3. Analysis Procedures

3.1 Scope

This chapter sets forth requirements for analysis of buildings using the Systematic Rehabilitation Method. Section 3.2 specifies general analysis requirements for the mathematical modeling of buildings including basic assumptions, consideration of torsion, diaphragm flexibility, P-Δ effects, soil-structure interaction, multidirectional effects, and overturning. Section 3.3 defines four analysis procedures included in this standard. Section 3.4 defines component acceptance criteria.

Analysis of buildings with seismic isolation or energy dissipation systems shall comply with the requirements of Chapter 9. Analysis of buildings using the Simplified Rehabilitation Method shall comply with the requirements of Chapter 10.

C3.1 Scope

The relationship of the analysis procedures described in this chapter with specifications in other chapters of this standard is as follows.

- Information on Rehabilitation Objectives, including Earthquake Hazard Levels and target Building Performance Levels, is provided in Chapter 1.
- The provisions set forth in this chapter are intended for Systematic Rehabilitation. 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 design requirements are specified in Section 2.6 for multidirectional excitation effects, P-Δ effects, horizontal torsion, overturning, continuity of the framing system, diaphragms, walls, nonstructural components, building separation, structures sharing common components, and vertical seismic effects.
- 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.

3.2 General Analysis Requirements

An analysis of the building, as specified in Section 2.4, shall be conducted in accordance with the requirements of this section and Section 2.6.

3.2.1 Analysis Procedure Selection

An analysis of the building shall be performed using the Linear Static Procedure (LSP), Linear Dynamic Procedure (LDP), Nonlinear Static Procedure (NSP), or Nonlinear Dynamic Procedure (NDP) selected based on the limitations specified in Section 2.4. Use of alternative rational analysis procedures as described in Section 2.4.3 shall also be permitted.

C3.2.1 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 2.4.

Linear procedures are appropriate when the expected level of nonlinearity is low. This is measured by component demand to capacity ratios (DCRs) of less than 2.0.

Static procedures are appropriate when higher mode effects are not significant. This is generally true for short, regular buildings. Dynamic procedures are required for tall buildings, buildings with torsional irregularities, or non-orthogonal systems.

The Nonlinear Static Procedure is acceptable for most buildings, but should be used in conjunction with the Linear Dynamic Procedure if mass participation in the first mode is low.

The term “linear” in linear analysis procedures implies “linearly elastic.” The analysis procedure, however, may include geometric nonlinearity of gravity loads acting through lateral displacements and implicit material nonlinearity of concrete and masonry components using properties of cracked sections. The term “nonlinear” in nonlinear analysis procedures implies explicit material nonlinearity or inelastic material response, but geometric nonlinearity may also be included.

3.2.2 Mathematical Modeling

3.2.2.1 Basic Assumptions

A building shall be modeled, analyzed, and evaluated as a three-dimensional assembly of elements and components. Alternatively, use of a two-dimensional model shall permitted if the building meets one of the following conditions:

1. The building has rigid diaphragms as defined in Section 3.2.4 and horizontal torsion effects do not exceed the limits specified in Section 3.2.2.2, or horizontal torsion effects are accounted for as specified in Section 3.2.2.2.

2. The building has flexible diaphragms as defined in Section 3.2.4.

If two-dimensional models are used, the three-dimensional nature of components and elements shall be considered when calculating stiffness and strength properties.

If the building contains out-of-plane offsets in vertical lateral-force-resisting elements, the model shall explicitly account for such offsets in the determination of diaphragm demands.

Modeling stiffness of structural components shall be based on the stiffness requirements of Chapters 4 through 8.

For nonlinear procedures, a connection shall be explicitly modeled if the connection is weaker, has less ductility than the connected components, or the flexibility of the connection results in a change in the connection forces or deformations greater than 10%.

C3.2.2.1 Basic Assumptions

For two-dimensional models, the three-dimensional nature of components and elements should be recognized in calculating their stiffness and strength properties. For example, shear walls and other bracing systems may have “L” or “T” or other three-dimensional cross-sections where contributions of both the flanges and webs should be accounted for in calculating stiffness and strength properties.

In this standard, component stiffness is generally taken as the effective stiffness based on the secant stiffness to yield level forces. Specific direction on calculating effective stiffness is provided in each material chapter for each type of structural system.

Examples of where connection flexibility may be important to model include the panel zone of steel moment-resisting frames and the “joint” region of perforated masonry or concrete walls.

3.2.2.2 Horizontal Torsion

The effects of horizontal torsion shall be considered in accordance with this section. Torsion need not be considered in buildings with flexible diaphragms as defined in Section 3.2.4.

3.2.2.2.1 Total Torsional Moment

The total horizontal torsional moment at a story shall be equal to the sum of the actual torsional moment and accidental torsional moment calculated as follows:

1. The actual torsional moment at a story shall be calculated by multiplying the seismic story shear force by the eccentricity between the center of mass and the center of rigidity measured perpendicular to the direction of the applied load. The center of mass shall be based on all floors above the story under consideration. The center of rigidity of a story shall include all vertical seismic elements in the story.

2. The accidental torsion moment at a story shall be calculated as the seismic story shear force multiplied by a distance equal to 5% of the horizontal dimension at the given floor level measured perpendicular to the direction of the applied load.

3.2.2.2.2 Consideration of Torsional Effects

Effects of horizontal torsion shall be considered in accordance with the following requirements:

1. Increased forces and displacements due to actual torsion shall be calculated for all buildings.

2. The displacement multiplier, η, at each floor shall be calculated as the ratio of the maximum displacement at any point on the floor diaphragm to the average displacement ($\delta_{max}/\delta_{avg}$). Displacements shall be calculated for the applied loads.

3. Increased forces and displacements due to accidental torsion shall be considered unless the accidental torsional moment is less than 25 percent of the actual torsional moment, or the displacement multiplier η due to the applied load and accidental torsion is less than 1.1 at every floor.

4. For linear analysis procedures, forces and displacements due to accidental torsion shall be amplified by a factor, A_x, as defined by Equation (3-1), when the displacement multiplier η due to total torsional moment exceeds 1.2 at any level.

$$A_x = \left(\frac{\eta_x}{1.2}\right)^2 \leq 3.0 \qquad (3\text{-}1)$$

5. If the displacement modifier η due to total torsional moment at any floor exceeds 1.50, two-dimensional models shall not be permitted and three-dimensional models that account for the spatial distribution of mass and stiffness shall be used.

6. When two-dimensional models are used, the effects of horizontal torsion shall be calculated as follows.

   6.1. For the LSP and the LDP, forces and displacements shall be amplified by the maximum value of η calculated for the building.

   6.2. For the NSP, the target displacement shall be amplified by the maximum value of η calculated for the building.

   6.3. For the NDP, the amplitude of the ground acceleration record shall be amplified by the maximum value of η calculated for the building.

7. The effects of accidental torsion shall not be used to reduce force and deformation demands on components and elements.

C3.2.2.2 Horizontal Torsion

Actual torsion is due to the eccentricity between centers of mass and stiffness. Accidental torsion is intended to cover the effects of the rotational component of the ground motion, differences between computed and actual stiffness, and unfavorable distributions of dead and live load masses.

The 10% threshold on additional displacement due to accidental torsion is based on judgment. The intent is to reward those building frames that are torsionally redundant and possess high torsional stiffness. Such structures are likely to be much less susceptible to torsional response than those framing systems possessing low redundancy and low torsional stiffness.

3.2.2.3 Primary and Secondary Elements and Components

Elements and components shall be classified as primary or secondary as defined in Section 2.4.4.2. Primary elements and components shall be evaluated for earthquake-induced forces and deformations in combination with gravity load effects. Secondary elements and components shall be evaluated for earthquake-induced deformations in combination with gravity load effects.

Mathematical models for use with linear analysis procedures shall include the stiffness and resistance of only the primary elements and components. If the total lateral stiffness of secondary elements exceeds 25% of the total initial stiffness of primary elements, some secondary elements shall be reclassified as primary to reduce the total stiffness of secondary elements to less than 25% of primary. If the exclusion of a secondary element will reduce the force or deformation demands on a primary element, the secondary element shall be included in the model.

Mathematical models for use with nonlinear procedures shall include the stiffness and resistance of primary and secondary elements and components. The strength and stiffness degradation of primary and secondary elements and components shall be explicitly modeled. For the simplified NSP of Section 3.3.3.2.1, only primary elements and components shall be included in the model and degradation shall not be modeled.

Nonstructural components shall be included in mathematical models if their lateral stiffness exceeds 10% of the total initial lateral stiffness of a story.

Components and elements shall not be selectively designated primary or secondary to change the configuration of a building from irregular to regular.

C3.2.2.3 Primary and Secondary Elements and Components

In linear analysis procedures, the 25% limit for the lateral stiffness of the secondary components and elements can be checked by initially including the secondary components and elements in the mathematical model and examining their stiffness contribution.

