floor level x

C. Horizontal Distribution of Seismic Forces

The seismic forces at each floor level of the building shall be distributed according to the distribution of mass at that floor level.

D. Floor Diaphragms

Floor diaphragms shall be designed to resist the effects of (1) the inertia forces developed at the level under consideration (equal to F_{px} in Equation 3-9), and (2) the horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm. Forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements shall be taken to be equal to the elastic forces (Equation 3-6) without reduction, unless smaller forces can be justified by rational analysis.

$$F_{px} = \frac{1}{C_1 C_2 C_3} \sum_{i=x}^{n} F_i \frac{w_x}{\sum_{i=x}^{n} w_i} \tag{3-9}$$

where:

F_{px} = Total diaphragm force at level x

F_i = Lateral load applied at floor level i given by Equation 3-7

w_i = Portion of the total building weight W located on or assigned to floor level i

w_x = Portion of the total building weight W located on or assigned to floor level x

Coefficients C_1, C_2, and C_3 are described above in Section 3.3.1.3A.

The lateral seismic load on each flexible diaphragm shall be distributed along the span of that diaphragm, considering its displaced shape.

E. Determination of Deformations

Structural deformations and story drifts shall be calculated using lateral loads in accordance with Equations 3-6, 3-7, and 3-9 and stiffnesses obtained from Chapters 5, 6, 7, and 8.

Table 3-1 Values for Modification Factor C_2

Performance Level	$T = 0.1$ second		$T \geq T_0$ second	
	Framing Type 1[1]	**Framing Type 2**[2]	**Framing Type 1**[1]	**Framing Type 2**[2]
Immediate Occupancy	1.0	1.0	1.0	1.0
Life Safety	1.3	1.0	1.1	1.0
Collapse Prevention	1.5	1.0	1.2	1.0

1. Structures in which more than 30% of the story shear at any level is resisted by components or elements whose strength and stiffness may deteriorate during the design earthquake. Such elements and components include: ordinary moment-resisting frames, concentrically-braced frames, frames with partially-restrained connections, tension-only braced frames, unreinforced masonry walls, shear-critical walls and piers, or any combination of the above.
2. All frames not assigned to Framing Type 1.

3.3.2 Linear Dynamic Procedure (LDP)

3.3.2.1 Basis of the Procedure

Under the Linear Dynamic Procedure (LDP), design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using a linearly-elastic, dynamic analysis. Restrictions on the applicability of this procedure are given in Section 2.9.

The basis, modeling approaches, and acceptance criteria of the LDP are similar to those for the LSP. The main exception is that the response calculations are carried out using either modal spectral analysis or Time-History Analysis. Modal spectral analysis is carried out using linearly-elastic response spectra that are not modified to account for anticipated nonlinear response. As with the LSP, it is expected that the LDP will produce displacements that are approximately correct, but will produce internal forces that exceed those that would be obtained in a yielding building.

Results of the LDP are to be checked using the applicable acceptance criteria of Section 3.4. Calculated displacements are compared directly with allowable values. Calculated internal forces typically will exceed those that the building can sustain because of anticipated inelastic response of components and elements. These obtained design forces are evaluated through the acceptance criteria of Section 3.4.2, which include modification factors and alternative analysis procedures to account for anticipated inelastic response demands and capacities.

3.3.2.2 Modeling and Analysis Considerations

A. General

The LDP shall conform to the criteria of this section. The analysis shall be based on appropriate characterization of the ground motion (Section 2.6.1). The modeling and analysis considerations set forth in Section 3.3.1.2 shall apply to the LDP but alternative considerations are presented below.

The LDP includes two analysis methods, namely, the Response Spectrum and Time-History Analysis Methods. The Response Spectrum Method uses peak modal responses calculated from dynamic analysis of a mathematical model. Only those modes contributing significantly to the response need to be considered. Modal responses are combined using rational methods to estimate total building response quantities. The Time-History Method (also termed Response-History Analysis) involves a time-step-by-time-step evaluation of building response, using discretized recorded or synthetic earthquake records as base motion input. Requirements for the two analysis methods are outlined in C and D below.

