C3. Modeling and Analysis (Systematic Rehabilitation)

C3.1 Scope

Section 3.1 provides a road map for the user of Chapter 3. Much information relevant to the provisions of Chapter 3 can be found in Chapters 2, 5, 6, 7, and 8; the relationship of the provisions in Chapter 3 to those in other chapters is summarized in Section 3.1. The reader should be familiar with the relevant information presented in these chapters before implementing the analysis methods presented in Chapter 3.

The *Guidelines* present strategies for both Systematic Rehabilitation and Simplified Rehabilitation. The procedures in Chapter 3 are applicable only to the Systematic Rehabilitation Method.

C3.2 General Requirements

C3.2.1 Analysis Procedure Selection

Chapter 3 provides guidance for implementation of the *Guidelines*' four Analysis Procedures for systematic rehabilitation of buildings. Guidance on selection of the appropriate Analysis Procedure is presented in Chapter 2.

In the Linear Static Procedure (LSP) and the Linear Dynamic Procedure (LDP), the term "linear" implies "linearly elastic." However, geometric nonlinearities associated with gravity loads acting through lateral displacements may be included in the analysis model. Furthermore, components of concrete and masonry may be modeled using cracked-section properties, so that some material nonlinearity is modeled, even though the numerical analysis assumes perfectly linear behavior. In the Nonlinear Static Procedure (NSP) and the Nonlinear Dynamic Procedure (NDP), the term "nonlinear" refers to material nonlinearities (inelastic material response); geometric nonlinearities may also be considered.

C3.2.2 Mathematical Modeling

C3.2.2.1 Basic Assumptions

The *Guidelines* promote the use of three-dimensional mathematical models for the systematic rehabilitation analysis of buildings, but were written recognizing that fully three-dimensional modeling is not always feasible given available analysis tools, especially those for nonlinear analysis. Therefore, three-dimensional models are required only in certain cases known to require such models.

Where two-dimensional models are used, the model should be developed recognizing the three-dimensional nature of the building structure. For example, shear walls and other bracing systems commonly have cross sections that form "L," "T," and other three-dimensional shapes. Strength and stiffness of a "T"-shaped wall should be developed including the effect of the flange.

Examples of cases 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.

C3.2.2.2 Horizontal Torsion

Research shows that effects of inelastic dynamic torsional response are more severe than effects indicated by linearly elastic models. Furthermore, it is clear that inelastic torsion can be driven both by stiffness eccentricities and by strength eccentricities; the latter are not directly indicated in linearly elastic models, but often may be identified by inspection of strengths of the earthquake-resisting components and elements. Premature failure of one or more components or elements in an otherwise symmetric structure may lead to torsional response. Structures with low levels of redundancy are likely to be more sensitive to this latter aspect than are highly redundant structures. The rehabilitation design should strive to improve the redundancy and the torsional stiffness and strength regularity of the building.

Currently, there are insufficient data available to correlate results of NSP and NDP results for torsionally sensitive systems. In the judgment of the writers, the NSP may underestimate torsional effects in some cases and overestimate effects in others.

The effects of torsion are classed as either actual, or accidental. Actual torsion is due to the eccentricity between centers of mass and stiffness. Accidental torsion is intended to cover the effects of several factors not addressed in the *Guidelines*. These factors include the rotational component of the ground motion;

differences between the computed and actual stiffnesses, strengths, and dead-load masses; and unfavorable distributions of dead- and live-load masses. The effects of accidental torsion are typically estimated by displacing the centers of mass in the same direction at one time and calculating the resulting distribution of displacements.

Checking the effects of torsion can be an onerous and time-consuming task. In the judgment of the writers, the additional effort associated with calculating the increase in component forces and deformations due to torsion is not warranted unless the effects of torsion are significant. The 10% threshold on additional displacement—due to either actual or accidental torsion—is based on judgment, not on hard data. 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. Examples of such systems are presented in Figure C3-1.

Three-dimensional models are preferred by the writers; such models likely provide considerably improved insight into building response. However, analysis of two-dimensional mathematical models is still favored by many engineers. An increase in displacement due to torsion exceeding 50% of the displacement of the center of mass is sufficient reason to require the engineer to prepare a three-dimensional mathematical model. In the event that such increases due to torsion are calculated, the engineer is strongly encouraged to modify the layout of the framing system and to substantially increase the torsional stiffness of the building frame.

The rules presented in the *Guidelines* for including the effects of horizontal torsion for the analysis of two-dimensional models are approximate and arguably punitive. The intent of these three requirements is to provide a simple means by which to account for torsion.

Note that torsional response causes nonuniform stiffness degradation of earthquake-resisting elements, which in turn further amplifies torsion calculated from elastic analysis. This behavior is not picked up by linear procedures. Therefore, for buildings with large torsion, nonlinear procedures are recommended.

C3.2.2.3 Primary and Secondary Actions, Components, and Elements

The designation of primary and secondary actions, components, and elements has been introduced to allow some flexibility in the rehabilitation analysis and design process. Primary components, elements, or actions are those that the engineer relies on to resist the specified earthquake effects. Secondary components are those that the engineer does not rely on to resist the specified earthquake effects. Typically, the secondary designation will be used when a component, element, or action does not add considerably or reliably to the earthquake resistance. In all cases, the engineer must verify that gravity loads are sustained by the structural system, regardless of the designation of primary and secondary components, elements, and actions.

The secondary designation typically will be used when one or both of the following cases apply.

1. In the first case, the secondary designation may be used when a component, element, or action does not contribute significantly or reliably to resist earthquake effects. A gypsum partition is a component that might be designated secondary in a building because it does not provide significant stiffness or strength. A slab-column interior frame is an element that might be designated as secondary in a building braced by much stiffer and stronger perimeter frames or shear walls. Moment resistance at the pinned base of a column where it connects to the foundation is an action that might be designated as secondary because the moment resistance is low, relative to the entire system resistance.

2. In the second case, the secondary designation may be used when a component, element, or action is deformed beyond the point where it can be relied on to resist earthquake effects. An example is coupling beams connecting two wall piers. It is conceivable that these beams will exhaust their deformation capacity before the entire structural system capacity is reached. In such cases, the engineer may designate these as secondary, allowing them to be deformed beyond their useful limits, provided that damage to these secondary components does not result in loss of gravity load capacity.

The manner in which primary and secondary components are handled differs for the linear and nonlinear procedures. In the linear procedures, only primary components, elements, and actions are

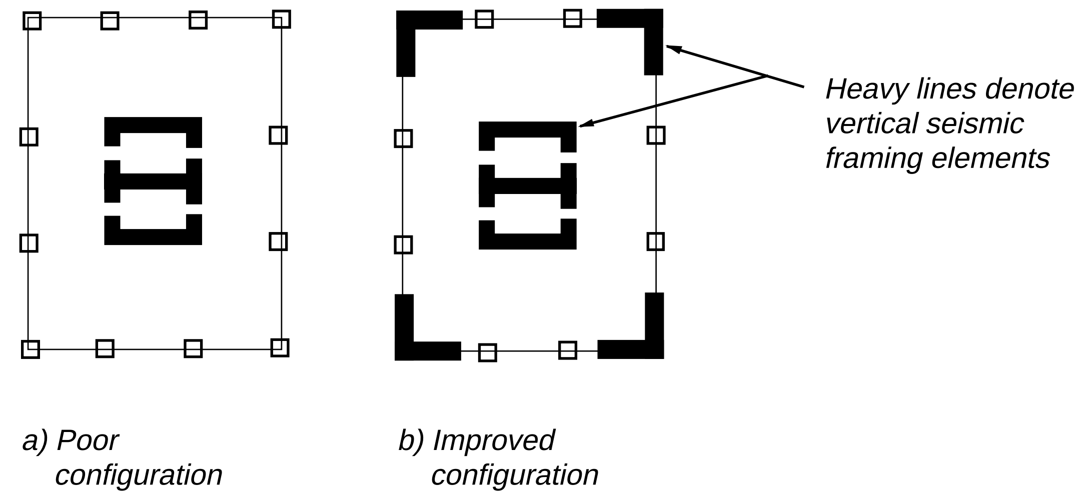


Figure C3-1 Examples of Torsional Redundancy and Torsional Stiffness

permitted to be included in the analysis model. Because of probable degradation of strength and stiffness, secondary components, elements, and actions are not permitted to be included in the linearly elastic analysis model. However, secondary components must still be checked against the acceptance criteria given in Chapters 5 through 8. In the nonlinear procedures, since strength degradation can be modeled, both primary and secondary components, elements, and actions are to be included in the nonlinear procedure model, and are to be checked against the acceptance criteria in Chapters 5 through 8.

For linear procedures, the *Guidelines* require that no more than 25% of the lateral resistance be provided by secondary components. The main reason for this limitation is that sudden loss of lateral-force-resisting components or elements can result in irregular response of a building that is difficult to detect. An example is a masonry infill wall that, if it collapses from one story of an infilled frame, may result in a severe strength and stiffness irregularity in the building. A secondary reason is to prevent the engineer from manipulating the analysis model to minimize design actions on critical components and elements. In the linear models, this 25% criterion can be checked by including the secondary components in the analysis model and examining their stiffness contribution.

Where secondary components contribute significantly to the stiffness and/or strength of the building, it is necessary to consider their effect on regularity classification of the building. In the linear procedures, it is not permitted in the analysis model to include stiffness associated with secondary components. However, if substantial secondary components result in irregular response—which can be determined by first including them in a preliminary analysis model—then the building should still be classified as irregular.

Nonstructural components and elements can profoundly, and in some cases negatively, influence the response of a building. The 10% rule of this section is based on judgment.