3.2.2.4 Stiffness and Strength Assumptions

Stiffness and strength properties of components and elements shall be determined in accordance with the requirements of Chapters 4 through 9, and 11.

3.2.2.5 Foundation Modeling

The foundation system shall be modeled considering the degree of fixity provided at the base of the structure. Rigid or flexible base assumptions shall be permitted in accordance with the requirements of Section 4.4.3. The foundation modeling shall consider movement due to geologic site hazards specified in Section 4.2.2.

3.2.3 Configuration

Building irregularities defined in Section 2.4.1.1 shall be based on the plan and vertical configuration of the rehabilitated structure. A structure defined as regular shall not have any irregularity defined in Section 2.4.1.1, both with and without the contribution of secondary components.

C3.2.3 Configuration

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

Adding seismic framing elements at certain locations will improve the regularity of the building and should be considered as a means to improve seismic performance of the building.

Secondary components can lose significant strength and stiffness after initial earthquake shaking and may no longer be effective. Therefore, regularity of the building should be determined independent of the contribution of secondary components.

3.2.4 Diaphragms

3.2.4.1 General

Diaphragms shall be classified as either flexible, stiff, or rigid in accordance with Section 3.2.4.2.

3.2.4.2 Classification of Diaphragms

Diaphragms shall be classified as flexible when the maximum horizontal deformation of the diaphragm along its length is more than twice the average interstory drift of the vertical lateral-force-resisting elements of the story immediately below the diaphragm. For diaphragms supported by basement walls, the average interstory drift of the story above the diaphragm shall be used.

Diaphragms shall be classified as rigid when the maximum lateral deformation of the diaphragm is less than half the average interstory drift of the vertical lateral-force-resisting elements of the associated story.

Diaphragms that are neither flexible nor rigid shall be classified as stiff.

For the purpose of classifying diaphragms, interstory drift and diaphragm deformations shall be calculated using the pseudo lateral load specified in Equation (3-10). The in-plane deflection of the diaphragm shall be calculated for an in-plane distribution of lateral force consistent with the distribution of mass, and all in-plane lateral forces associated with offsets in the vertical seismic framing at that diaphragm level.

3.2.4.3 Mathematical Modeling

Mathematical modeling of buildings with rigid diaphragms shall account for the effects of horizontal torsion as specified in Section 3.2.2.2. Mathematical models of buildings with stiff or flexible diaphragms shall account for the effects of diaphragm flexibility by modeling the diaphragm as an element with an in-plane stiffness consistent with the structural characteristics of the diaphragm system. Alternatively, for buildings with flexible diaphragms at each floor level, each lateral-force-resisting element in a vertical plane shall be permitted to be designed independently, with seismic masses assigned on the basis of tributary area.

C3.2.4 Diaphragms

Evaluation of diaphragm demands should be based on the likely distribution of horizontal inertia forces. For flexible diaphragms, such a distribution may be given by Equation (C3-1) and illustrated in Figure C3-1 below.

$$f_d = \frac{1.5F_d}{L_d}\left[1-\left(\frac{2x}{L_d}\right)^2\right] \qquad \text{(C3-1)}$$

where:

f_d = Inertial load per foot

F_d = Total inertial load on a flexible diaphragm

x = Distance from the center line of flexible diaphragm

L_d = Distance between lateral support points for diaphragm

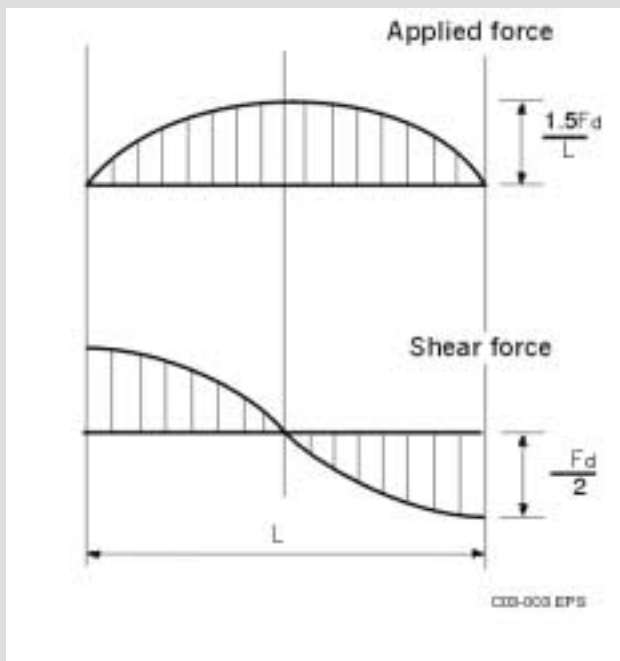

Figure C3-1 Plausible Force Distribution in a Flexible Diaphragm

3.2.5 P-Δ Effects

Buildings shall be evaluated for static P-Δ effects and dynamic P-Δ effects as specified in this section.

3.2.5.1 Static P-Δ Effects

Static P-Δ effects shall be included in linear or nonlinear analysis procedures as specified in Sections 3.2.5.1.1 and 3.2.5.1.2, respectively.

3.2.5.1.1 Linear Procedures

For linear procedures, the stability coefficient θ shall be evaluated for each story in the building and for each direction of response using Equation (3-2).

$$\theta_i = \frac{P_i \delta_i}{V_i h_i} \qquad (3\text{-}2)$$

where:

P_i = Portion of the total weight of the structure including dead, permanent live, and 25% of transient live loads acting on the columns and bearing walls within story level i

V_i = The total calculated lateral shear force in the direction under consideration at story i due to earthquake response to the selected ground shaking level, as indicated by the selected linear analysis procedure

h_i = Height of story i, which shall be taken as the distance between either the centerline of floor framing at each of the levels above and below, or the top of floor slabs at each of the levels above and below (or other common points of reference)

δ_i = Lateral drift in story i, in the direction under consideration, at its center of rigidity, using the same units as for measuring h_i

When the stability coefficient θ_i is less than 0.1 in all stories, the P-Δ effects need not be considered. If the stability coefficient lies between 0.1 and 0.33, seismic forces and deformations in story i shall be increased by the factor $1/(1-\theta_i)$. When the stability coefficient θ_i exceeds 0.33, the structure shall be considered unstable and the rehabilitation design modified to reduce the computed lateral deflections in the story to comply with this limitation.

3.2.5.1.2 Nonlinear Procedures

For nonlinear procedures, static P-Δ effects shall be incorporated in the analysis by including in the mathematical model the nonlinear force-deformation relationship of all elements and components subjected to axial forces.

3.2.5.2 Dynamic P-Δ Effects

Dynamic P-Δ effects shall be included using the coefficient C_3 defined for linear procedures in Section 3.3.1.3.1 or for the NSP in Section 3.3.3.3.2.

C3.2.5 P-Δ Effects

Static P-Δ effects are caused by gravity loads acting through the deformed configuration of a building and result in an increase in lateral displacements.

A negative post-yield stiffness may significantly increase interstory drift and the target displacement. Dynamic P-Δ effects are introduced to consider this additional drift. The degree by which dynamic P-Δ effects increase displacements depends on the following:

1. The ratio α of the negative post-yield stiffness to the effective elastic stiffness;
2. The fundamental period of the building;
3. The strength ratio, R;
4. The hysteretic load-deformation relations for each story;
5. The frequency characteristics of the ground motion; and
6. The duration of the strong ground motion.

Because of the number of parameters involved, it is difficult to capture dynamic P-Δ effects with a single modification factor. Coefficient C_3 represents a substantial simplification and interpretation of much analysis data. Dynamic P-Δ effects are automatically captured in the NDP.

3.2.6 Soil-Structure Interaction

The effects of soil-structure interaction (SSI) shall be evaluated for those buildings in which an increase in fundamental period due to SSI effects will result in an increase in spectral accelerations. For other buildings, the effects of SSI need not be evaluated.

SSI effects shall be calculated in accordance with this section, or other approved rational procedure. The simplified procedure shall be permitted only when the LSP is used. The explicit modeling procedure shall be used when the LDP, NSP, or NDP are used. It shall be permitted to ignore the effects of damping in the calculation of SSI when SSI effects are not required to be evaluated.

C3.2.6 Soil-Structure Interaction

Soil-structure interaction (SSI) may modify the seismic demand on a building.

For those rare cases (such as near-field and soft soil sites) in which the increase in period due to SSI increases spectral accelerations, the effects of SSI on building response must be evaluated.

3.2.6.1 Simplified Procedure

Calculation of SSI effects using the simplified procedure shall comply with the procedure in *ASCE 7* utilizing the effective fundamental period and effective fundamental damping ratio of the foundation-structure system.

When the simplified procedure is used to evaluate SSI effects, reduction in seismic demands on elements and components shall not exceed 25% of the demands calculated without SSI effects.