B. Ground Motion Characterization

The horizontal ground motion shall be characterized for design by the requirements of Section 2.6 and shall be one of the following:

- A response spectrum (Section 2.6.1.5)
- A site-specific response spectrum (Section 2.6.2.1)
- Ground acceleration time histories (Section 2.6.2.2)

C. Response Spectrum Method

The requirement that all significant modes be included in the response analysis may be satisfied by including sufficient modes to capture at least 90% of the participating mass of the building in each of the building's principal horizontal directions. Modal damping ratios shall reflect the damping inherent in the building at deformation levels less than the yield deformation.

The peak member forces, displacements, story forces, story shears, and base reactions for each mode of response shall be combined by recognized methods to estimate total response. Modal combination by either the SRSS (square root sum of squares) rule or the CQC (complete quadratic combination) rule is acceptable.

Multidirectional excitation effects shall be accounted for by the requirements of Section 3.2.7.

D. Time-History Method

The requirements for the mathematical model for Time-History Analysis are identical to those developed for Response Spectrum Analysis. The damping matrix associated with the mathematical model shall reflect the damping inherent in the building at deformation levels less than the yield deformation.

Time-History Analysis shall be performed using time histories prepared according to the requirements of Section 2.6.2.2.

Response parameters shall be calculated for each Time-History Analysis. If three Time-History Analyses are performed, the maximum response of the parameter of interest shall be used for design. If seven or more pairs of horizontal ground motion records are used for Time-History Analysis, the average response of the parameter of interest may be used for design.

Multidirectional excitation effects shall be accounted for in accordance with the requirements of Section 3.2.7. These requirements may be satisfied by analysis of a three-dimensional mathematical model using simultaneously imposed pairs of earthquake ground motion records along each of the horizontal axes of the building.

3.3.2.3 Determination of Actions and Deformations

A. Modification of Demands

All actions and deformations calculated using either of the LDP analysis methods—Response Spectrum or Time-History Analysis—shall be multiplied by the product of the modification factors C_1, C_2, and C_3 defined in Section 3.3.1.3, and further increased as necessary to account for the effects of torsion (see Section 3.2.2.2). However, floor diaphragm actions need not be increased by the product of the modification factors.

B. Floor Diaphragms

Floor diaphragms shall be designed to resist simultaneously (1) the seismic forces calculated by the LDP, and (2) the horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm. The seismic forces calculated by the LDP shall be taken as not less than 85% of the forces calculated using Equation 3-9. Forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements shall be taken to be equal to the elastic forces without reduction, unless smaller forces can be justified by rational analysis.

3.3.3 Nonlinear Static Procedure (NSP)

3.3.3.1 Basis of the Procedure

Under the Nonlinear Static Procedure (NSP), a model directly incorporating inelastic material response is displaced to a target displacement, and resulting internal deformations and forces are determined. The nonlinear load-deformation characteristics of individual components and elements of the building are modeled directly. The mathematical model of the building is subjected to monotonically increasing lateral forces or displacements until either a target displacement is exceeded or the building collapses. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The target displacement may be calculated by any procedure that accounts for the effects of nonlinear response on displacement amplitude; one rational procedure is presented in Section 3.3.3.3. Because the mathematical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake.

Results of the NSP are to be checked using the applicable acceptance criteria of Section 3.4.3. Calculated displacements and internal forces are compared directly with allowable values.

3.3.3.2 Modeling and Analysis Considerations

A. General

In the context of these *Guidelines*, the NSP involves the monotonic application of lateral forces or displacements to a nonlinear mathematical model of a building until the displacement of the control node in the mathematical model exceeds a target displacement. For buildings that are not symmetric about a plane perpendicular to the applied lateral loads, the lateral loads must be applied in both the positive and negative directions, and the maximum forces and deformations used for design.

The relation between base shear force and lateral displacement of the control node shall be established for control node displacements ranging between zero and 150% of the target displacement, δ_t, given by Equation 3-11. Acceptance criteria shall be based on those forces and deformations (in components and elements) corresponding to a minimum horizontal displacement of the control node equal to δ_t.