C3.2.2.4 Deformation- and Force-Controlled Actions

The method used for evaluating acceptance of an action is dependent on whether the action is classified as deformation-controlled or force-controlled. Deformation-controlled actions (forces or moments) are those actions for which the component has, by virtue of its detailing and configuration, capacity to deform inelastically without failure. Furthermore, a deformation-controlled action is limited to the action at the location of inelastic deformation. All other actions are designated as force-controlled actions.

Consider a cantilever column resisting axial force, shear, and bending moment. If the column has flexural ductility capacity at the connection with the footing, and if the rehabilitation design allows flexural yielding at that location, then the associated action is considered to be a deformation-controlled action. Assuming that inelastic deformation associated with axial force, shear, or moment at other locations is not permitted as part of the design, these actions are designated force-controlled actions. Table C3-1 provides examples of deformation- and force-controlled actions in common seismic framing systems.

Table C3-1 Typical Deformation-Controlled and Force-Controlled Actions

Component	Deformation-Controlled Action	Force-Controlled Action
Moment Frames • Beams • Columns • Joints	 Moment (M) M --	 Shear (V) Axial load (P), V V[1]
Shear Walls	M, V	P
Braced Frames • Braces • Beams • Columns • Shear Link	 P -- -- V	 -- P P P, M
Connections	--	P, V, M

1. Shear may be a deformation-controlled action in steel moment frame construction.

C3.2.2.5 Stiffness and Strength Assumptions

Element and component stiffness and strength assumptions specified for the *Guidelines* may differ from those commonly used in the design of new buildings. For example, reduced stiffnesses corresponding to effective cracked sections are used for concrete building analyses, whereas it has been common practice to base new designs on analyses using gross-section properties. Expected strengths, corresponding to expected material properties, are also common in the *Guidelines*, as opposed to design strengths as specified in codes for new building design. The engineer should review the stiffness and strength specifications of the relevant materials chapters of the *Guidelines* (Chapters 4 through 8, and 11) and use those values unless, through familiarity and expertise with the earthquake response and design issues, the engineer is able to identify more appropriate stiffness and strength properties.

For the NSP, it is likely that component load-deformation behavior will be represented using multilinear relationships of the types illustrated in Figure 2-4. Considerable judgment may be required in selecting the appropriate degree of complexity of the model. In most cases, simple models are preferred. The choice of the model may be guided by the following issues.

- One of the simplest component models for the NSP is a bilinear model consisting of an initial linear stiffness to yield, followed by a reduced linear stiffness. This model requires only four pieces of information: a representative elastic stiffness, the expected yield force, a post-yield stiffness, and a limiting deformation, δ_u, corresponding to a target Performance Level. Note that if a component exhibits reliable strain hardening, it is advisable to include a strain-hardening stiffness, because its neglect will lead to an overestimation of P-Δ effects and an underestimation of the maximum forces that can be delivered to force-controlled components. The bilinear model may be adequate for cases in which exceedence of the limiting deformation δ_u is unacceptable at all Performance Levels, and therefore knowledge of component behavior beyond this deformation becomes unnecessary.

- For cases in which significant component strength deterioration constitutes an acceptable state (e.g., a beam whose loss of bending resistance at the connection will not pose a life-safety hazard), the model shown as Type 1 Curve in Figure 2-4 may be appropriate. In this case, a residual strength, which could be zero, needs to be specified. The incorporation of the residual strength range in the analytical model is necessary to permit redistribution of internal forces if the deformation threshold at point 2 in the curve is exceeded.

Section 3.2.2.3 provides guidance on primary and secondary component definition, including when the stiffness of certain components, elements, or actions can be excluded from the analysis model.

C3.2.2.6 Foundation Modeling

Chapter 4 presents guidelines for stiffness and strength of foundation materials, and Chapters 5 through 8 present guidelines for steel, concrete, wood, and masonry components and elements of foundations.

Where the foundation is assumed to be rigid in the evaluation, it is necessary to evaluate the forces applied from the structure to the foundation using the acceptance criteria of Chapters 4 through 8, and 10. If the design actions exceed the allowable values, then either the structure can be rehabilitated to achieve acceptance, or the mathematical model can be modified to include the foundation according to the guidelines of Chapter 4.

C3.2.3 Configuration

Configuration plays an important role in the seismic response of buildings. Poorly-configured buildings (in many cases irregular buildings) have performed poorly in recent earthquakes (EERC, 1995; EERI, 1996). Furthermore, regular buildings can be more reliably evaluated than irregular buildings. As such, designers are encouraged to add seismic framing elements in locations that will improve the regularity of a building. Judicious location of new framing to improve regularity will simplify the analysis process and likely ensure that the analysis results will more closely represent the actual response of the building in an earthquake.

Contribution of secondary components to stiffness of the structure is expected to vary substantially during an earthquake event. In the initial earthquake excursions, secondary components are fully effective. During the latter part of an earthquake, the secondary components can lose a significant part of their strength and stiffness. For a structure to be considered regular, it needs to satisfy regularity requirements for both cases with and without contribution of secondary components.

C3.2.4 Floor Diaphragms

Floor diaphragms are a key element of the seismic load path in a building. Diaphragms transfer seismically-induced inertia forces at floor and roof levels to vertical elements of the seismic framing system, and distribute forces among vertical elements where relative stiffnesses and strengths of vertical elements differ from location to location.

In the *Guidelines*, diaphragms in provisions for Systematic Rehabilitation are classed as rigid, stiff, or flexible. Diaphragm stiffnesses in Simplified Rehabilitation are defined differently (Chapter 10). A rule for classifying diaphragm stiffness is presented; the rule is based on the relative stiffness of the diaphragm and the vertical seismic framing. Information on the stiffness and strength of diaphragms composed of different materials is presented in Chapters 5 through 8. Such information shall be used to compare the maximum lateral deformation of a diaphragm with the average inter-story drift of the story below the diaphragm.

Diaphragm flexibility results in: (1) an increase in the fundamental period of the building, (2) decoupling of the vibrational modes of the horizontal and vertical seismic framing, and (3) modification of the inertia force distribution in the plane of the diaphragm.

There are numerous single-story buildings with flexible diaphragms. For example, precast concrete tilt-up buildings with timber-sheathed diaphragms are common throughout the United States. An equation for the fundamental period of a single-story building with a flexible diaphragm is presented in Equation 3-5. Terms used in this equation are defined schematically in Figure C3-2. To calculate the fundamental period using the Rayleigh method (Clough and Penzien, 1993), a lateral load equal to the weight of the building is applied to the building in accordance with the weight distribution, and the average wall displacement, Δ_w, and diaphragm deformation, Δ_d, are calculated.

Evaluation of diaphragm demands should be based on the likely distribution of horizontal inertia forces (Mehrain and Graf, 1990). Such a distribution may be given by Equation C3-1 below; this distribution is illustrated in Figure C3-3.

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

where:

f_d = Inertial load per foot

F_d = Total inertial load on a flexible diaphragm

x = Distance from the centerline of the flexible diaphragm

L_d = Distance between lateral support points for diaphragm

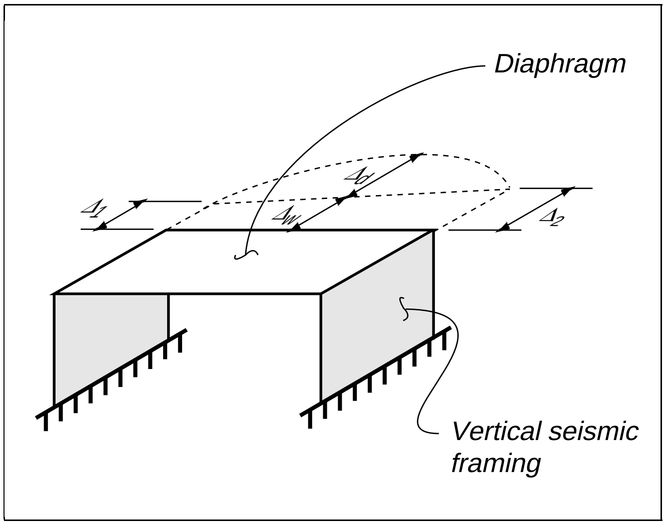


Figure C3-2 Diaphragm and Wall Displacement Terminology

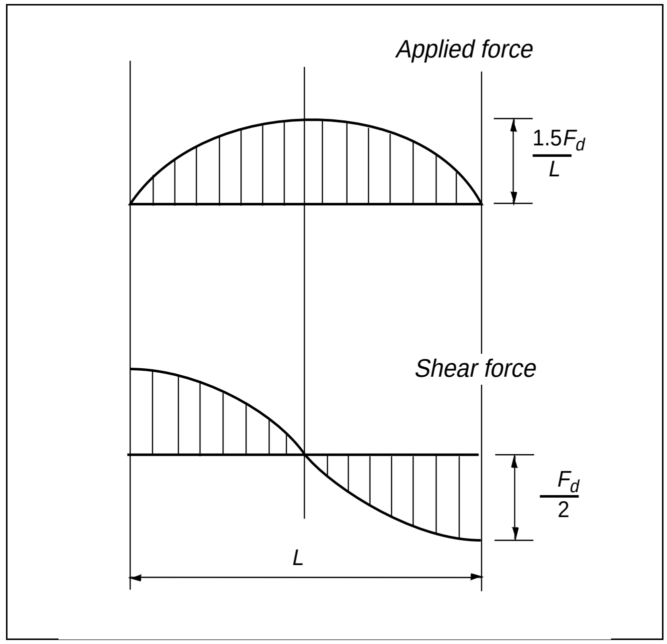
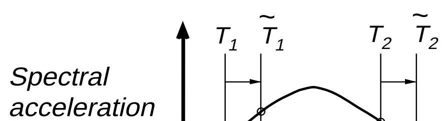


Figure C3-3 Plausible Force Distribution in a Flexible Diaphragm

C3.2.5 P-Δ Effects



element or finite difference formulation. The direct method is amenable to Linear and Nonlinear Dynamic Procedures. For the impedance function method, impedance functions representing the force-displacement characteristics of the foundation soil are specified; the soil impedance functions can be dependent or independent of excitation frequency and may include both stiffness and damping. Frequency-dependent formulations typically require frequency-domain solutions and are unsuitable for nonlinear procedures. The evaluation of foundation stiffness values, using the procedures set forth in Section 4.4.2, constitutes an impedance function approach using frequency-independent stiffness values. A discussion of methods for SSI analysis may be found in the *ASCE Standard for the Seismic Analysis of Safety-Related Nuclear Structures and Commentary* (ASCE, 1986).