3.2.6.2 Explicit Modeling Procedure

Calculation of SSI effects using the explicit modeling procedure shall explicitly model the stiffness and damping of individual foundation elements. Foundation stiffness parameters shall comply with the requirements of Section 4.4.2. In lieu of explicitly modeling damping, the effective damping ratio, β, of the structure-foundation system shall be permitted to be calculated using the simplified procedure. The damping ratio used for individual foundation elements shall not exceed the value used for the elastic superstructure. For the NSP, the effective damping ratio of the foundation-structure system shall be used to calculate the spectral demands. If the simplified procedure is used to calculate the effective damping ratio, reduction in seismic demands shall not exceed 25% of the demands calculated without SSI effects.

3.2.7 Multidirectional Seismic Effects

Buildings shall be designed for seismic motion in any horizontal direction. Multidirectional seismic effects shall be considered to act concurrently as specified in Section 3.2.7.1for buildings meeting the following criteria:

1. The building has plan irregularities as defined in Section 2.4.1.1; or

2. The building has one or more primary columns which form a part of two or more intersecting frame or braced frame elements.

All other buildings shall be permitted to be designed for seismic motions acting nonconcurrently in the direction of each principal axis of the building.

3.2.7.1 Concurrent Seismic Effects

When concurrent multidirectional seismic effects must be considered, horizontally oriented orthogonal X and Y axes shall be established. Elements and components of the building shall be designed for combinations of forces and deformations from separate analyses performed for ground motions in X and Y directions as follows:

1. Where the LSP or LDP are used as the basis for design, elements and components shall be designed for (a) forces and deformations associated with 100% of the design forces in the X direction plus the forces and deformations associated with 30% of the design forces in the perpendicular horizontal Y direction, and for (b) forces and deformations associated with 100% of the design forces in the Y direction plus the forces and deformations associated with 30% of the design forces in the X direction. Other combination rules shall be permitted where verified by experiment or analysis.

2. Where the NSP or NDP are used as the basis for design, elements and components of the building shall be designed for (a) forces and deformations associated with 100% of the design displacement in the X direction plus the forces (not deformations) associated with 30% of the design displacements in the perpendicular horizontal Y direction, and for (b) forces and deformations associated with 100% of the design displacements in the Y direction plus the forces (not deformations) associated with 30% of the design displacements in the X direction. Other combination rules shall be permitted where verified by experiment or analysis.

3.2.7.2 Vertical Seismic Effects

For components in which Section 2.6.11 requires consideration of vertical seismic effects, the vertical response of a structure to earthquake ground motion need not be combined with the effects of the horizontal response.

3.2.8 Component Gravity Loads for 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, the gravity loads shall be obtained in accordance with Equation (3-3).

$$Q_G = 1.1(Q_D + Q_L + Q_S) \qquad (3\text{-}3)$$

When the effects of gravity and seismic loads are counteracting, the gravity loads shall be obtained in accordance with Equation (3-4).

$$Q_G = 0.9Q_D \qquad (3\text{-}4)$$

where:

Q_D = Dead-load (action).

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

Q_S = Effective snow load (action) contribution to *W*, specified in Section 3.3.1.3.1.

C3.2.8 Component Gravity Loads for Load Combinations

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 along the component length to verify that locations of potential inelastic action have been properly accounted for in the analysis. Each component shall also be evaluated for post-earthquake residual gravity load capacity by an approved rational analysis procedure that accounts for potential redistribution of gravity loads and reduction of strength or stiffness caused by earthquake damage to the structure.

C3.2.9 Verification of Design Assumptions

It is important that assumptions about locations of potential inelastic activity in the structure are verified. In linear procedures, the potential for inelastic flexural action is restricted to the beam ends because flexural yielding along the span length can lead to unconservative results. In nonlinear procedures, potential inelastic activity should only occur where specifically modeled. Where demands due to gravity load combinations of Section 3.2.8 exceed 50 percent of the capacity of the component at any location along its length, the potential for inelastic activity exists and should be investigated. Sample procedures for verifying design assumptions are contained in Section C3.2.9 of *FEMA 274*.

3.2.10 Overturning

Structures shall be designed to resist overturning effects caused by seismic forces. Each vertical-force-resisting element receiving earthquake forces due to overturning shall be investigated for the cumulative effects of seismic forces applied at and above the level under consideration. The effects of overturning shall be evaluated at each level of the structure as specified in Section 3.2.10.1 for linear procedures, or Section 3.2.10.2 for nonlinear procedures. The effects of overturning on foundations and geotechnical components shall be considered in the evaluation of foundation strength and stiffness as specified in Chapter 4.

C3.2.10 Overturning

Response to earthquake ground motion results in a tendency for structures and individual vertical elements of structures to overturn about their bases. Although actual overturning failure is very rare, overturning effects can result in significant stresses, as demonstrated in some local and global failures. In new building design, earthquake effects, including overturning, are evaluated for lateral forces that are significantly reduced (by an *R*-factor) from those that may actually develop in the structure.

For elements with positive attachment between levels that behave as single units—such as reinforced concrete walls—the overturning effects are resolved into component forces (e.g., flexure and shear at the base of the wall). The element is then proportioned with adequate strength using *m*-factors, where appropriate, to resist overturning effects resulting from these force levels.

Some elements, such as wood shear walls and foundations, may not be designed with positive attachment between levels. An overturning stability check is typically performed for such elements when designed using codes for new buildings. If the element has sufficient dead load to remain stable under the overturning effects of the design lateral forces and sufficient shear connection to the level below, then the design is deemed adequate. However, if dead load is inadequate to provide stability, then hold-downs, piles, or other types of uplift anchors are provided to resist the residual overturning caused by the design forces.

In the linear and nonlinear procedures of this standard, lateral forces are not reduced by an *R*-factor, as they are for new buildings, so computed overturning effects are larger than typically calculated for new buildings. Although the procedure used for new buildings is not completely rational, it has resulted in successful performance. Therefore, it may not be appropriate to require that structures and elements of structures remain stable for the pseudo lateral forces used in the linear procedures in this standard. Instead, the designer must determine if positive direct attachment will be used to resist overturning effects or if dead loads will be used. If positive direct attachment will be used, then this attachment is treated just as any other element or component action.

However, if dead loads alone are used to resist overturning, then overturning is treated as a force-controlled behavior and the overturning demands are reduced to an estimate of the real overturning demands that can be transmitted to the element, considering the overall limiting strength of the structure.

There is no simple rational method available, shown to be consistent with observed behavior, to design or evaluate elements for overturning effects. The method described in this standard is rational, but inconsistent with procedures used for new buildings. To improve damage control, the full lateral forces used in the linear procedures of this standard are required for checking acceptability for performance levels higher than life safety.

Additional studies are needed on the parameters that control overturning in seismic rehabilitation.

3.2.10.1 Linear Procedures

When linear procedures are used, overturning effects shall be resisted through the stabilizing effect of dead loads acting alone or in combination with positive connection of structural components to elements or components below the level under consideration.

Where dead loads alone are used to resist the effects of overturning, Equation (3-5) shall be satisfied:

$$M_{ST} > M_{OT} / (C_1 C_2 C_3 J) \qquad (3\text{-}5)$$

where:

M_{OT} = Total overturning moment induced on the element by seismic forces applied at and above the level under consideration

M_{ST} = Stabilizing moment produced by dead loads acting on the element

C_1, C_2, and C_3 = Coefficients defined in Section 3.3.1.3

J =Coefficient defined in Section 3.4.2.1.2The quantity $M_{OT}/(C_1C_2C_3J)$ need not exceed the overturning moment on the element, as limited by the expected strength of the structure. The element shall be evaluated for the effects of increased compression at the end about which it is being overturned. For this purpose, compression at the end of the element shall be considered a force-controlled action.

Alternatively, the load combination represented by Equation (3-6) shall be permitted for evaluating the adequacy of dead loads alone to resist the effects of overturning.

$$0.9M_{ST} > M_{OT}/(C_1C_2C_3R_{OT}) \qquad (3\text{-}6)$$

where:

R_{OT} = 10.0 for Collapse Prevention
= 8.0 for Life Safety
= 4.0 for Immediate Occupancy.

When Equations (3-5) or (3-6) for dead load stability against the effects of overturning are not satisfied, positive attachment between elements of the structure above and below the level under consideration shall be provided. If the level under consideration is the base of the structure, positive attachment shall be provided between the structure and the supporting soil, unless nonlinear procedures are used to rationalize overturning stability. Positive attachments shall be capable of resisting earthquake forces in combination with gravity loads as force- or deformation-controlled actions in accordance with Equations (3-18) or (3-19) and applicable acceptance criteria of Equations (3-20) or (3-21).

C3.2.10.1 Linear Procedures

For evaluating the adequacy of dead loads to provide stability against overturning, the alternative procedure of Section 3.2.10.1 is intended to provide a method that is consistent with prevailing practice specified in current codes for new buildings.

3.2.10.2 Nonlinear Procedures

When nonlinear procedures are used, the effects of earthquake-induced uplift on the tension side of an element, or rocking, shall be included in the analytical model as a nonlinear degree of freedom. The adequacy of elements above and below the level at which uplift or rocking occurs, including the foundations, shall be evaluated for any redistribution of forces or deformations that occurs as a result of this rocking.