Gravity loads shall be applied to appropriate elements and components of the mathematical model during the NSP. The loads and load combination presented in Equation 3-2 (and Equation 3-3 as appropriate) shall be used to represent such gravity loads.

The analysis model shall be discretized in sufficient detail to represent adequately the load-deformation response of each component along its length. Particular attention should be paid to identifying locations of inelastic action along the length of a component, as well as at its ends.

B. Control Node

The NSP requires definition of the control node in a building. These *Guidelines* consider the control node to be the center of mass at the roof of a building; the top of a penthouse should not be considered as the roof. The displacement of the control node is compared with the target displacement—a displacement that characterizes the effects of earthquake shaking.

C. Lateral Load Patterns

Lateral loads shall be applied to the building in profiles that approximately bound the likely distribution of inertia forces in an earthquake. For three-dimensional analysis, the horizontal distribution should simulate the distribution of inertia forces in the plane of each floor diaphragm. For both two- and three-dimensional analysis, at least two vertical distributions of lateral load shall be considered. The first pattern, often termed the uniform pattern, shall be based on lateral forces that are proportional to the total mass at each floor level. The second pattern, termed the modal pattern in these *Guidelines*, should be selected from one of the following two options:

- a lateral load pattern represented by values of C_{vx} given in Equation 3-8, which may be used if more than 75% of the total mass participates in the fundamental mode in the direction under consideration; or
- a lateral load pattern proportional to the story inertia forces consistent with the story shear distribution calculated by combination of modal responses using (1) Response Spectrum Analysis of the building including a sufficient number of modes to capture 90% of the total mass, and (2) the appropriate ground motion spectrum.

D. Period Determination

The effective fundamental period T_e in the direction under consideration shall be calculated using the force-displacement relationship of the NSP. The nonlinear relation between base shear and displacement of the target node shall be replaced with a bilinear relation to estimate the effective lateral stiffness, K_e, and the yield strength, V_y, of the building. The effective lateral stiffness shall be taken as the secant stiffness calculated at a base shear force equal to 60% of the yield strength. The effective fundamental period T_e shall be calculated as:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \qquad (3\text{-}10)$$

where:

T_i = Elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis

K_i = Elastic lateral stiffness of the building in the direction under consideration

K_e = Effective lateral stiffness of the building in the direction under consideration

See Figure 3-1 for further information.

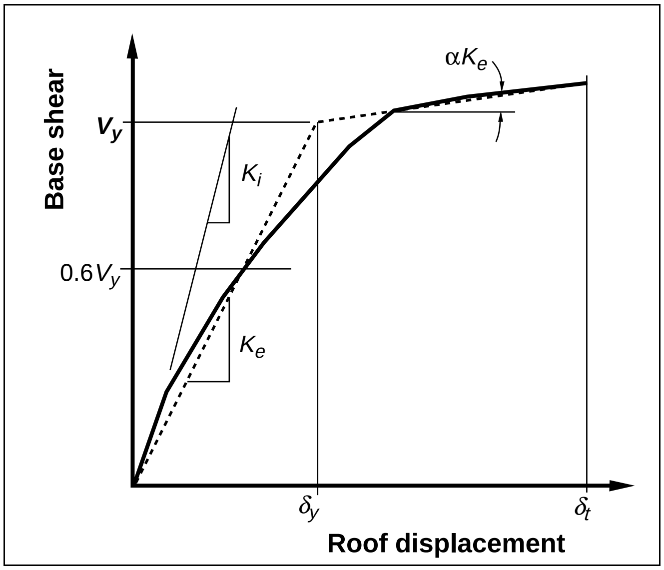

Figure 3-1 ***Calculation of Effective Stiffness, K_e***

E. Analysis of Three-Dimensional Models

Static lateral forces shall be imposed on the three-dimensional mathematical model corresponding to the mass distribution at each floor level. The effects of accidental torsion shall be considered (Section 3.2.2.2).

Independent analysis along each principal axis of the three-dimensional mathematical model is permitted unless multidirectional evaluation is required (Section 3.2.7).