C3.2.6.1 Procedures for Period and Damping

The procedures that are referenced in Section 3.2.6.1 of the *Guidelines* provide a means to calculate the effective building period and damping of the combined soil-structure system. The effective fundamental period of the building is used to determine the response spectrum acceleration used in Equation 3-6. Note that the referenced NEHRP *Provisions* (BSSC, 1995) has a typographical error in the equation for *T*. ATC (1984) Section 6.2.1 contains the correct equation for *T*.

C3.2.7 Multidirectional Excitation Effects

The rules governing multidirectional excitation effects are similar to those of BSSC (1995). Greater attention to the issue may be warranted for existing buildings, because of the greater likelihood that existing buildings will be vulnerable to brittle or low-ductility failures in force-controlled components that are overloaded by effects of multidirectional loading. The effects may be particularly important for certain vertical-load-carrying components, such as corner columns, that may receive significant overturning axial loads due to lateral loading along each of the principal horizontal axes of the building.

The 30% combination rule is a procedure that may be applied for any of the Analysis Procedures. To clarify the intention of the combination rule, consider an example of a column design. Under longitudinal loading, denote axial load as P^L, moment about *x* axis as M_x^L, and moment about *y* axis as M_y^L. Under transverse loading, similarly use P^T, M_x^T, and M_y^T. Design actions are then determined as the worse of two cases. For case one, the simultaneous design actions are axial load P, moment about *x* axis M_x, and moment about *y* axis M_y, where:

$$P = P^T + 0.3P^L$$
$$M_x = M_x^T + 0.3M_x^L \; .$$
$$M_y = M_y^T + 0.3M_y^L$$

For case two, the simultaneous design actions are calculated as:

$$P = 0.3P^T + P^L$$
$$M_x = 0.3M_x^T + M_x^L$$
$$M_y = 0.3M_y^T + M_y^L$$

Where either the LDP or the NDP is used, the effects of multidirectional loading may be accounted for directly by applying appropriate bidirectional ground motions and directly monitoring maximum responses. Alternatively, where the LDP is used, either the 30% rule or the square root sum of squares (SRSS) rule may be used. If the objective is to find the maximum response to multicomponent ground motions for a single response quantity, a preferred approach is the SRSS combination rule. On the other hand, if the objective is to locate the response to multicomponent ground motion on a failure surface (such as a P-M_x-M_y interaction diagram for a column, as described previously), then the 30% combination rule is preferred. The complete quadratic combination (CQC) (Wilson, et al., 1981) method is not appropriate for combining actions from multidirectional ground motions.

Where the NSP is used, the 30% combination rule may be interpreted as recommending that components be checked for forces and deformations associated with the structure being displaced to 100% of the target displacement in one direction and simultaneously to 30% of the target displacement in the orthogonal direction. Limitations of currently available nonlinear analysis computer software may prevent the engineer from following this procedure explicitly. Furthermore, biaxial deformation acceptance criteria are generally lacking in Chapters 5 through 8. As an alternative, the engineer is encouraged to consider indirectly the effects of biaxial loading in implementing the evaluation. In particular, it may be important to recognize the effects of bidirectional loading on forces developed in force-controlled components. Figure C3-5 illustrates one such case, where the axial load in a corner column under bidirectional lateral loading is equal to nearly twice the axial load under unidirectional loading.

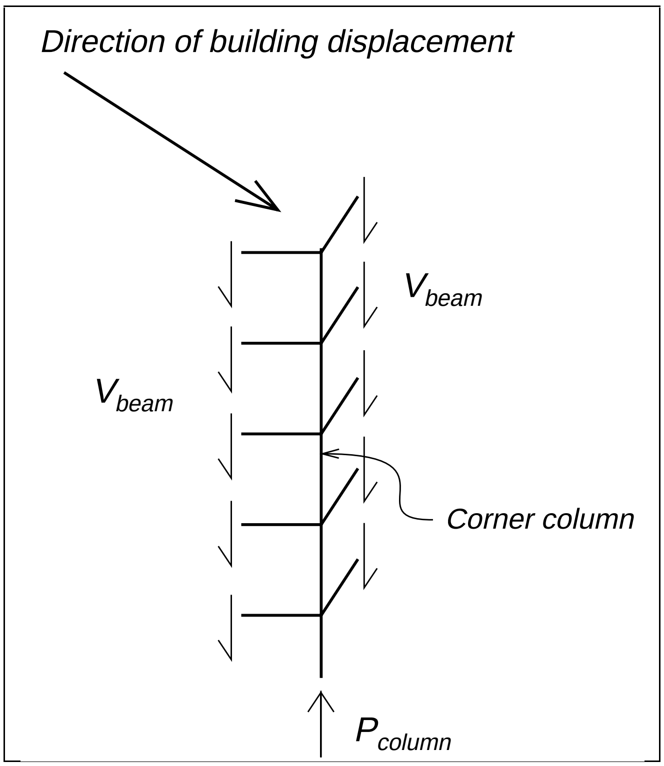


Figure C3-5 Multidirectional Effects on Calculation of Design Actions

The rule for combining multidirectional earthquake shaking effects assumes minimal correlation between ground motion components. This combination rule may be nonconservative in the near field for earthquakes with magnitudes greater than 6.5. As such, the engineer should use this rule with caution.

Vertical accelerations in past earthquakes are suspected of causing damage to long-span structures and to horizontal cantilevers. The *Guidelines* recommend that effects of vertical accelerations be considered for these structures as part of the rehabilitation design. The vertical ground shaking is defined according to Section 2.6.1.5. The procedure to be used for the analysis is the same as that described for horizontal excitations in the various portions of the *Guidelines*. Acceptance criteria are in the relevant Chapters 5 through 8. One caution with regard to vertical accelerations is that they add to gravity loads in one direction and subtract from them in the opposite direction. The possibility that response will be skewed in one direction or the other, and that plastic deformations may accumulate in the direction of gravity loads, should be considered.

C3.2.8 Component Gravity Loads and Load Combinations

In general, both the load combinations represented by Equations 3-2 and 3-3 should be analyzed as part of the Systematic Rehabilitation Method. For the linear procedures, superposition principles can be used to develop design actions for the different load cases—a relatively simple process involving algebraic manipulation of results obtained from lateral and gravity load analyses. For the nonlinear procedures, superposition cannot, in general, be used, so that application of both Equations 3-2 and 3-3 requires two completely separate analyses, a process that may require considerable effort. It may be possible in certain cases to determine by inspection that one of the two gravity load combinations will not control the design.

The load case represented by Equation 3-3 is critical for cases where earthquake effects result in actions that are opposite those due to gravity loads. Although these cases are seemingly ubiquitous and noncritical in any structure, they are considered especially critical for force-controlled components or actions. Examples include tension forces in corner columns and in vertical chords of shear walls and braced frames.

The gravity load combinations set forth in Equations 3-2 and 3-3 for use in seismic evaluation differ from those presented in regulations for new construction. The resulting member actions are smaller than those calculated for corresponding new construction. The gravity load combinations were modified on the following bases: (1) the *Guidelines* require on-site evaluation of dead loads and permanent live loads, thereby reducing the likely scatter in the magnitudes of the gravity loads assumed for analysis; (2) the building is known to have existed under the action of loads and is known to be adequate for those loads; (3) the Performance Levels identified in the *Guidelines* are not necessarily the same as those implicit in the design basis for new buildings; and (4) the *Guidelines* use different definitions of materials and component strengths from those used for the design of new buildings.

The component loads and load combinations presented in Equations 3-2 and 3-3 are intended for seismic evaluation only. Component loads and load combinations for gravity and wind load checking are identified in other regulations; the component loads and load combinations set forth in Section 3.2.8 must not be used for gravity load evaluation.

The minimum live load specification equal to 0.25 of the unreduced design live load is a traditionally applied value used in design to represent the likely live load acting in a structure. Where the load is likely to be larger, use this larger load.

C3.2.9 Verification of Design Assumptions

The goals of this section are (1) to require the engineer to check design actions and associated strengths at all locations within the component rather than just at the end points or nodes used to define the component in the mathematical model, and (2) to ensure that the post-earthquake residual gravity-load capacity of a component is not substantially compromised due to redistribution of moments resulting from earthquake shaking. The first goal addresses component response during earthquake shaking; the second addresses component response following earthquake shaking. High gravity-load actions, identified using the 50% rule presented in the *Guidelines*, will increase the likelihood that these items will be critical for design.