3.3 Analysis Procedures

Selection of an appropriate analysis procedure shall comply with Section 3.2.1.

3.3.1 Linear Static Procedure

3.3.1.1 Basis of the Procedure

If the Linear Static Procedure (LSP) is selected for seismic analysis of the building, the design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements shall be determined using a linearly elastic, static analysis in accordance with this section.

Buildings shall be modeled with linearly elastic stiffness and equivalent viscous damping values consistent with components responding at or near yield level, as defined in Section 2.4.4. The pseudo lateral load defined in Section 3.3.1.3 shall be used to calculate internal forces and system displacements due to the design earthquake.

Results of the LSP shall be checked using the acceptance criteria of Section 3.4.2.

C3.3.1.1 Basis of the Procedure

The magnitude of the pseudo lateral load has been selected with the intention that, when applied to the linearly elastic model of the building, it will result in design displacement amplitudes approximating maximum displacements expected during the design earthquake. The procedure is keyed to the displacement response of the building because displacements are a better indicator of damage in the nonlinear range of building response than are forces. In this range relatively small changes in force demand correspond to large changes in displacement demand. 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 commonly will be the case, the actual internal forces that would develop in the yielding building will be less than the internal forces calculated using a pseudo lateral load.

Calculated internal forces typically will exceed those that the building can develop because of anticipated inelastic response of components and elements. These 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 Period Determination

The fundamental period of a building shall be calculated for the direction under consideration using one of the following analytical, empirical, or approximate methods specified in this section.

3.3.1.2.1 Method 1—Analytical

Eigenvalue (dynamic) analysis of the mathematical model of the building shall be performed to determine the fundamental period of the building.

3.3.1.2.2 Method 2—Empirical

The fundamental period of the building shall be determined in accordance with Equation (3-7):

$$T = C_t h_n^{\beta} \tag{3-7}$$

where:

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

C_t = 0.035 for steel moment-resisting frame systems

= 0.018 for concrete moment-resisting frame systems

= 0.030 for steel eccentrically-braced frame systems

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

= 0.020 for all other framing systems

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

β = 0.80 for steel moment-resisting frame systems

= 0.90 for concrete moment-resisting frame systems

= 0.75 for all other framing systems

3.3.1.2.3 Method 3—Approximate

1. For any building, use of the Rayleigh-Ritz method to approximate the fundamental period shall be permitted.

2. For one-story buildings with single span flexible diaphragms, use of Equation (3-8) to approximate the fundamental period shall be permitted.

$$T = (0.1\Delta_w + 0.078\Delta_d)^{0.5} \tag{3-8}$$

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 of the diaphragm.

3. For one-story buildings with multiple-span diaphragms, use of Equation (3-8) shall be permitted as follows: a lateral load equal to the weight tributary to the diaphragm span under consideration shall be applied to calculate a separate period for each diaphragm span. The period that maximizes the pseudo lateral load shall be used for design of all walls and diaphragm spans in the building.

4. For unreinforced masonry buildings with single span flexible diaphragms, six stories or less in height, use of Equation (3-9) to approximate the fundamental period shall be permitted.

$$T = (0.078\Delta_d)^{0.5} \qquad (3\text{-}9)$$

where Δ_d is the maximum in-plane diaphragm displacement in inches, due to a lateral load in the direction under consideration, equal to the weight tributary to the diaphragm.

C3.3.1.2 Period Determination

C3.3.1.2.1 Method 1—Analytical

For many buildings, including multistory buildings with well-defined framing systems, the preferred approach to obtaining the period for design is Method 1. By this method, the building is modeled using the modeling procedures of Chapters 4 through 8 and 11, and the period is obtained by Eigenvalue analysis. Flexible diaphragms may be modeled as a series of lumped masses and diaphragm finite elements.

Contrary to procedures in codes for new buildings, there is no maximum limit on period calculated using Method 1. This omission is intended to encourage the use of more advanced analyses. It is felt that sufficient controls on analyses and acceptance criteria are present within this standard to provide appropriately conservative results using calculated periods.

C3.3.1.2.2 Method 2—Empirical

Empirical equations for period, such as that used in Method 2, intentionally underestimate the actual period and will generally result in conservative estimates of pseudo lateral load. Studies have shown that depending on actual mass or stiffness distributions in a building, the results of Method 2 may differ significantly from those of Method 1. The C_t values specified for Method 2 are generally consistent with *FEMA 302* but have been modified based on recent published research on measured building response to earthquakes. The C_t value for wood buildings is not substantiated by field measurements and is based on engineering judgment.

C3.3.1.2.3 Method 3—Approximate

Method 3 is appropriate for systems with rigid vertical elements and flexible diaphragms in which the dynamic response of the system is concentrated in the diaphragm. Use of Method 2 on these systems to calculate the period based on the stiffness of the vertical elements will substantially underestimate the period of actual dynamic response and overestimate the pseudo lateral load.

Equation (3-9) is a special case developed specifically for URM buildings. In this method, wall deformations are assumed negligible compared to diaphragm deflections.

For illustration of wall and diaphragm displacements see Figure C3-2. When calculating diaphragm displacements for the purpose of estimating period using Equations (3-8) or (3-9), the diaphragm shall be considered to remain elastic under the prescribed lateral loads.

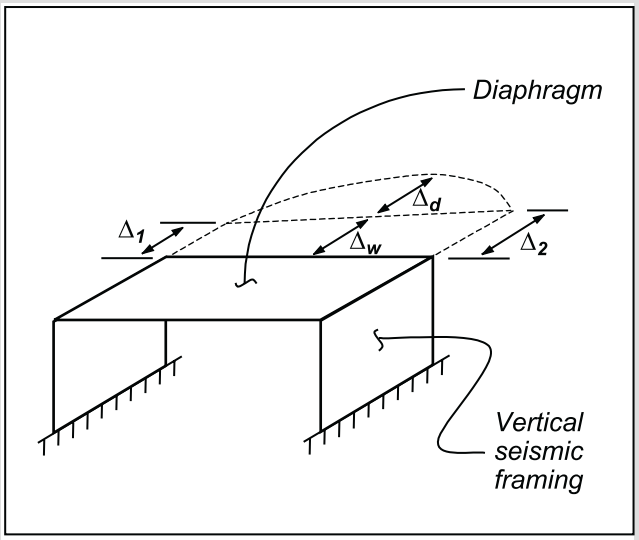

Figure C3-2 ***Diaphragm and Wall Displacement Terminology***

3.3.1.3 Determination of Forces and Deformations

Forces and deformations in elements and components shall be calculated for the psuedo lateral load of Section 3.3.1.3.1, using component stiffnesses calculated in accordance with Chapters 4 through 8. Pseudo lateral loads shall be distributed throughout the building in accordance with Sections 3.3.1.3.2 through 3.3.1.3.4. Alternatively, for unreinforced masonry buildings in which the fundamental period is calculated using Equation (3-9), pseudo lateral loads shall be permitted to be distributed in accordance with Section 3.3.1.3.5. Actions and deformations shall be modified to consider the effects of horizontal torsion in accordance with Section 3.2.2.2.

3.3.1.3.1 Pseudo Lateral Load

The pseudo lateral load in a given horizontal direction of a building shall be determined using Equation (3-10). This load shall be used to design the vertical elements of the lateral-force-resisting system.

$$V = C_1 C_2 C_3 C_m S_a W \qquad (3\text{-}10)$$

where:

V = Pseudo lateral load.

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response, calculated either using the procedure indicated in Section 3.3.3.3 with the elastic base shear capacity substituted for shear yield strength V_y in Equation (3-16) or calculated as follows:

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

$C_1 = 1.0$ for $T \geq T_S$ 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, calculated in accordance with Section 3.3.1.2, including modification for SSI effects of Section 3.2.6, if applicable.

T_S = 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 in accordance with Sections 1.6.1.5 and 1.6.2.1.

C_2 = Modification factor to represent the effects of pinched hysteresis shape, stiffness degradation, and strength deterioration on maximum displacement response. For linear procedures C_2 shall be taken as 1.0.

C_3 = Modification factor to represent increased displacements due to dynamic P-Δ effects specified in Section 3.2.5.2. For values of the stability coefficient θ_i per Equation (3-2) less than 0.1 in all stories, C_3 shall be set equal to 1.0, otherwise C_3 shall be calculated as $1 + 5(\theta - 0.1)/T$ using θ equal to the maximum value of θ_i of all stories.

C_m = Effective mass factor to account for higher mode mass participation effects obtained from Table 3-1. C_m shall be taken as 1.0 if the fundamental period, T, is greater than 1.0 second.

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 specified in Section 1.6.