F. Analysis of Two-Dimensional Models

Mathematical models describing the framing along each axis (axis 1 and axis 2) of the building shall be developed for two-dimensional analysis. The effects of horizontal torsion shall be considered (Section 3.2.2.2).

If multidirectional excitation effects are to be considered, component deformation demands and actions shall be computed for the following cases: 100% of the target displacement along axis 1 and 30% of the target displacement along axis 2; and 30% of the target displacement along axis 1 and 100% of the target displacement along axis 2.

3.3.3.3 Determination of Actions and Deformations

A. Target Displacement

The target displacement δ_t for a building with rigid diaphragms (Section 3.2.4) at each floor level shall be estimated using an established procedure that accounts for the likely nonlinear response of the building.

Actions and deformations corresponding to the control node displacement equaling or exceeding the target displacement shall be used for component checking in Section 3.4.

One procedure for evaluating the target displacement is given by the following equation:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \tag{3-11}$$

where:

T_e = Effective fundamental period of the building in the direction under consideration, sec

C_0 = Modification factor to relate spectral displacement and likely building roof displacement

Estimates for C_0 can be calculated using one of the following:

- the first modal participation factor at the level of the control node
- the modal participation factor at the level of the control node calculated using a shape vector corresponding to the deflected shape of the building at the target displacement
- the appropriate value from Table 3-2

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response

= 1.0 for $T_e \geq T_0$

= $[1.0 + (R - 1)T_0 / T_e] / R$ for $T_e < T_0$

Values for C_1 need not exceed those values given in Section 3.3.1.3.
In no case may C_1 be taken as less than 1.0.

T_0 = Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum. (See Sections 2.6.1.5 and 2.6.2.1.)

R = Ratio of elastic strength demand to calculated yield strength coefficient. See below for additional information.

C_2 = Modification factor to represent the effect of hysteresis shape on the maximum displacement response. Values for C_2 are established in Section 3.3.1.3.

C_3 = Modification factor to represent increased displacements due to dynamic P-Δ effects. For buildings with positive post-yield stiffness, C_3 shall be set equal to 1.0. For buildings with negative post-yield stiffness, values of C_3 shall be calculated using Equation 3-13. Values for C_3 need not exceed the values set forth in Section 3.3.1.3.

S_a = Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration, g. The value of S_a is calculated in Sections 2.6.1.5 and 2.6.2.1.

The strength ratio R shall be calculated as:

$$R = \frac{S_a}{V_y / W} \cdot \frac{1}{C_0} \qquad (3\text{-}12)$$

Table 3-2 Values for Modification Factor C_0

Number of Stories	Modification Factor[1]
1	1.0
2	1.2
3	1.3
5	1.4
10+	1.5

1. Linear interpolation should be used to calculate intermediate values.

where S_a and C_0 are as defined above, and:

V_y = Yield strength calculated using results of NSP, where the nonlinear force-displacement (i.e., base shear force versus control node displacement) curve of the building is characterized by a bilinear relation (Figure 3-1)

W = Total dead load and anticipated live load, as calculated in Section 3.3.1.3

Coefficient C_3 shall be calculated as follows if the relation between base shear force and control node displacement exhibits negative post-yield stiffness.

$$C_3 = 1.0 + \frac{|\alpha|(R-1)^{3/2}}{T_e} \qquad (3\text{-}13)$$

where R and T_e are as defined above, and:

α = Ratio of post-yield stiffness to effective elastic stiffness, where the nonlinear force-displacement relation is characterized by a bilinear relation (Figure 3-1)

For a building with flexible diaphragms (Section 3.2.4) at each floor level, a target displacement shall be estimated for each line of vertical seismic framing. The target displacements shall be estimated using an established procedure that accounts for the likely nonlinear response of the seismic framing. One procedure for evaluating the target displacement for an individual line of vertical seismic framing is given by Equation 3-11. The fundamental period of each vertical line of seismic framing, for calculation of the target displacement, shall follow the general procedures described for the NSP; masses shall be assigned to each level of the mathematical model on the basis of tributary area.