If component actions due to gravity loads are much smaller than the expected component strengths at all locations, it is neither probable that flexural hinges will form between the component ends nor is it likely that flexural hinges will form between the component ends due to small increases in gravity loads following an earthquake. The 50% rule presented in the *Guidelines* is based on the judgment of the writers. Note that this comparison of component actions and strengths is based on the load combinations set forth in Equations 3-2 and 3-3, and not on load combinations set forth in other regulations for gravity load checking. For components with gravity load actions exceeding the 50% rule, verification of Item 1 is mandatory, and checking for Item 2 is recommended.

Hinge Formation at Component Ends. For beams evaluated or designed using the linear procedures, inelastic flexural action normally should be restricted to the beam ends. This is because linear procedures can lead to nonconservative results, and may completely misrepresent actual behavior, when flexural yielding occurs along the span length (that is, between the component ends). To check for flexural yielding along the span length, construct a free-body diagram of the beam loaded at its ends with the expected moment strengths Q_{CE} and along its length with the gravity loads given by Equations 3-2 and 3-3. (See Figure C3-6 for details.) The moment diagram can then be constructed from equilibrium principles. The moments along the length of the beam can then be compared with the strengths at all locations. For this purpose, the strength may be calculated as an expected strength rather than a lower-bound strength. Where this comparison indicates that flexural strength may be reached at locations more than one beam depth from the beam ends, either the beam should be rehabilitated to prevent inelastic action along the length, or the design should be based on one of the nonlinear procedures (Sections 3.3.3 or 3.3.4).

For beams evaluated or designed using the nonlinear procedures, it is required that inelastic flexural actions be restricted to nodes that define the beam in the mathematical model. It is recommended that nodes be placed at the locations of significant mass and/or reactions (likely corresponding to the locations of maximum gravity moments). To check for flexural yielding along the span length, construct a free-body diagram of the beam loaded at its ends with the moments calculated by nonlinear procedures and along its length with the gravity loads given by Equations 3-2 and 3-3. This is similar to that shown in Figure C3-6, except that calculated moments from nonlinear procedures replace the expected strengths calculated by cross-section analysis. The moment diagram can then be constructed from equilibrium principles. The moments along the length of the beam can then be compared with the strengths at all locations. For this purpose, the strength may be calculated as an expected strength rather than a lower-bound strength. Where this comparison indicates that flexural strength may be reached at locations other than nodes in the mathematical model, the mathematical model should be refined and the building reanalyzed.

Post-Earthquake Residual Gravity Load Capacity. Earthquake shaking can substantially affect the magnitude of gravity load actions in a building frame. Consider a steel beam in a simple building frame shown in Figure C3-7. Assume that the beam moment strength is constant along its length. The gravity moment diagram is shown in Figure C3-7a. At the beam ends the gravity moment is equal to 50% of the beam strength, while at the mid-span of the beam the gravity moment is equal to 75% of the beam strength. (For this beam the total static moment due gravity loads is equal to 125% of the beam strength.) Evaluation of this beam for gravity moment strength would find this beam adequate at all locations. Due to moment redistribution within the frame, it is plausible that the post-earthquake moment diagram due to gravity loads could be that given by Figure C3-7b. At the beam ends the gravity moment is equal to 25% of the beam strength. At the mid-span of the beam the gravity moment is equal to 100% of the beam strength. Although evaluation of this beam for gravity moment strength would find this beam adequate at all locations, any increase in gravity loads would produce flexural hinging at the mid-span of the beam. If this beam is not designed for ductile behavior at this location, local failure of the beam may ensue. (Note that the moment diagrams presented in Figures C3-6 and C3-7 are somewhat arbitrary, and are intended to illustrate the issues identified above.)

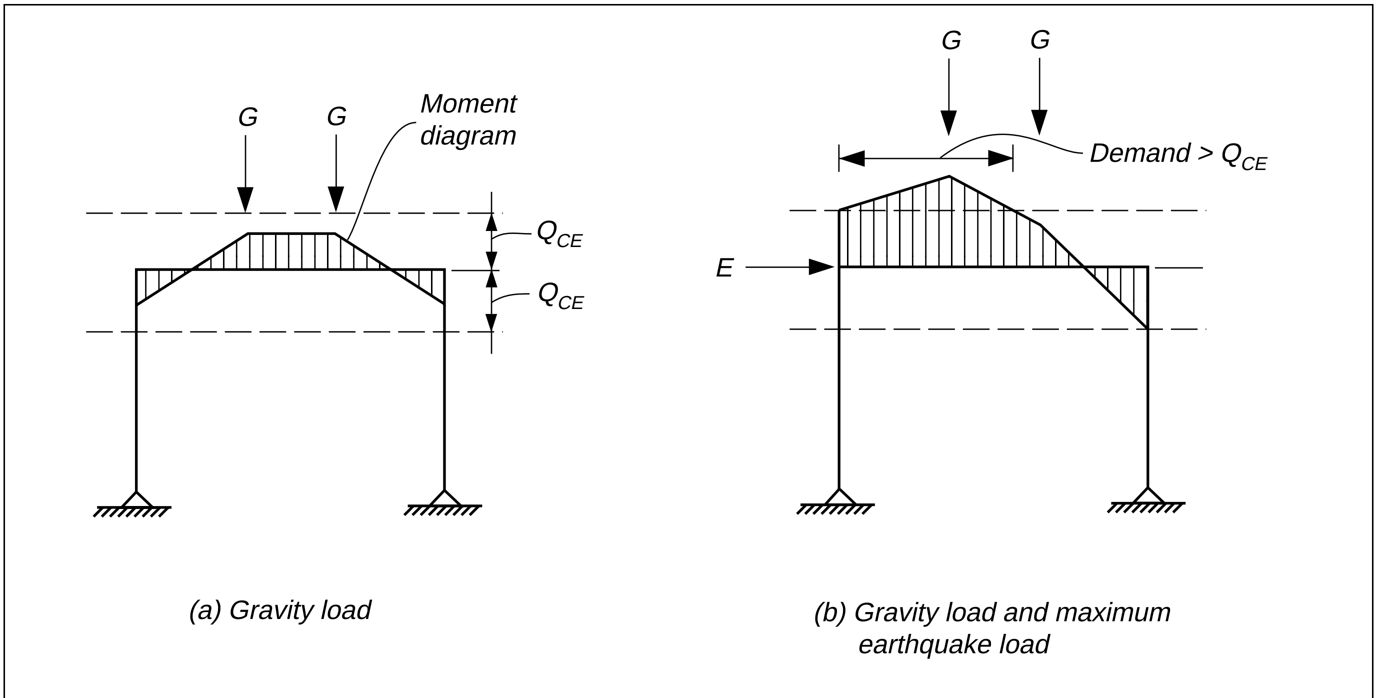


Figure C3-6 Hinge Formation Along Beam Span

For beams designed using linear procedures, a very conservative method for checking post-earthquake residual gravity-load capacity is to load the beam ends with zero moment and the beam along its length with the gravity loads given by Equation 3-2 or 3-3.

For beams designed using the NSP, one method for checking post-earthquake residual gravity-load capacity is to unload the frame (that is, load the frame with lateral forces equal and opposite to those corresponding to the target displacement, for a total of zero applied lateral load). Gravity loads should be applied through all stages of the analysis. For beams designed using the

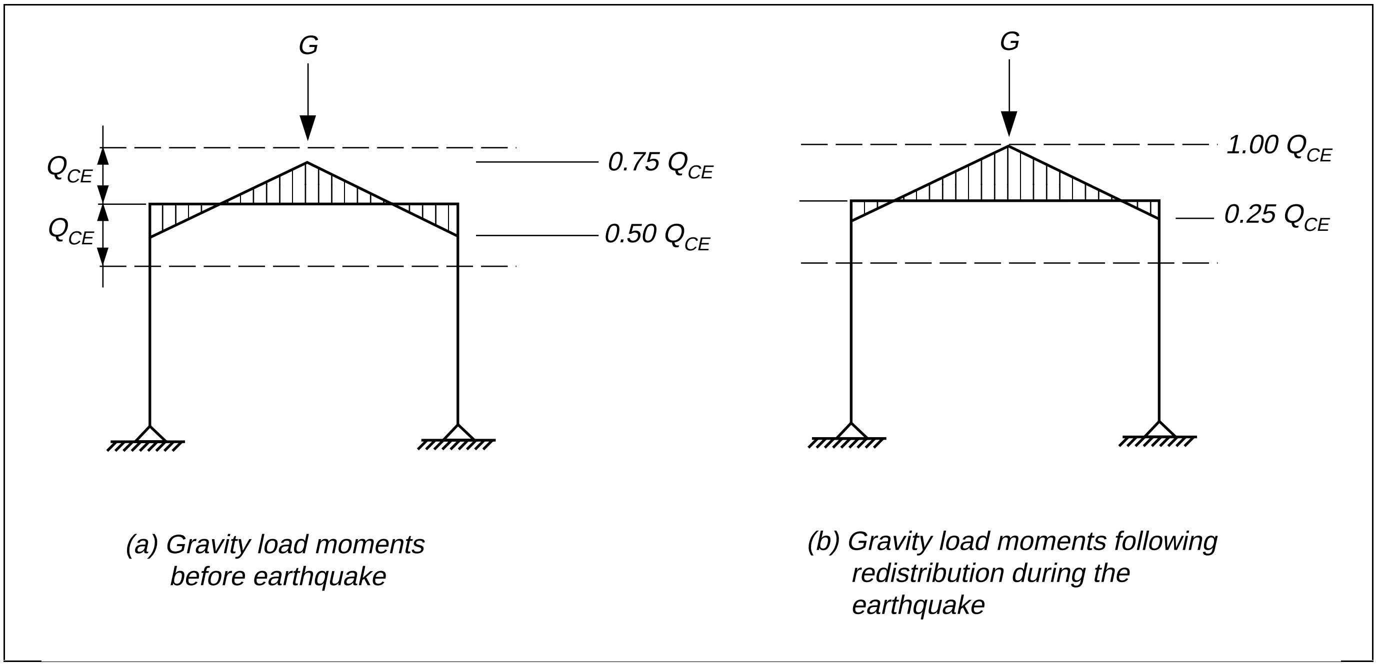


Figure C3-7 Post-Earthquake Residual Gravity Load Capacity

NDP, the effects of moment redistribution due to earthquake shaking can be directly evaluated by review of the gravity load actions at the end of the time-history analysis.