W = Effective seismic weight of the building including the total dead load and applicable portions of other gravity loads listed below:

1. In areas used for storage, a minimum 25% of the floor live load shall be applicable. The live load shall be permitted to be reduced for tributary area as approved by the code official. Floor live load in public garages and open parking structures is not applicable.
2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 psf of floor area, whichever is greater, shall be applicable.
3. Total operating weight of permanent equipment.
4. Where the design flat roof snow load calculated in accordance with *ASCE 7* exceeds 30 psf, the effective snow load shall be taken as 20% of the design snow load. Where the design flat roof snow load is less than 30 psf, the effective snow load shall be permitted to be zero.

C3.3.1.3.1 Pseudo Lateral Load

Coefficient C_1. This modification factor is to account for the difference in maximum elastic and inelastic displacement amplitudes in structures with relatively stable and full hysteretic loops. The values of the coefficient are based on analytical and experimental investigations of the earthquake response of yielding structures. The quantity, R, is the ratio of the required elastic strength to the yielding strength of the structure. Where the quantity R is defined, it is preferable to use the appropriate value of C_1 given by the equations in Section 3.3.3.3. Where the quantity R is not defined, as permitted for the LSP, the coefficient C_1 may be read from the expressions given in Section 3.3.1.3.1.

Coefficient C_2. This coefficient adjusts design values based on component hysteresis characteristics, stiffness degradation, and strength deterioration. See Section C3.3.3.3.2 and *FEMA 274* for additional discussion.

Coefficient C_3. For framing systems that exhibit negative post-yield stiffness, dynamic P-Δ effects may lead to significant amplification of displacements. Such effects cannot be explicitly addressed with linear procedures. No measure of the degree of negative post-yield stiffness can be explicitly included in a linear procedure.

Table 3-1 Values for Effective Mass Factor C_m[1]

No. of Stories	Concrete Moment Frame	Concrete Shear Wall	Concrete Pier-Spandrel	Steel Moment Frame	Steel Concentric Braced Frame	Steel Eccentric Braced Frame	Other
1–2	1.0	1.0	1.0	1.0	1.0	1.0	1.0
3 or more	0.9	0.8	0.8	0.9	0.9	0.9	1.0

1. C_m shall be taken as 1.0 if the fundamental period, T, is greater than 1.0 second.

3.3.1.3.2 Vertical Distribution of Seismic Forces

The vertical distribution of the pseudo lateral load shall be as specified in this section for all buildings except unreinforced masonry buildings for which the pseudo lateral loads shall be permitted to be distributed in accordance with Section 3.3.1.3.5. The lateral load F_x applied at any floor level x shall be determined in accordance with Equation (3-11) and Equation (3-12):

$$F_x = C_{vx} V \tag{3-11}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{3-12}$$

where:

C_{vx} = Vertical distribution factor

k = 2.0 for $T \geq 2.5$ seconds

= 1.0 for $T \leq 0.5$ seconds

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

V = Pseudo lateral load from Equation (3-10)

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

3.3.1.3.3 Horizontal Distribution of Seismic Forces

The seismic forces at each floor level of the building calculated using Equation (3-11) shall be distributed according to the distribution of mass at that floor level.

3.3.1.3.4 Diaphragms

Diaphragms shall be designed to resist the combined effects of the inertial force, F_{px}, calculated in accordance with Equation (3-13), and horizontal forces resulting from offsets in or changes in the stiffness of the vertical seismic framing elements above and below the diaphragm. Forces resulting from offsets in or changes in the stiffness of the vertical seismic framing elements shall be taken as the forces due to the pseudo lateral load of Equation (3-10) without reduction, unless smaller forces are justified by a limit-state or other rational analysis, and shall be added directly to the diaphragm inertial forces.

$$F_{px} = \sum_{i=x}^{n} F_i \frac{w_x}{\sum_{i=x}^{n} w_i} \tag{3-13}$$

where:

F_{px} = Total diaphragm inertial force at level x

F_i = Lateral load applied at floor level i given by Equation (3-11)

w_i = Portion of the effective seismic weight W located on or assigned to floor level i

w_x = Portion of the effective seismic weight W located on or assigned to floor level x

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

Diaphragms receiving horizontal forces from discontinuous vertical elements shall be taken as force-controlled. Actions on other diaphragms shall be considered force- or deformation-controlled as specified for diaphragm components in Chapters 5 through 8.

3.3.1.3.5 Distribution of Seismic Forces for Unreinforced Masonry Buildings with Flexible Diaphragms

For unreinforced masonry buildings with flexible diaphragms for which the fundamental period is calculated using Equation (3-9), it shall be permitted to calculate and distribute the pseudo lateral loads as follows:

1. For each span of the building and at each level, calculate period from Equation (3-9).

2. Using Equation (3-10), calculate pseudo lateral load for each span.

3. Apply the lateral loads calculated for all spans and calculate forces in vertical seismic-resisting elements using tributary loads.

4. Diaphragm forces for evaluation of diaphragms shall be determined from the results of step 3 above and distributed along the diaphragm span considering its deflected shape.

5. Diaphragm deflection shall not exceed 6 inches for this method of distribution of pseudo lateral loads to be applicable.

3.3.2 Linear Dynamic Procedure

3.3.2.1 Basis of the Procedure

If the Linear Dynamic Procedure (LDP) is selected for seismic analysis of the building, the design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements shall be determined using a linearly-elastic, dynamic analysis in compliance with the requirements of this section.

Buildings shall be modeled with linearly elastic stiffness and equivalent viscous damping values consistent with components responding at or near yield level, as defined in Section 2.4.4. Modeling and analysis procedures to calculate forces and deformations shall be in accordance with Section 3.3.2.2.

Results of the LDP shall be checked using the acceptance criteria of Section 3.4.2.

C3.3.2.1 Basis of the Procedure

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 approximate maximum displacements expected during the design earthquake, but will produce internal forces that exceed those that would be obtained in a yielding building.

Calculated internal forces typically will exceed those that the building can sustain because of anticipated inelastic response of components and elements. These 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

3.3.2.2.1 General

The ground motion characterized for dynamic analysis shall comply with the requirements of Section 3.3.2.2.2. The dynamic analysis shall be performed using the response spectrum method in accordance with Section 3.3.2.2.3 or the time-history method in accordance with Section 3.3.2.2.4.

3.3.2.2.2 Ground Motion Characterization

The horizontal ground motion shall be characterized for design by the requirements of Section 1.6 and shall be one of the following:

1. A response spectrum as specified in Section 1.6.1.5.

2. A site-specific response spectrum as specified in Section 1.6.2.1.

3. Ground acceleration time histories as specified in Section 1.6.2.2.

3.3.2.2.3 Response Spectrum Method

Dynamic analysis using the response spectrum method shall calculate peak modal responses for sufficient modes to capture at least 90% of the participating mass of the building in each of two orthogonal principal horizontal directions of the building. Modal damping ratios shall reflect the damping in the building at deformation levels less than the yield deformation.

Peak member forces, displacements, story forces, story shears, and base reactions for each mode of response shall be combined by either the SRSS (square root sum of squares) rule or the CQC (complete quadratic combination) rule.

Multidirectional seismic effects shall be considered in accordance with the requirements of Section 3.2.7.

3.3.2.2.4 Time History Method

Dynamic analysis using the time history method shall calculate building response at discrete time steps using discretized recorded or synthetic time histories as base motion. The damping matrix associated with the mathematical model shall reflect the damping in the building at deformation levels near the yield deformation.

Response parameters shall be calculated for each time history analysis. If three or more time history analyses are performed, the maximum response of the parameter of interest shall be used for design. If seven or more consistent pairs of horizontal ground motion records are used for time history analysis, use of the average of all responses of the parameter of interest shall be permitted for design.

Multidirectional seismic effects shall be considered in accordance with the requirements of Section 3.2.7. Alternatively, an analysis of a three-dimensional mathematical model using simultaneously imposed consistent pairs of earthquake ground motion records along each of the horizontal axes of the building shall be permitted.

C3.3.2.2 Modeling and Analysis Considerations

The LDP includes two analysis methods, namely, the Response Spectrum Method and the Time History Method. 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. Pairs of ground motion records for simultaneous analysis along each horizontal axis of the building should be consistent. Consistent pairs are the orthogonal motions expected at a given site based on the same earthquake.

3.3.2.3 Determination of Forces and Deformations

3.3.2.3.1 Modification of Demands

All forces and deformations calculated using either the Response Spectrum or Time History Analysis Methods 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 modified to consider the effects of torsion in accordance with Section 3.2.2.2.

3.3.2.3.2 Diaphragms

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 (3-13). 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 are justified by an approved rational analysis. Diaphragm actions need not be multiplied by the product of the modification factors C_1, C_2, and C_3.

3.3.3 Nonlinear Static Procedure

3.3.3.1 Basis of the Procedure

If the Nonlinear Static Procedure (NSP) is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is exceeded. Mathematical modeling and analysis procedures shall comply with the requirements of Section 3.3.3.2. The target displacement shall be calculated by the procedure in Section 3.3.3.3.

Calculated displacements and internal forces shall meet the acceptance criteria specified in Section 3.4.3.

C3.3.3.1 Basis of the Procedure

The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. 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.