For a building with neither rigid nor flexible diaphragms at each floor level, the target displacement shall be calculated using rational procedures. One acceptable procedure for including the effects of diaphragm flexibility is to multiply the displacement calculated using Equation 3-11 by the ratio of the maximum displacement at any point on the roof and the displacement of the center of mass of the roof, both calculated by modal analysis of a three-dimensional model of the building using the design response spectrum. The target displacement so calculated shall be no less than that displacement given by Equation 3-11, assuming rigid diaphragms at each floor level. No vertical line of seismic framing shall be evaluated for displacements smaller than the target displacement. The target displacement should be modified according to Section 3.2.2.2 to account for system torsion.

B. Floor Diaphragms

Floor diaphragms may be designed to resist simultaneously both the seismic forces determined using either Section 3.3.1.3D or Section 3.3.2.3B, and the horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm.

3.3.4 Nonlinear Dynamic Procedure (NDP)

3.3.4.1 Basis of the Procedure

Under the Nonlinear Dynamic Procedure (NDP), design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using an inelastic response history dynamic analysis.

The basis, modeling approaches, and acceptance criteria of the NDP are similar to those for the NSP. The main exception is that the response calculations are carried out using Time-History Analysis. With the NDP, the design displacements are not established using a target displacement, but instead are determined directly through dynamic analysis using ground motion histories. Calculated response can be highly sensitive to characteristics of individual ground motions; therefore, it is recommended to carry out the analysis with more than one ground motion record. Because the numerical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake.

Results of the NDP are to be checked using the applicable acceptance criteria of Section 3.4. Calculated displacements and internal forces are compared directly with allowable values.

3.3.4.2 Modeling and Analysis Assumptions

A. General

The NDP shall conform to the criteria of this section. The analysis shall be based on characterization of the seismic hazard in the form of ground motion records (Section 2.6.2). The modeling and analysis considerations set forth in Section 3.3.3.2 shall apply to the NDP unless the alternative considerations presented below are applied.

The NDP requires Time-History Analysis of a nonlinear mathematical model of the building, involving a time-step-by-time-step evaluation of building response, using discretized recorded or synthetic earthquake records as base motion input.

B. Ground Motion Characterization

The earthquake shaking shall be characterized by ground motion time histories meeting the requirements of Section 2.6.2.

C. Time-History Method

Time-History Analysis shall be performed using horizontal ground motion time histories prepared according to the requirements of Section 2.6.2.2.

Multidirectional excitation effects shall be accounted for by meeting the requirements of Section 3.2.7. The requirements of Section 3.2.7 may be satisfied by analysis of a three-dimensional mathematical model using simultaneously imposed pairs of earthquake ground motion records along each of the horizontal axes of the building.

3.3.4.3 Determination of Actions and Deformations

A. Modification of Demands

The effects of torsion shall be considered according to Section 3.2.2.2.

B. Floor Diaphragms

Floor diaphragms shall be designed to resist simultaneously both the seismic forces calculated by dynamic analysis and the horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm.

3.4 Acceptance Criteria

3.4.1 General Requirements

Components and elements analyzed using the linear procedures of Sections 3.3.1 and 3.3.2 shall satisfy the requirements of this section and Section 3.4.2. Components and elements analyzed using the nonlinear procedures of Sections 3.3.3 and 3.3.4 shall satisfy the requirements of this section and Section 3.4.3.

For the purpose of evaluating acceptability, actions shall be categorized as being either deformation-controlled or force-controlled, as defined in Section 3.2.2.4.

Foundations shall satisfy the criteria set forth in Chapter 4.

3.4.2 Linear Procedures

3.4.2.1 Design Actions

A. Deformation-Controlled Actions

Design actions Q_{UD} shall be calculated according to Equation 3-14.

$$Q_{UD} = Q_G \pm Q_E \tag{3-14}$$

where:

Q_E = Action due to design earthquake loads calculated using forces and analysis models described in either Section 3.3.1 or Section 3.3.2

Q_G = Action due to design gravity loads as defined in Section 3.2.8

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

B. Force-Controlled Actions

The value of a 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 building. It is recommended that this value be based on limit analysis. In lieu of more rational analysis, design actions may be calculated according to Equation 3-15 or Equation 3-16.