Rules for minimum residual gravity load capacity above that required by the load combinations set forth in Equations 3-2 and 3-3 are not provided because the residual capacity is likely a function of the Performance Level used for the design. The engineer should develop rules on a project-by-project basis. The reader is referred to Bertero (1996) for additional information.

C3.3 Analysis Procedures

The *Guidelines* present four specific Analysis Procedures. The writers recognize that variations on these procedures—and completely different procedures—are currently in use, and that these alternate procedures may be equally valid, and in some cases may provide added insight into the evaluation and design process. Some of these alternative procedures, described in this *Commentary*, may be considered to be acceptable alternatives to the four procedures presented in the *Guidelines*, although the engineer should verify that they are applicable to the particular conditions of the building and its Rehabilitation Objectives.

C3.3.1 Linear Static Procedure (LSP)

C3.3.1.1 Basis of the Procedure

According to the LSP, static lateral forces are applied to the structure to obtain design displacements and forces. Two important assumptions are involved. First, it is implied that an adequate measure of the design actions can be obtained using a static analysis, even though it is recognized that earthquake response is dynamic. Section 2.9 provides criteria to determine when this simplification is unsatisfactory, and when dynamic analysis is required as an alternative. Second, it is implied that an adequate measure of the design actions can be obtained using a linearly-elastic model, even though nonlinear response to strong ground shaking may be anticipated. Section 2.9 provides criteria to determine when this assumption is unsatisfactory, and when nonlinear procedures are required as an alternative. In general, the writers of the *Guidelines* recognize that improved estimates of response quantities can be obtained using dynamic analysis, and further improvements can be obtained using nonlinear response analysis where nonlinear response is anticipated. Use of these approaches is encouraged.

The *Guidelines* adopt a widely-accepted philosophy that permits nonlinear response of a building when subjected to a ground motion that is representative of the design earthquake loading. For some structures, total allowable deformations may be several times higher than yield deformations. The primary measure of the performance of a "yielding" building lies in the level of deformation imposed on individual components and elements, compared with their reliable deformation capacities. Stress, force, and moment amplitudes are of secondary importance for ductile components and elements, as it is accepted that ductile materials will reach their stress capacities, and be deformed beyond the yield point. Stress, force, and moment amplitudes may be of primary importance for brittle (force-controlled) components and elements that may fail when force demands reach force capacities.

Ideally, the evaluation of a "yielding" building should be carried out using nonlinear procedures that explicitly account for nonlinear deformations in yielding components. As an alternative, the *Guidelines* permit evaluation to be carried out using linear procedures. In a linear procedure, there is a direct relation between internal forces and internal deformations for any given loading pattern. Therefore, it is simpler when using linear procedures to express acceptability in terms of internal forces rather than internal deformations. This is the approach adopted with the LSP.

Figure C3-8 illustrates the intent of the LSP. The solid curve in the figure represents the backbone load-displacement relation of the building as it is deformed to the maximum displacement δ_{max} by the design earthquake loading. The LSP represents the building by a linearly-elastic stiffness that approximately corresponds to the effective lateral load stiffness for loading below the effective yield point of the building. To achieve the maximum displacement, δ_{max}, using the linearly-elastic model, the model must be loaded by a pseudo lateral load V defined by Equation 3-6. This pseudo lateral load may be several times larger than the base shear capacity of the building, and corresponding internal component forces may similarly be several times the component force capacities. The acceptance procedures of Section 3.4 take this aspect into account, allowing component overstress levels that vary with the expected nonlinear deformation capacity of the individual component.

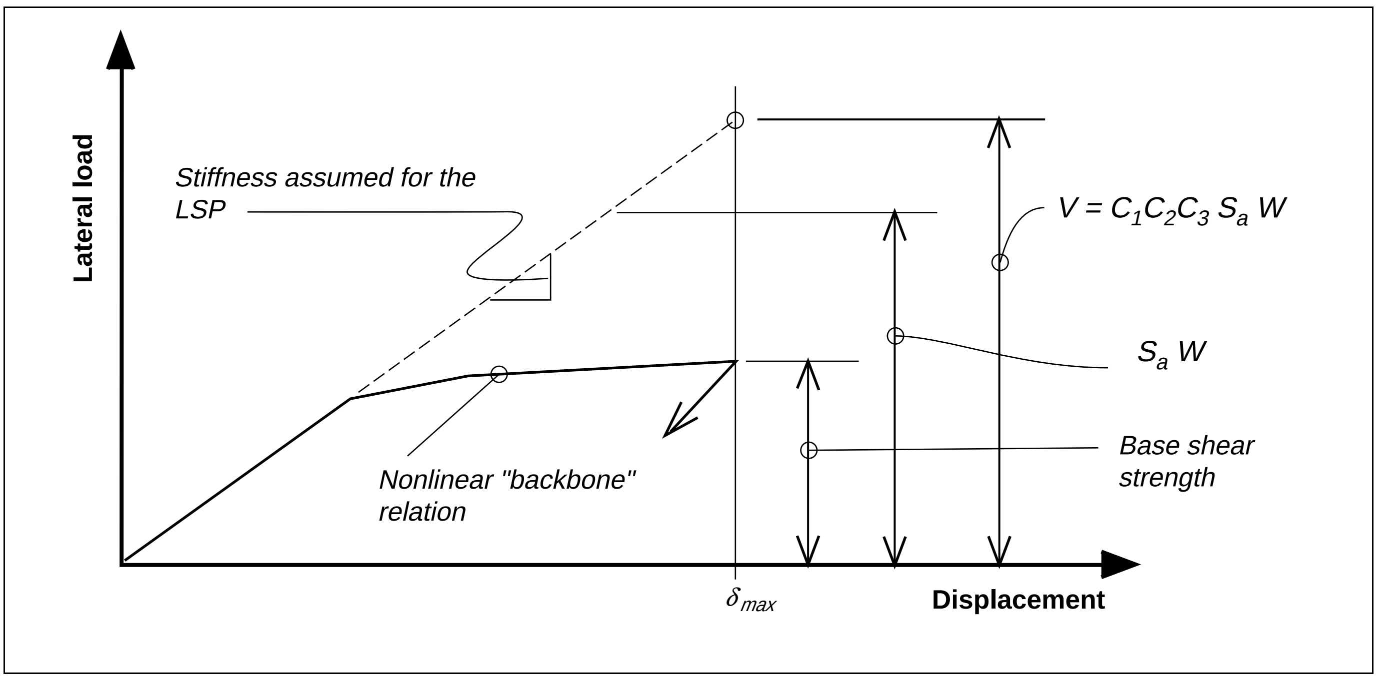


Figure C3-8 Basis for the Linear Static Procedure

C3.3.1.2 Modeling and Analysis Considerations

The following commentary contains essential details of the LSP.

A. Period Determination

In accordance with the basis of the LSP as illustrated in Figure C3-8, the period used for design should correspond to the fundamental translational period of the building responding in the linearly-elastic range. Other definitions of period—for example, secant values—are not generally appropriate for the LSP.

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. Many programs available from commercial software providers are capable of determining the period specified in Method 1.

Method 2 provides an approximate value of the fundamental translational period for use in design. The expressions for period are the same as those that appear in the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* (BSSC, 1995). Method 2 may be most suitable for small buildings for which detailed mathematical models are not developed. Method 2 may also be useful to check that periods calculated by Method 1 are reasonable. On average, actual measured periods, and those calculated according to Method 1, exceed those obtained by Method 2.

Method 3 applies only to one-story buildings with single span flexible diaphragms. Equation 3-5 is derived from an assumed first-mode shape for the building (Figure C3-2). The equation is not applicable to other buildings.

Periods obtained from the three different methods should not be expected to be the same, as each is based on a different set of approximating assumptions. Design forces and displacements in the LSP are intended to be obtained by applying a pseudo lateral load (Section 3.3.1.3A) to a mathematical model of the building. The most conservative design results will be obtained for the period that produces the maximum pseudo lateral load. Usually, this will be achieved by using a low estimate of the fundamental period, although for certain site-specific spectra the opposite will be the case. The engineer should investigate this possibility on a case-by-case basis.

The approximate formula, $T = 0.1N$, for the period T of steel or reinforced concrete moment frames of 12 stories or less ($N \leq 12$) is added here for historical completeness.

C3.3.1.3 Determination of Actions and Deformations

A. Pseudo Lateral Load

The pseudo lateral load is the sum of lateral inertial forces that must be applied to the linearly-elastic model of the building to produce displacements approximately equal to those the actual structure is expected to undergo during ground motion corresponding to the design earthquake loading. In Equation 3-6, the quantity $S_a W$ is the elastic spectral force associated with the design earthquake loading. When this force is applied to a linearly-elastic model of the structure, it produces deformations expected for the linearly-elastic structure subjected to the design earthquake loading. Coefficients C_1, C_2, and C_3 modify the elastic force levels for the purpose of correspondingly modifying the design deformations in the “yielding” structure. The effect of coefficient C_1 is illustrated in Figure C3-9. Note that the purpose of the coefficients is to modify the design displacements to be more representative of those expected for a “yielding” s$T = 0.1N$ ubjected to the design earthquake loading.