3.3.3.2 Modeling and Analysis Considerations

3.3.3.2.1 General

The selection of a control node, the selection of lateral load patterns, the determination of the fundamental period, and analysis procedures shall comply with the requirements of this section.

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.

The component gravity loads shall be included in the mathematical model for combination with lateral loads as specified in Section 3.2.8. The lateral loads shall be applied in both the positive and negative directions, and the maximum seismic effects shall be used for design.

The analysis model shall be discretized to represent the load-deformation response of each component along its length to identify locations of inelastic action.

All primary and secondary lateral-force-resisting elements shall be included in the model, as specified in Section 3.2.2.3.

The force-displacement behavior of all components shall be explicitly included in the model using full backbone curves that include strength degradation and residual strength, if any.

Alternatively, the use of a simplified NSP analysis shall be permitted. In a simplified NSP analysis only primary lateral force resisting elements are modeled, the force-displacement characteristics of such elements are bilinear, and the degrading portion of the backbone curve is not explicitly modeled. The simplified NSP analysis shall only be used in conjunction with the acceptance criteria of Section 3.4.3.2.2. Elements not meeting the acceptance criteria for primary components shall be designated as secondary, and removed from the mathematical model.

C3.3.3.2.1 General

When using the simplified NSP analysis, care should be taken to make sure that removal of degraded elements from the model does not result changes in the regularity of the structure that would significantly alter the dynamic response. In pushing with a static load pattern, the NSP does not capture changes in the dynamic characteristics of the structure as yielding and degradation take place.

In order to explicitly evaluate deformation demands on secondary elements that are to be excluded from the model, one might consider including them in the model, but with negligible stiffness, to obtain deformations demands without significantly affecting the overall response.

3.3.3.2.2 Control Node Displacement

The control node shall be located at the center of mass at the roof of a building. For buildings with a penthouse, the floor of the penthouse shall be regarded as the level of the control node. The displacement of the control node in the mathematical model shall be calculated for the specified lateral loads.

3.3.3.2.3 Lateral Load Distribution

Lateral loads shall be applied to the mathematical model in proportion to the distribution of inertia forces in the plane of each floor diaphragm. For all analyses, at least two vertical distributions of lateral load shall be applied. One pattern shall be selected from each of the following two groups:

1. A modal pattern selected from one of the following:

   1.1. A vertical distribution proportional to the values of C_{vx} given in Equation (3-12). Use of this distribution shall be permitted only when more than 75% of the total mass participates in the fundamental mode in the direction under consideration, and the uniform distribution is also used.

   1.2. A vertical distribution proportional to the shape of the fundamental mode in the direction under consideration. Use of this distribution shall be permitted only when more than 75% of the total mass participates in this mode.

   1.3. A vertical distribution proportional to the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building, including sufficient modes to capture at least 90% of the total building mass, and using the appropriate ground motion spectrum.This distribution shall be used when the period of the fundamental mode exceeds 1.0 second.

2. A second pattern selected from one of the following:

   2.1. A uniform distribution consisting of lateral forces at each level proportional to the total mass at each level.

   2.2. An adaptive load distribution that changes as the structure is displaced. The adaptive load distribution shall be modified from the original load distribution using a procedure that considers the properties of the yielded structure.

C3.3.3.2.3 Lateral Load Distribution

The distribution of lateral inertial forces determines relative magnitudes of shears, moments, and deformations within the structure. The distribution of these forces will vary continuously during earthquake response as portions of the structure yield and stiffness characteristics change. The extremes of this distribution will depend on the severity of the earthquake shaking and the degree of nonlinear response of the structure. Use of more than one lateral load pattern is intended to bound the range of design actions that may occur during actual dynamic response.

In lieu of using the uniform distribution to bound the solution, changes in the distribution of lateral inertial forces can be investigated using adaptive load patterns that change as the structure is displaced to larger amplitudes. Procedures for developing adaptive load patterns include the use of story forces proportional to the deflected shape of the structure (Fajfar and Fischinger), the use of load patterns based on mode shapes derived from secant stiffnesses at each load step (Eberhard and Sozen), and the use of load patterns proportional to the story shear resistance at each step (Bracci et al.). Use of an adaptive load pattern will require more analysis effort, but may yield results that are more consistent with the characteristics of the building under consideration.

3.3.3.2.4 Idealized Force-Displacement Curve

The nonlinear force-displacement relationship between base shear and displacement of the control node shall be replaced with an idealized relationship to calculate the effective lateral stiffness, K_e, and effective yield strength, V_y, of the building as shown in Figure 3-1. This relationship shall be bilinear, with initial slope K_e and post-yield slope α. Line segments on the idealized force-displacement curve shall be located using an iterative graphical procedure that approximately balances the area above and below the curve. The effective lateral stiffness, K_e, shall be taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure. The post-yield slope, α, shall be determined by a line segment that passes through the actual curve at the calculated target displacement. The effective yield strength shall not be taken as greater than the maximum base shear force at any point along the actual curve.

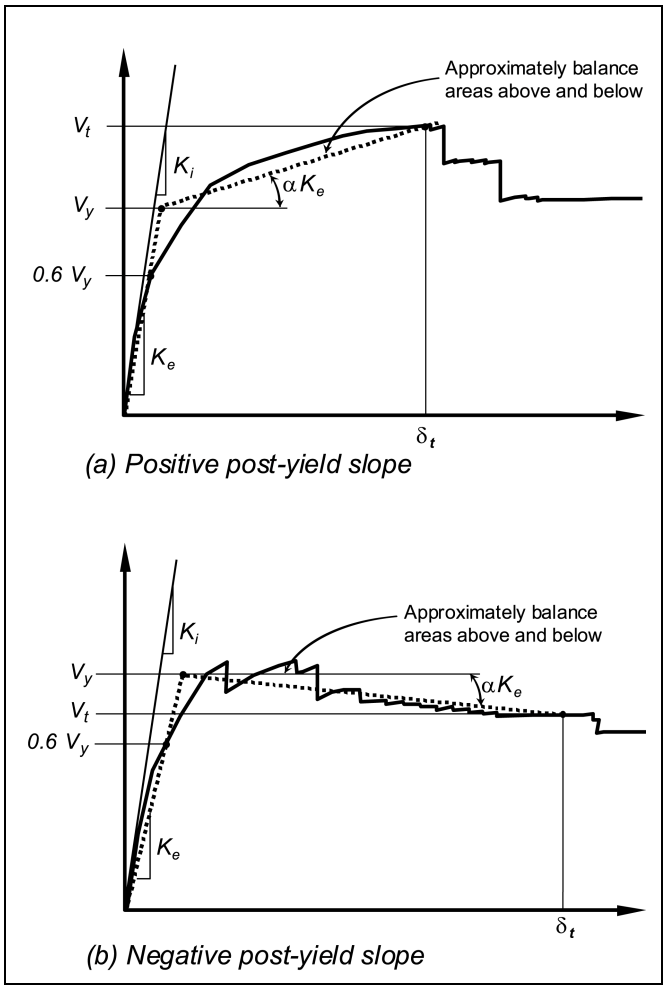

Figure 3-1 Idealized Force-Displacement Curves

3.3.3.2.5 Period Determination

The effective fundamental period in the direction under consideration shall be based on the idealized force-displacement curve defined in Section 3.3.3.2.4. The effective fundamental period, T_e, shall be calculated in accordance with Equation (3-14):

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \qquad (3\text{-}14)$$

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

3.3.3.2.6 Analysis of Mathematical Models

Separate mathematical models representing the framing along two orthogonal axes of the building shall be developed for two-dimensional analysis. A mathematical model representing the framing along two orthogonal axes of the building shall be developed for three-dimensional analysis.

The effects of horizontal torsion shall be evaluated in accordance with Section 3.2.2.2.

Independent analysis along each of the two orthogonal principal axes of the building shall be permitted unless concurrent evaluation of multidirectional effects is required by Section 3.2.7.

3.3.3.3 Determination of Forces and Deformations

3.3.3.3.1 General

For buildings with rigid diaphragms at each floor level, the target displacement, δ_t, shall be calculated in accordance with Equation (3-15) or by an approved procedure that accounts for the nonlinear response of the building.

For buildings with non-rigid diaphragms at each floor level, diaphragm flexibility shall be explicitly included in the model. The target displacement shall be calculated as specified for rigid diaphragms, except that it shall be amplified by the ratio of the maximum displacement at any point on the roof to the displacement at the center of mass of the roof (δ_{max}/δ_{cm}). δ_{max} and δ_{cm} shall be based on a response spectrum analysis of a three-dimensional model of the building. The target displacement so calculated shall be no less than that displacement given by Equation (3-15). No line of vertical seismic framing shall be evaluated for displacements smaller than the target displacement.

Alternatively, for buildings with flexible diaphragms at each floor level, a target displacement shall be calculated for each line of vertical seismic framing. The target displacement for an individual line of vertical seismic framing shall be as specified for buildings with rigid diaphragms, except that the masses shall be assigned to each line on the basis of tributary area.