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3 J} \tag{3-15}$$

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3} \tag{3-16}$$

where:

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

J = Force-delivery reduction factor given by Equation 3-17

Equation 3-16 can be used in all cases. Equation 3-15 can only be used if the forces contributing to Q_{UF} are delivered by yielding components of the seismic framing system.

The coefficient J shall be established using Equation 3-17.

$$J = 1.0 + S_{XS}\text{, not to exceed 2} \tag{3-17}$$

where:

S_{XS} = Spectral acceleration, calculated in Section 2.6.1.4

Alternatively, J may be taken as equal to the smallest DCR of the components in the load path delivering force to the component in question.

3.4.2.2 Acceptance Criteria for Linear Procedures

A. Deformation-Controlled Actions

Deformation-controlled actions in primary and secondary components and elements shall satisfy Equation 3-18.

$$m\kappa Q_{CE} \geq Q_{UD} \qquad (3\text{-}18)$$

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 4 through 8)

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

κ = Knowledge factor (Section 2.7.2)

For Q_{CE}, the expected strength shall be determined considering all coexisting actions acting on the component under the design loading condition. Procedures to determine the expected strength are given in Chapters 4 through 8.

B. Force-Controlled Actions

Force-controlled actions in primary and secondary components and elements shall satisfy Equation 3-19.

$$\kappa Q_{CL} \geq Q_{UF} \qquad (3\text{-}19)$$

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 coexisting actions acting on the component under the design loading condition. Procedures to determine the lower-bound strength are specified in Chapters 5 through 8.

C. Verification of Design Assumptions

Each component shall be evaluated to determine that assumed locations of inelastic deformations are consistent with strength and equilibrium requirements at all locations along the component length.

Where moments due to gravity loads in horizontally-spanning primary components exceed 75% of the expected moment strength at any location, the possibility for inelastic flexural action at locations other than member ends shall be specifically investigated by comparing flexural actions with expected member strengths. Formation of flexural plastic hinges away from member ends shall not be permitted where design is based on the LSP or the LDP.

3.4.3 Nonlinear Procedures

3.4.3.1 Design Actions and Deformations

Design actions (forces and moments) and deformations shall be the maximum values determined from the NSP or the NDP, whichever is applied.

3.4.3.2 Acceptance Criteria for Nonlinear Procedures

A. Deformation-Controlled Actions

Primary and secondary components shall have expected deformation capacities not less than the maximum deformations. Expected deformation capacities shall be determined considering all coexisting forces and deformations. Procedures for determining expected deformation capacities are specified in Chapters 5 through 8.

B. Force-Controlled Actions

Primary and secondary components shall have lower-bound strengths Q_{CL} not less than the maximum design actions. Lower-bound strength shall be determined considering all coexisting forces and deformations. Procedures for determining lower-bound strengths are specified in Chapters 5 through 8.

3.5 Definitions

This section provides definitions for all key terms used in this chapter and not previously defined.

Action: Sometimes called a generalized force, most commonly a single force or moment. However, an action may also be a combination of forces and moments, a distributed loading, or any combination of forces and moments. Actions always produce or cause displacements or deformations. For example, a bending moment action causes flexural deformation in a beam; an axial force action in a column causes axial deformation in the column; and a torsional moment action on a building causes torsional deformations (displacements) in the building.

Base: The level at which earthquake effects are considered to be imparted to the building.

Components: The basic structural members that constitute the building, such as beams, columns, slabs, braces, piers, coupling beams, and connections. Components, such as columns and beams, are combined to form elements (e.g., a frame).

Control node: The node in the mathematical model of a building used to characterize mass and earthquake displacement.

Deformation: Relative displacement or rotation of the ends of a component or element.

Displacement: The total movement, typically horizontal, of a component or element or node.

Flexible diaphragm: A diaphragm that meets requirements of Section 3.2.4.

Framing type: Type of seismic resisting system.

Element: An assembly of structural components that act together in resisting lateral forces, such as moment-resisting frames, braced frames, shear walls, and diaphragms.

Fundamental period: The first mode period of the building in the direction under consideration.