The anticipated live load in W is different from the Q_L of Section 3.2.8.

Note that reduction of base shear due to multimode effects has conservatively not been used in the LSP.

Further discussion on the coefficients in Equation 3-6 follows.

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 (Nassar and Krawinkler, 1991; Miranda and Bertero, 1994; Bonacci, 1989). The continuous curves in Figure C3-9 illustrate mean values of the coefficient C_1 as formulated by Miranda and Bertero (1994). In that figure, 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 values of C_1 given by the continuous curves in Figure C3-9. Where the quantity R is not defined, as permitted for the LSP, the coefficient C_1 may be read from the broken curve in Figure C3-9, which is a graphical representation of the expressions given in Section 3.3.1.3A.

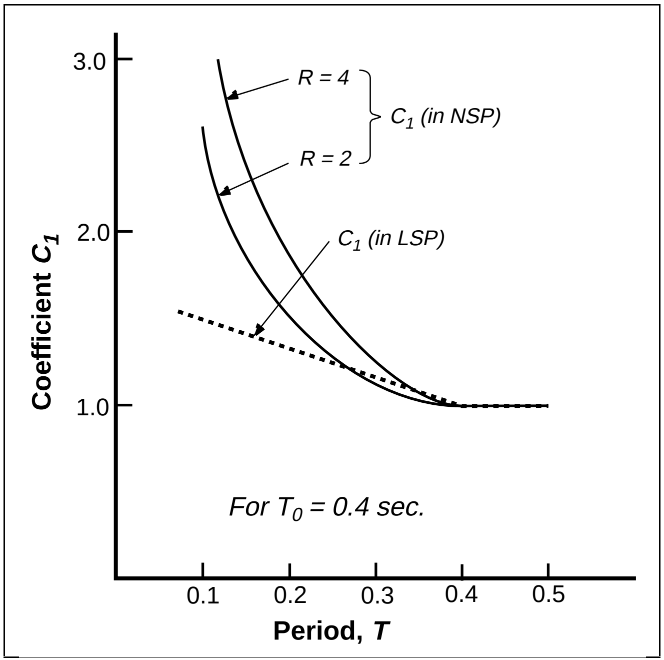


Figure C3-9 Relation between R and C_1

Note that the relations represented in Figure C3-9 are mean relations, and that considerable scatter exists about the mean (Miranda, 1991). For critical structures, the engineer should consider increasing the value of the coefficient C_1 to account approximately for the expected scatter.

Recent studies by Constantinou et al. (1996) 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. The above description of Coefficient C_1 is based on mean responses of inelastic single-degree-of-freedom (SDOF) systems with bilinear hysteresis models. If the hysteresis loops exhibit significant pinching or stiffness deterioration (Figure C6-22), the energy absorption and dissipation capacities decrease, and larger displacement excursions should be expected. At the time of this writing, only limited data are available to quantify this increase in displacement, but it is known that this effect is important for short-period, low-strength structures with very pinched hysteresis loops. Pinching is a manifestation of structural damage; the smaller the degree of nonlinear response, the smaller the degree of pinching. 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 Immediate Occupancy (little-to-no damage) than for Collapse Prevention (moderate-to-major damage). The period-dependence of this displacement modifier has been established by analysis; sample data comparing the displacement responses of a severely pinched SDOF system and a bilinear SDOF system are presented in Figure C3-10 (Krawinkler, 1994).

Framing systems whose components exhibit pinched hysteresis will likely experience strength degradation in severe earthquakes. This deterioration will further increase earthquake displacements. The values for C_2 given in Table 3-1 are intended to account for both stiffness degradation and strength deterioration, and are based on judgment at the time of this writing.

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. The equation given for coefficient C_3 for flexible buildings ($\theta > 0.1$), namely:

$$C_3 = 1 + \frac{5(\theta - 0.1)}{T} \qquad \text{(C3-2)}$$

is loosely based on the equation for coefficient C_3 presented for use with the NSP. Note that no measure of

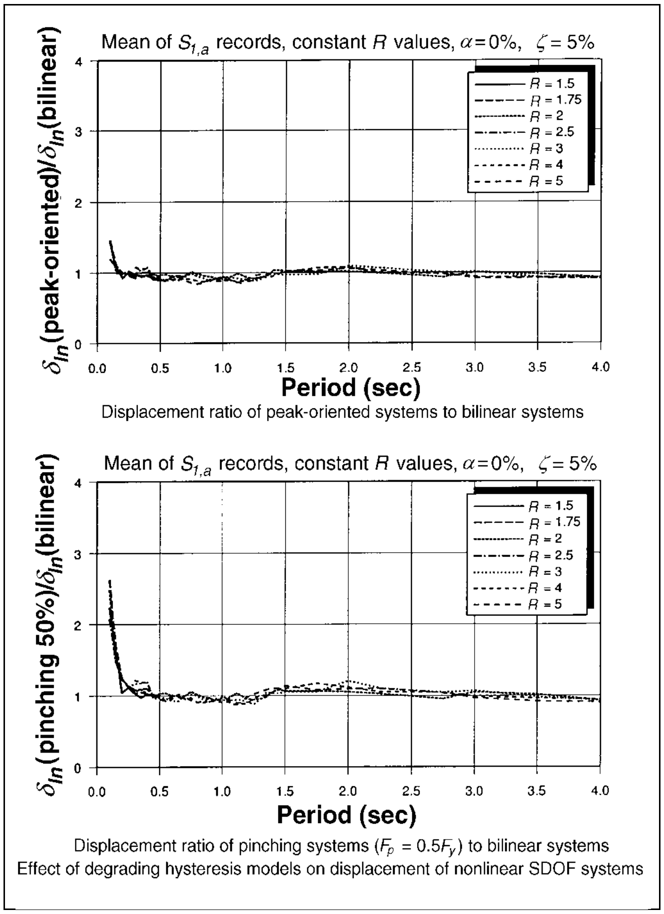


Figure C3-10 ***Increased Displacements Due to Pinched Hysteresis***

the degree of negative post-yield stiffness can be explicitly included in a linear procedure.

B. Vertical Distribution of Seismic Forces

The distribution of inertia forces over the height of a building during earthquake shaking varies continuously in a complex manner. Sample inertia force distributions are presented in Figure C3-11. Key to design is capturing the critical distribution(s) that will maximize design actions.

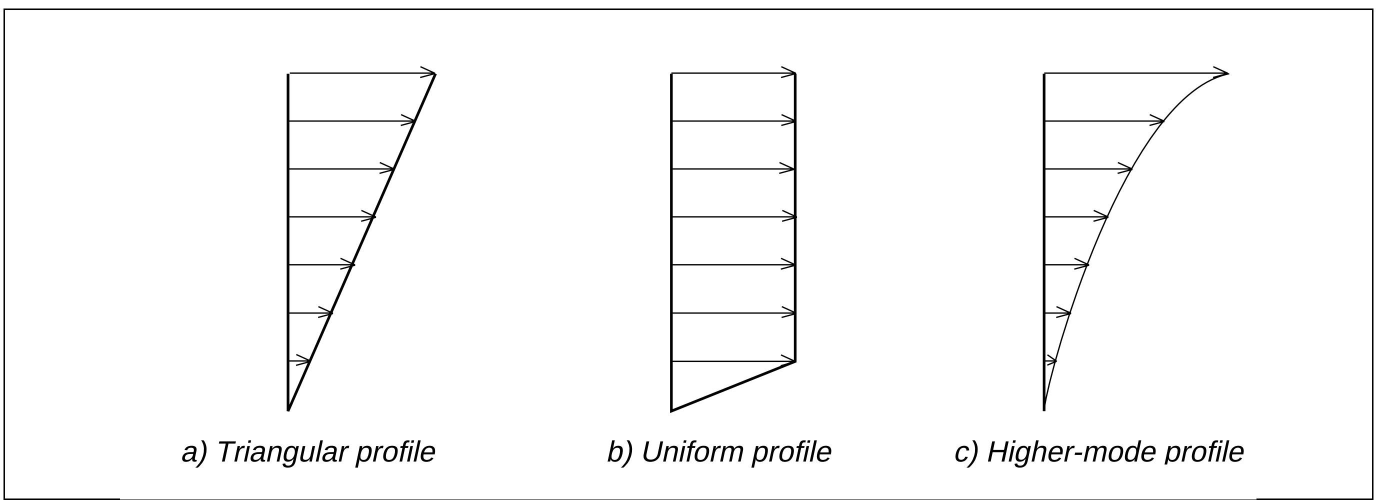


Figure C3-11 ***Sample Inertia Force Distributions***

If the building is responding in the linearly-elastic range, the distribution of inertia forces is a function of many factors, such as the frequency characteristics and amplitude of the earthquake shaking, and the modal frequencies and shapes of the building. If the building is responding in the nonlinear (inelastic) range, the distribution of inertia forces is further complicated by localized, and perhaps global, yielding in the building.

For analysis and design, simplified procedures are needed that will likely capture the worst-case distribution of inertia forces. The method for vertical distribution of seismic forces assumes linear response in the building; the method is virtually identical to that used in the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* (BSSC, 1995).