Forces and deformations corresponding to the control node displacement equaling or exceeding the target displacement shall comply with acceptance criteria of Section 3.4.3.

3.3.3.3.2 Target Displacement

The target displacement, δ_t, at each floor level shall be calculated in accordance with Equation (3-15) and as specified in Section 3.3.3.3.1.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \qquad (3\text{-}15)$$

where:

C_0 = Modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system calculated using one of the following procedures:

- 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. This procedure shall be used if the adaptive load pattern defined in paragraph 2.2 of Section 3.3.3.2.3 is used; or
- 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 $T_e \geq T_S$

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

but not greater than the values given in Section 3.3.1.3 nor less than 1.0.

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

T_S = 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 per Sections 1.6.1.5 and 1.6.2.1.

R = Ratio of elastic strength demand to calculated yield strength coefficient calculated by Equation (3-16).

C_2 = Modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response. Values of C_2 for different framing systems and Structural Performance Levels shall be obtained from Table 3-3. Alternatively, use of C_2 = 1.0 shall be permitted for nonlinear procedures.

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 (3-17) but not to 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, as calculated in Sections 1.6.1.5 and 1.6.2.1.

g = acceleration of gravity

The strength ratio R shall be calculated in accordance with Equation (3-16):

$$R = \frac{S_a}{V_y/W} \cdot C_m \qquad (3\text{-}16)$$

where S_a is defined above, and:

V_y = Yield strength calculated using results of the NSP for the idealized nonlinear force-displacement curve developed for the building in accordance with Section Section 3.3.3.2.4

W = Effective seismic weight, as calculated in Section 3.3.1.3.1

$_m$ = Effective mass factor from Table 3-1. Alternatively, C_m taken as the effective model mass calculated for the fundamental mode using an Eigenvalue analysis shall be permitted

Coefficient C_3 shall be calculated in accordance with Equation (3-17) if the relation between base shear force and control node displacement exhibits negative post-yield stiffness.

$$C_3 = 1.0 + \frac{|\alpha|(R-1)^{3/2}}{T_e} \qquad (3\text{-}17)$$

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 shall be characterized by a bilinear relation as shown in Figure 3-1

Table 3-2 Values for Modification Factor C_0 [1]

	Shear Buildings[2]		Other Buildings
Number of Stories	**Triangular Load Pattern (1.1, 1.2, 1.3)**	**Uniform Load Pattern (2.1)**	**Any Load Pattern**
1	1.0	1.0	1.0
2	1.2	1.15	1.2
3	1.2	1.2	1.3
5	1.3	1.2	1.4
10+	1.3	1.2	1.5

1. Linear interpolation shall be used to calculate intermediate values.
2. Buildings in which, for all stories, interstory drift decreases with increasing height.

Table 3-3 Values for Modification Factor C_2

	$T \le 0.1$ second[3]		$T \ge T_S$ second[3]	
Structural Performance Level	**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 any combination of the following components, elements, or frames: ordinary moment-resisting frames, concentrically-braced frames, frames with partially-restrained connections, tension-only braces, unreinforced masonry walls, shear-critical, piers, and spandrels of reinforced concrete or masonry.
2. All frames not assigned to Framing Type 1.
3. Linear interpolation shall be used for intermediate values of *T*.

C3.3.3.3.2 Target Displacement

Calculation Method. This standard presents the Coefficient Method for calculating target displacement. Other procedures can also be used. Section C3.3.3.3 of *FEMA 274* presents additional background information on the Coefficient Method and another acceptable procedure referred to as the Capacity Spectrum Method.

Coefficient C_0. This coefficient accounts for the difference between the roof displacement of an MDOF building and the displacement of the equivalent SDOF system. Using only the first mode shape (ϕ_1) and elastic behavior, coefficient C_0 is equal to the first-mode participation factor at the roof (control node) level. If the absolute value of the roof (control node) ordinate of each mode shape is set equal to unity, the value of coefficient C_0 is equal to the first mode mass participation factor.

Explicit calculation of C_0 using the actual deflected shape is more accurate and may be beneficial in terms of lower amplification of target displacement. The actual shape vector may take on any form, particularly since it is intended to simulate the time-varying deflection profile of the building responding inelastically to the ground motion. Based on past studies, the use of a shape vector corresponding to the deflected shape at the target displacement level may be more appropriate. This shape will likely be different from the elastic first-mode shape. While the use of such a deflected shape vector in the estimation of C_0 is preferred, the choice of the elastic first-mode shape vector is a simpler alternative that takes into account at least the relative mass distribution over the height of the structure.

The use of the tabulated values, which are based on a straight-line vector with equal masses at each floor level, is approximate, particularly if masses vary much over the height of the building, and may be overly conservative.

Coefficient C_1. This coefficient accounts for the observed difference in peak displacement response amplitude for nonlinear response as compared with linear response, as observed for buildings with relatively short initial vibration periods. For use with the NSP, it is recommended to calculate the value of this coefficient using the strength ratio, R, given by Equation (3-16). Additional discussion of this coefficient is in the commentary to Section 3.3.1.3.1.

Recent studies suggest that maximum elastic and inelastic displacement amplitudes may differ considerably if either the strength ratio R is large or the building is located in the near field of the causative fault. Specifically, the inelastic displacements will exceed the elastic displacement. If the strength ratio exceeds five, it is recommended that a displacement larger than the elastic displacement be used as the basis for calculating the target displacement.

Coefficient C_2. If the hysteresis loops exhibit significant pinching or stiffness deterioration, the energy absorption and dissipation capacities decrease, and larger displacement excursions should be expected. This effect is important for short-period, low-strength structures with very pinched hysteresis loops. Framing Types 1 and 2 are introduced for the purpose of cataloguing systems prone to exhibit pinching and strength degradation—that is, Type 1. Type 2 systems are those not specifically identified as Type 1. Values for C_2 are reduced for smaller levels of damage; that is, the values for C_2 are smaller for the Immediate Occupancy Structural Performance Level (little to no damage) than for the Collapse Prevention Structural Performance Level (moderate to major damage). However, the selected performance level is not a direct measure of the extent of inelastic behavior that a building will experience, particularly in zones of moderate and low seismicity. If, for example, the structure meets a performance level superior to the selected performance level, use of a lower value of C_2 corresponding to the actual performance level achieved would be justified. This may be done by interpolation between the C_2 values specified for the performance levels above and below the level achieved. The values for C_2 given in Table 3-3 are intended to account for both stiffness degradation and strength deterioration, and are based on judgment at the time this standard was written.

While single-degree-of-freedom systems with pinched hysteretic behavior do experience amplified displacements, some recent research has shown that the displacement response of multiple-degree-of-freedom building structures is not significantly affected by the pinched hysteretic behavior of individual elements. These new results are not yet entirely conclusive. At present, Table 3-3 has been retained in this standard, but the use of $C_2 = 1.0$ is permitted.

Coefficient C_3. P-Δ effects caused by gravity loads acting through the deformed configuration of a building will always result in an increase in lateral displacements. Static P-Δ effects can be captured using procedures set forth in Section 3.2.5. If P-Δ effects result in a negative post-yield stiffness in any one story, such effects may significantly increase the interstory drift and the target displacement. The degree by which dynamic P-Δ effects increase displacements depends on (1) the ratio α of the negative post-yield stiffness to the effective elastic stiffness, (2) the fundamental period of the building, (3) the strength ratio, (4) the hysteretic load-deformation relations for each story, (5) the frequency characteristics of the ground motion, and (6) the duration of the strong ground motion. Because of the number of parameters involved, it is difficult to capture dynamic P- Δ effects with a single modification factor. Coefficient C_3, calculated only for those buildings that exhibit negative post-yield stiffness, given by Equation (3-17), represents a substantial simplification and interpretation of much analysis data.

3.3.3.3.3 Modification of Demands

The target displacement shall be modified to consider the effects of horizontal torsion in accordance with Section 3.2.2.2.

3.3.3.3.4 Diaphragms

Diaphragms shall be designed to resist the combined effects of the horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm, and the diaphragm forces determined using either Section 3.3.1.3.4 or Section 3.3.2.3.2.

3.3.4 Nonlinear Dynamic Procedure

3.3.4.1 Basis of the Procedure

If the Nonlinear Dynamic Procedure (NDP) is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be subjected to earthquake shaking represented by ground motion time histories in accordance with Section 1.6.2.2 to obtain forces and displacements.

Calculated displacements and internal forces shall be compared directly with acceptance criteria specified in Section 3.4.3.

C3.3.4.1 Basis of the Procedure

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 time histories. Calculated response can be highly sensitive to characteristics of individual ground motions; therefore, the analysis should be carried out 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.

3.3.4.2 Modeling and Analysis Considerations

3.3.4.2.1 General

The modeling and analysis requirements specified in Section 3.3.3.2 for the NSP shall apply to the NDP, excluding considerations of control node and target displacements.