Inter-story drift: The relative horizontal displacement of two adjacent floors in a building. Inter-story drift can also be expressed as a percentage of the story height separating the two adjacent floors.

Primary component: Those components that are required as part of the building's lateral-force-resisting system (as contrasted to secondary components).

Rigid diaphragm: A diaphragm that meets requirements of Section 3.2.4

Secondary component: Those components that are not required for lateral force resistance (contrasted to primary components). They may or may not actually resist some lateral forces.

Stiff diaphragm: A diaphragm that meets requirements of Section 3.2.4.

Target displacement: An estimate of the likely building roof displacement in the design earthquake.

3.6 Symbols

This section provides symbols for all key variables used in this chapter and not defined previously.

C_0	Modification factor to relate spectral displacement and likely building roof displacement
C_1	Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response
C_2	Modification factor to represent the effect of hysteresis shape on the maximum displacement response
C_3	Modification factor to represent increased displacements due to second-order effects
C_t	Numerical values following Equation 3-4
C_{vx}	Vertical distribution factor for the pseudo lateral load
F_d	Total lateral load applied to a single bay of a diaphragm
F_i and F_x	Lateral load applied at floor levels *i* and *x*, respectively
F_{px}	Diaphragm lateral force at floor level *x*
J	A coefficient used in linear procedures to estimate the actual forces delivered to force-controlled components by other (yielding) components.
K_e	Effective stiffness of the building in the direction under consideration, for use with the NSP
K_i	Elastic stiffness of the building in the direction under consideration, for use with the NSP
L_d	Single-bay diaphragm span
Q_{CE}	Expected strength of a component or element at the deformation level under consideration in a deformation-controlled action

Q_{CL}	Lower-bound estimate of the strength of a component or element at the deformation level under consideration for force-controlled actions
Q_D	Dead load force (action)
Q_E	Earthquake force (action) calculated using procedures of Section 3.3.1 or 3.3.2
Q_G	Gravity load force (action)
Q_L	Effective live load force (action)
Q_S	Effective snow load force (action)
Q_{UD}	Deformation-controlled design action
Q_{UF}	Force-controlled design action
R	Ratio of the elastic strength demand to the yield strength coefficient
S_a	Response spectrum acceleration at the fundamental period and damping ratio of the building, *g*
S_{XS}	Spectral response acceleration at short periods for any hazard level and damping, *g*
T	Fundamental period of the building in the direction under consideration
T_e	Effective fundamental period of the building in the direction under consideration, for use with the NSP
T_i	Elastic fundamental period of the building in the direction under consideration, for use with the NSP
T_0	Period at which the constant acceleration and constant velocity regions of the design spectrum intersect
V	Pseudo lateral load
V_y	Yield strength of the building in the direction under consideration, for use with the NSP
W	Total dead load and anticipated live load
W_i and W_x	Weight of floors *i* and *x*, respectively
f_d	Lateral load per foot of diaphragm span
g	Acceleration of gravity (386.1 in./sec^2, or 9,807 mm/sec^2 for SI units)
h_i and h_x	Height from the base of a building to floor levels *i* and *x*, respectively
h_n	Height to roof level, ft
k	Exponent used for determining the vertical distribution of lateral forces
m	A modification factor used in the acceptance criteria of deformation-controlled components or elements, indicating the available ductility of a component action
w_i and w_x	Portion of the total building weight corresponding to floor levels *i* and *x*, respectively
x	Distance from the diaphragm center line
Δ_d	Diaphragm deformation
Δ_w	Average in-plane wall displacement
α	Ratio of post-yield stiffness to effective stiffness
δ_t	Target roof displacement
δ_y	Yield displacement of building (Figure 3-1)
η	Displacement multiplier, greater than 1.0, to account for the effects of torsion
θ	Stability coefficient (Equation 2-14)—a parameter indicative of the stability of a structure under gravity loads and earthquake-induced deflection
κ	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.)

3.7 References

BSSC, 1995, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings, 1994 Edition, Part 1: Provisions and Part 2: Commentary*, prepared by the Building Seismic Safety Council, for the Federal Emergency Management Agency (Report Nos. FEMA 222A and 223A), Washington, D.C.

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