For short-period buildings ($T \le 0.5$ second), the vertical distribution of inertia forces assumes first-mode response only — approximated by setting k equal to 1.0. The resulting inertia force distribution is the inverted-triangular distribution that formed the basis of seismic design provisions for many years.

For long-period buildings ($T \ge 2.5$ seconds), higher-mode effects may substantially influence the distribution of inertia forces, producing higher relative accelerations in the upper levels of a building. Higher mode effects are introduced using a value of k greater than 1.0. The use of values of k greater than 1.0 has the effect of increasing both the story shear forces in the upper levels of a building, and the global overturning moment for a given base shear, by moving the seismic force resultant up toward the roof of the building. Note that increasing the ratio of moment to shear demand may not be conservative in the design of shear-critical elements such as reinforced-concrete structural walls.

C. Horizontal Distribution of Seismic Forces

The inertia forces F_x from Equation 3-7 arise from acceleration of the individual masses attributed to floor level x. Therefore, this section specifies that the forces F_x be distributed across the level in proportion to the mass distribution of the floor.

The total story shear force, overturning moment, and horizontal torsional moment are to be determined from statics considering the application of the inertia forces to the levels above the story being considered. The distribution of these to individual resisting elements is to be determined by analysis, considering equilibrium and compatibility among the vertical and horizontal elements of the structural system.

D. Floor Diaphragms

The floor diaphragm is a key component of the seismic load path in a building. Diaphragms serve to transfer seismic-induced inertia forces to vertical members of the seismic framing system.

The connection between a diaphragm and the associated vertical seismic framing element is a critical element in the seismic load path. Buildings have failed during earthquake shaking due to a lack of strength in such connections. Diaphragm connections should be designed to have sufficient strength to transfer the maximum calculated diaphragm forces to the vertical framing elements.

The seismic loading in the plane of a diaphragm includes the distributed inertia force equal to the response acceleration at the level of the diaphragm multiplied by its distributed mass. Equation 3-9 provides an approximate method for determining the seismic forces for design. Coefficients C_1, C_2, and C_3 are removed from the diaphragm inertia force calculation because they are displacement multipliers (on vertical lateral-force-resisting elements) and not force multipliers. The diaphragm must also be designed to transfer the concentrated shear forces from vertical seismic framing above the diaphragm to vertical seismic framing below the diaphragm wherever there are changes in the stiffness or plan location of such framing.

C3.3.2 Linear Dynamic Procedure (LDP)

C3.3.2.1 Basis of the Procedure

The LDP uses the same linearly-elastic structural model as does the LSP. Because the LDP represents dynamic response characteristics directly, it may provide greater insight into structural response than does the Linear Static Procedure. However, as with the LSP, it does not explicitly account for effects of nonlinear response. The writers of the *Guidelines* recognize that improved estimates of response for use in design may be achieved in many cases by using nonlinear response analysis, and encourage the use of the nonlinear procedures where appropriate.

Section C3.3.1.1 provides additional discussion of the basis of the linear procedures.

C3.3.2.2 Modeling and Analysis Considerations

A. General

For the LDP, the results of linear dynamic analysis are not scaled to the base shear from the LSP. Thus, the equivalent base shear in the LDP is expected to be lower than the value obtained from the LSP, due to higher-mode participation effects.

B. Ground-Motion Characterization

The Response Spectrum Method uses either the response spectrum as defined in Section 2.6.1.5 or a site-specific response spectrum as defined in Section 2.6.2.1. The Time-History Method uses ground-motion time histories as defined in Section 2.6.2.2.

C. Response Spectrum Method

The Response Spectrum Method requires dynamic analysis of a mathematical model of a building to establish modal frequencies and mode shapes. Using standard mathematical procedures (Clough and Penzien, 1993) and a response spectrum corresponding to the damping in the building, the modal frequencies and shapes are used to establish spectral demands. The spectral demands are then used to calculate member forces, displacements, story forces, story shears, and base reactions for each mode of response considered. These forces and displacements are then combined using an established rule to calculate total response quantities.

The *Guidelines* require that a sufficient number of modes of response be considered in the analysis so as to capture at least 90% of the building mass in each of the building's principal horizontal directions. The 90% rule is the industry standard and has been used in the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* and the *Uniform Building Code* for many years.

Two modal combination rules are identified in the *Guidelines*. The first, the square root sum of squares (SRSS) rule (Clough and Penzien, 1993), has been widely used for more than 30 years. The second, the complete quadratic combination (CQC) rule (Wilson et al., 1981) has seen much use since the mid-1980s. The reader is referred to the literature for additional information.

Requirements for simultaneous, multidirectional seismic excitation are given in Section 3.2.7.

D. Time-History Method

The Time-History Method involves a step-by-step analysis of the mathematical model of a building using discretized earthquake time histories as base motion inputs. Torsional effects shall be captured explicitly using the Time-History Method. Time-History Analysis of two- and three-dimensional mathematical models is permitted by the *Guidelines*. Three-dimensional mathematical models may be analyzed using either ground-motion time histories applied independently along each principal horizontal axis, or orthogonal ground-motion time histories (constituting a pair of time histories) applied simultaneously.

Earthquake ground-motion time histories, and pairs of such time histories, shall be established in accordance with the requirements of Section 2.6.2.2. Correlation between ground-motion time histories that constitute a pair of ground-motion time histories shall be consistent with the source mechanism and assumed epicentral distance to the building site.

Multidirectional excitation effects can be considered by either (1) simultaneously applying pairs of ground-motion time histories to the mathematical model (with appropriate phasing of the ground motion components), or (2) following the procedures set forth in Section 3.2.7.

C3.3.2.3 Determination of Actions and Deformations

A. Modification of Demands

The actions and deformations calculated using either the Response Spectrum or Time-History Methods shall be factored by the coefficients C_1, C_2, and C_3 developed for the LSP. For information on these coefficients, the reader is referred to the commentary above.

B. Floor Diaphragms

The reader is referred to the commentary on Section 3.3.1.3D for pertinent information. The 85% rule of Section 3.3.2.3B is intended to offer the engineer an incentive to use the LDP; the value of 85% is arbitrary.

C3.3.3 Nonlinear Static Procedure (NSP)

C3.3.3.1 Basis of the Procedure

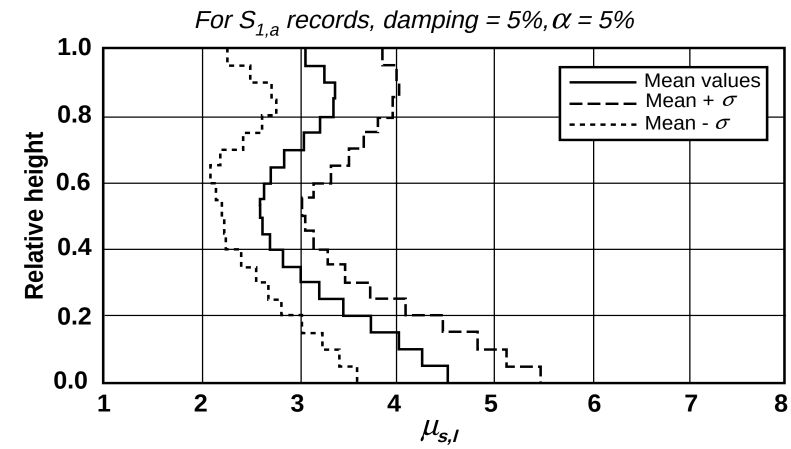
According to the NSP, static lateral forces are applied incrementally to a mathematical model of the structure until a target displacement is exceeded. Building deformations and internal forces are monitored continuously as the model is displaced laterally. The procedure parallels that of the LSP, but with two very important differences. First, in the NSP the nonlinear load-deformation behavior of individual components and elements is modeled directly in the mathematical model. Second, in the NSP the earthquake effect is defined in terms of a target displacement rather than a pseudo lateral load. The NSP requires that the behavior of components in which internal forces reach strengths be described by multilinear (in the simplest case, bilinear) force-deformation models with well-defined strength and deformation capacities. The design force and deformation demands in each component are calculated for the design earthquake displacement(s), and acceptability is evaluated by comparing the computed force and deformation demands with available capacities. Capacities for different Performance Levels are provided in Chapters 4 through 9, and 11. Although the NSP requires considerably more analysis effort than does the LSP, it usually provides improved insight into the expected nonlinear behavior of the structure, and therefore usually provides better design information.

The NSP uses ground motion information derived from smoothed design spectra, thereby avoiding the narrow valleys and peaks that often characterize individual ground motion records, and consequently providing a more robust design loading. The procedure's shortcoming is its inability to represent realistically all changes in nonlinear dynamic response characteristics of the structure caused by cyclic stiffness degradation and strength redistribution. This shortcoming may lead to deficient estimates of local force and plastic deformation demands, particularly when higher modes gain in importance as yielding progresses in the structure. Thus, when higher modes are important, preference should be given to the NDP. Chapter 2 presents restrictions on the use of the NSP based on considerations of the higher-mode dynamic effects.