3.3.4.2.2 Ground Motion Characterization

For the NDP, earthquake shaking shall be characterized by discretized recorded or synthetic earthquake records as base motion meeting the requirements of Section 1.6.2.2.

3.3.4.2.3 Time History Method

For the NDP, Time History Analysis shall be performed using horizontal ground motion time histories prepared according to the requirements of Section 1.6.2.2.

Multidirectional seismic effects shall be accounted for in accordance with Section 3.2.7. Alternatively, an analysis of a three-dimensional mathematical model using simultaneously imposed consistent pairs of earthquake ground motion records along each of the horizontal axes of the building shall be permitted.

3.3.4.3 Determination of Forces and Deformations

Forces and deformations shall be determined in accordance with Section 3.3.2.2.4.

3.3.4.3.1 Modification of Demands

The effects of torsion shall be considered in accordance with Section 3.2.2.2.

3.3.4.3.2 Diaphragms

Diaphragms shall be designed to resist the effects of the seismic forces calculated by dynamic analysis including the effects of 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 Section 3.3.1 and Section 3.3.2 shall satisfy the requirements of Section 3.4.2. Components and elements analyzed using the nonlinear procedures of Section 3.3.3 and Section 3.3.4 shall satisfy the requirements of Section 3.4.3.

Prior to selecting component acceptance criteria, components shall be classified as primary or secondary, and actions shall be classified as deformation-controlled or force-controlled, as defined in Section 2.4.4.

Foundations shall satisfy the criteria specified in Chapter 4.

3.4.2 Linear Procedures

3.4.2.1 Design Forces and Deformations

Component design forces and deformations shall be calculated in accordance with linear analysis procedures of Sections 3.3.1 or 3.3.2.

3.4.2.1.1 Deformation-Controlled Actions

Deformation-controlled design actions Q_{UD} shall be calculated in accordance with Equation (3-18):

$$Q_{UD} = Q_G \pm Q_E \tag{3-18}$$

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} = Deformation-controlled design action due to gravity loads and earthquake loads

C3.4.2.1.1 Deformation-Controlled Actions

Because of possible anticipated nonlinear response of the structure, the design actions as represented by Equation (3-18) may exceed the actual strength of the component or element to resist these actions. The acceptance criteria of Section 3.4.2.2.1 take this overload into account through use of a factor, *m*, which is an indirect measure of the nonlinear deformation capacity of the component or element.

3.4.2.1.2 Force-Controlled Actions

Force-controlled design actions, Q_{UF}, shall be calculated using one of the following methods:

1. Q_{UF} shall be taken as the maximum action that can be developed in a component based on a limit-state analysis considering the expected strength of the components delivering load to the component under consideration, or the maximum action developed in the component as limited by the nonlinear response of the building.

2. Alternatively, Q_{UF} shall be calculated in accordance with Equation (3-19).

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3 J} \tag{3-19}$$

where:

Q_{UF} = Force-controlled design action due to gravity loads in combination with earthquake loads

J = Force-delivery reduction factor, greater than or equal to 1.0, taken as the smallest DCR of the components in the load path delivering force to the component in question, calculated in accordance with Equation (2-1).

Alternatively, values of J equal to 2.0 in Zones of High Seismicity, 1.5 in Zones of Moderate Seismicity, and 1.0 in Zones of Low Seismicity shall be permitted when not based on calculated DCRs. J shall be taken as 1.0 for the Immediate Occupancy Structural Performance Level. In any case where the forces contributing to Q_{UF} are delivered by components of the lateral force resisting system that remain elastic, J shall be taken as 1.0.

C3.4.2.1.2 Force-Controlled Actions

The basic approach for calculating force-controlled actions for design differs from that used for deformation-controlled actions because nonlinear deformations associated with forced-controlled actions are not permitted. Therefore, force demands for force-controlled actions should not exceed the force capacity (strength).

Ideally, an inelastic mechanism for the structure will be identified, and the force-controlled actions, Q_{UF}, for design will be determined by limit analysis using that mechanism. This approach will always produce a conservative estimate of the design actions, even if an incorrect mechanism is selected. Where it is not possible to use limit (or plastic) analysis, or in cases where design forces do not produce significant nonlinear response in the building, it is acceptable to determine the force-controlled actions for design using Equation (3-19).

Coefficients C_1, C_2, and C_3 were introduced in Equation (3-10) to amplify the design base shear to achieve a better estimate of the maximum displacements expected for buildings responding in the inelastic range. Displacement amplifiers, C_1, C_2, and C_3 are divided out of Equation (3-19) when seeking an estimate of the force level present in a component when the building is responding inelastically.

Since J is included for force-controlled actions, it may appear to be more advantageous to treat an action as force-controlled when m-factors are less than J. However, proper application of force-controlled criteria requires a limit state analysis of demand and lower bound calculation of capacity that will yield a safe result whether an action is treated as force- or deformation-controlled.

3.4.2.2 Acceptance Criteria for Linear Procedures

3.4.2.2.1 Deformation-Controlled Actions

Deformation-controlled actions in primary and secondary components and elements shall satisfy Equation (3-20).

$$m\kappa Q_{CE} \geq Q_{UD} \tag{3-20}$$

where:

m = Component or element demand modifier (factor) to account for expected ductility associated with this action at the selected Structural Performance Level. m-factors are specified in 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 defined in Section 2.2.6.4

Q_{CE}, the expected strength, shall be determined considering all coexisting actions on the component under the design loading condition by procedures specified in Chapters 4 through 8.

3.4.2.2.2 Force-Controlled Actions

Force-controlled actions in primary and secondary components and elements shall satisfy Equation (3-21):

$$\kappa Q_{CL} \geq Q_{UF} \tag{3-21}$$

where:

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

Q_{CL}, the lower-bound strength, shall be determined considering all coexisting actions on the component under the design loading condition by procedures specified in Chapters 5 through 8.

3.4.2.2.3 Verification of Design Assumptions

In addition to the requirements in Section 3.2.9, the following verification of design assumptions shall be made.

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.

3.4.3 Nonlinear Procedures

3.4.3.1 Design Forces and Deformations

Component design forces and deformations shall be calculated in accordance with nonlinear analysis procedures of Sections 3.3.3 or 3.3.4.

3.4.3.2 Acceptance Criteria for Nonlinear Procedures

3.4.3.2.1 Deformation-Controlled Actions

Primary and secondary components shall have expected deformation capacities not less than maximum deformation demands calculated at the target displacement. Primary and secondary component demands shall be within the acceptance criteria for secondary components at the selected Structural Performance Level. Expected deformation capacities shall be determined considering all coexisting forces and deformations in accordance with Chapters 4 through 8.

The base shear at the target displacement, V_t, shall not be less than 80% of the effective yield strength of the structure, V_y, defined in Section 3.3.3.2.4.

Acceptance criteria for the simplified NSP analysis of Section 3.3.3.2.1 shall be as specified in Section 3.4.3.2.2.

C3.4.3.2.1 Deformation-Controlled Actions

When all components are explicitly modeled with full backbone curves, the NSP can be used to evaluate the full contribution of all components to the lateral force resistance of the structure as they degrade to residual strength values. When degradation is explicitly evaluated in the NSP, components can be relied upon for lateral-force resistance out to the secondary component limits of response. As components degrade, the post-yield slope of the force-displacement curve becomes negative, and the target displacement begins to grow. The procedure is self-limiting in that small negative post-yield slopes will result in large amplifications of the target displacement through the C_3 coefficient, quickly causing nonconvergence of the solution. As an additional control on the extent of degradation, V_t is limited to not less than 80% of V_y.

3.4.3.2.2 Deformation-Controlled Actions for the Simplified Nonlinear Static Analysis

Primary and secondary components modeled using the alternative simplified NSP analysis of Section 3.3.3.2.1 shall meet the requirements of this section. Expected deformation capacities shall not be less than maximum deformation demands calculated at the target displacement. Primary component demands shall be within the acceptance criteria for primary components at the selected Structural Performance Level. Demands on other components shall be within the acceptance criteria for secondary components at the selected Structural Performance Level. Expected deformation capacities shall be determined considering all coexisting forces and deformations by procedures specified in Chapters 4 through 8.

C3.4.3.2.2 Deformation-Controlled Actions for the Simplified Nonlinear Static Analysis

In the simplified NSP analysis, primary components are not modeled with full backbone curves. Degradation cannot be explicitly evaluated and degraded elements cannot be reliably used to the secondary component limits of response. For this reason, the lateral-force-resistance of the structure consists of primary components measured against primary component acceptance criteria.

3.4.3.2.3 Force-Controlled Actions

Primary and secondary components shall have lower-bound strengths not less than the maximum design forces. Lower-bound strengths shall be determined considering all coexisting forces and deformations by procedures specified in Chapters 4 through 8.

3.4.3.2.4 Verification of Design Assumptions

In addition to the requirements in Section 3.2.9, the following verification of design assumptions shall be made.

Flexural plastic hinges shall not form away from component ends unless they are explicitly accounted for in modeling and analysis.