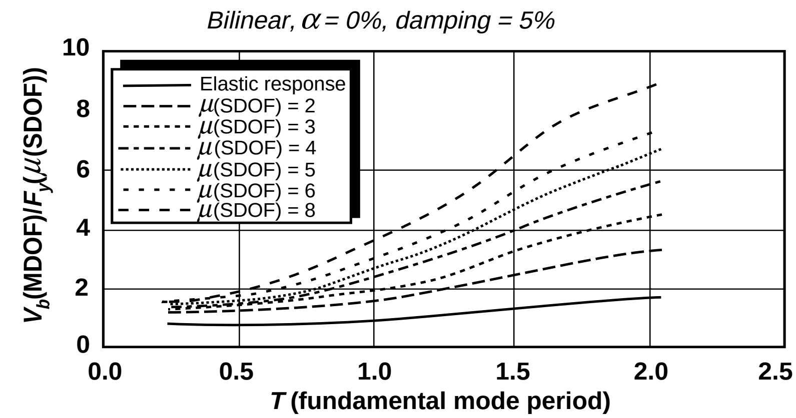
It is possible, when evaluating a building having multiple failure modes, that the NSP will identify only one of these modes, effectively overlooking the other modes. An example is a multistory building with weak columns in multiple floors. Analysis by the NSP using a single lateral load distribution is likely to identify vulnerability of only a single floor, especially if there is insignificant strain-hardening associated with column failure. The other floors may be equally or more vulnerable to collapse under dynamic loading for which the lateral inertia force distribution is continually changing. The NSP requires that at least two lateral load distributions be considered in the evaluation, in part to identify the potential for multiple failure modes. The engineer needs to be generally aware that multiple failure modes may be possible, and needs to implement rehabilitation strategies that mitigate the vulnerabilities in each of these modes.

Figure C3-12 illustrates some of the limitations of the NSP. The top diagram shows the mean and mean $\pm \sigma$ values of the story ductility demands for a 1.2-second frame structure subjected to a set of 15 ground motion records (Seneviratna, 1995). In this structure the strength of each story is tuned such that simultaneous yielding will occur in each story under the 1994 UBC seismic load pattern. Thus, if this load pattern is applied in an NSP, equal story ductility demands will be predicted in every story. The dynamic analysis results demonstrate that this is not the case and that significant variations of demands over the height have to be expected. These variations are caused by higher mode effects and are not present for structures whose response is governed by the fundamental mode. To some extent the importance of higher mode effects can be captured by the LDP, which is the reason why such an analysis should be performed to supplement the NSP when higher mode effects become important. Section 2.9.2.1 identifies the conditions under which an LDP is required.

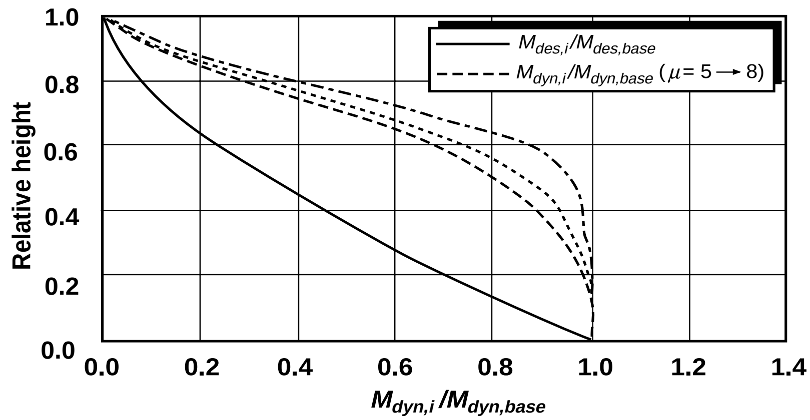
An example that demonstrates other potential problems with the NSP is that of multistory wall structures modeled by a single shear wall. In these wall structures it is assumed that the bending strength of the wall is constant over the height, and that the shear strength and stiffness are large, so that the behavior of the wall is controlled by bending. It is also assumed that no strain hardening exists once a plastic hinge has formed in the wall. The NSP will predict hinging at the base of the wall for all rational load patterns. A mechanism exists once this single plastic hinge has formed; the wall will rotate around its base, and the lateral loads can no longer be increased. Thus, the NSP will not permit propagation of plastic hinging to other stories and will predict a base shear demand that corresponds to the sum



Variation of story ductility demand over height



Amplification of base shear demand for wall structures



Story moment envelopes

Figure C3-12 Limitations of the NSP Illustrated with Story Ductility Demand, Amplification of Base Shear, and Moment Envelopes

of lateral loads needed to create the plastic hinge at the base.

Nonlinear dynamic Time-History Analysis gives very different results (Seneviratna, 1995). Higher mode effects significantly amplify the story shear forces that can be generated in the wall once a plastic hinge has formed at the base. This is illustrated in the middle diagram of Figure C3-12, which shows mean values of base shear amplification obtained by subjecting multistory wall structures to 15 ground motion records. The amplification depends on the period (number of stories) of the wall structure and on the wall bending strength (represented by μ[SDOF], the ductility ratio of the equivalent SDOF system). The diagram shows that the amplification of base shear demands may be as high as 5 for wall structures with reasonable bending strength (μ(SDOF) ≤ 4). This amplification implies that the base shear demand may be much higher than the base shear obtained from the lateral loads that cause flexural hinging at the base of the structure. Thus, wall shear failure may occur even though the NSP indicates flexural hinging at the base.

Nonlinear dynamic Time-History Analysis also shows that flexural hinging is not necessarily limited to the first story. It may propagate into other stories to an extent that depends on the period and flexural strength of the structure. This is illustrated in the story moment envelopes presented in the bottom diagram of Figure C3-12 for a wall structure with a period of 1.2 seconds. The moment envelope obtained from dynamic analyses is very different from that obtained from a code type load pattern (solid line).

No static analysis, whether linear or nonlinear, could have predicted this behavior. This example shows that additional measures need to be taken in some cases to allow a realistic performance assessment. Such measures need to be derived from the NDP and need to be formalized to the extent that they can be incorporated systematically in both the LSP and the NSP.

The user needs to be aware that the NSP in its present format has been based and tested on ground motions whose effects on structures can be represented reasonably by the smoothed response spectra given in Section 2.6.1 for soil classes A, B, C, and D. The prediction of the target displacement (Equation 3-11) is expected to be on the high side for soil class E. The NSP has not been tested on site-specific spectra or on near-field ground motions characterized by large displacement pulses. Moreover, the approximate modification factors contained in Equation 3-11 are calibrated for structures with a strength ratio *R* of about 5 or less. The modification factors may have to be increased for structures with a larger strength ratio.

C3.3.3.2 Modeling and Analysis Considerations

A. General

The general procedure for execution of the NSP is as follows.

1. An elastic structural model is developed that includes all new and old components that have significant contributions to the weight, strength, stiffness, and/or stability of the structure and whose behavior is important in satisfying the desired level of seismic performance. The structure is loaded with gravity loads in the same load combination(s) as used in the linear procedures before proceeding with the application of lateral loads.

2. The structure is subjected to a set of lateral loads, using one of the load patterns (distributions) described in the *Guidelines*. At least two analyses with different load patterns should be performed in each principal direction.

3. The intensity of the lateral load is increased until the weakest component reaches a deformation at which its stiffness changes significantly (usually the yield load or member strength). The stiffness properties of this “yielded” component in the structural model are modified to reflect post-yield behavior, and the modified structure is subjected to an increase in lateral loads (load control) or displacements (displacement control), using the same shape of the lateral load distribution or an updated shape as permitted in the *Guidelines*. Modification of component behavior may be in one of the following forms:

   a. Placing a hinge where a flexural element has reached its bending strength; this may be at the end of a beam, column, or base of a shear wall

   b. Eliminating the shear stiffness of a shear wall that has reached its shear strength in a particular story

c. Eliminating a bracing element that has buckled and whose post-buckling strength decreases at a rapid rate

d. Modifying stiffness properties if an element is capable of carrying more loads with a reduced stiffness

4. Step 3 is repeated as more and more components reach their strength. Note that although the intensity of loading is gradually increasing, the load pattern usually remains the same for all stages of the “yielded” structure, unless the user decides on the application of an adaptive load pattern (Bracci et al., 1995). At each stage, internal forces and elastic and plastic deformations of all components are calculated.

5. The forces and deformations from all previous loading stages are accumulated to obtain the total forces and deformations (elastic and plastic) of all components at all loading stages.

6. The loading process is continued until unacceptable performance is detected or a roof displacement is obtained that is larger than the maximum displacement expected in the design earthquake at the control node.

   Note: Steps 3 through 6 can be performed systematically with a nonlinear computer analysis program using an event-by-event strategy or an incremental analysis with predetermined displacement increments in which iterations are performed to balance internal forces.

7. The displacement of the control node versus first story (base) shear at various loading stages is plotted as a representative nonlinear response diagram of the structure. The changes in slope of this curve are indicative of the yielding of various components.

8. The control node displacement versus base shear curve is used to estimate the target displacement by means of Equation 3-11. Note that this step may require iteration if the yield strength and stiffnesses of the simplified bilinear relation are sensitive to the target displacement.

9. Once the target displacement is known, the accumulated forces and deformations at this displacement of the control node should be used to evaluate the performance of components and elements.

   a. For deformation-controlled actions (e.g., flexure in beams), the deformation demands are compared with the maximum permissible values given in Chapters 5 through 8.

   b. For force-controlled actions (e.g., shear in beams), the strength capacity is compared with the force demand. Capacities are given in Chapters 5 through 8.

10. If either (a) the force demand in force-controlled actions, components, or elements, or (b) the deformation demand in deformation-controlled actions, components, or elements, exceeds permissible values, then the action, component, or element is deemed to violate the performance criterion.

Asymmetry of a building in the direction of lateral loading will affect the force and deformation demands in individual components. Asymmetric elements and components in a building, such as reinforced concrete shear walls with T- or L-shaped cross section, have force and deformation capacities that may vary substantially for loading in opposite directions. Accordingly, it is necessary to perform two nonlinear procedures along each axis of the building with loads applied in the positive and negative directions, unless the building is symmetric in the direction of lateral loads or the effects of asymmetry can be evaluated with confidence through judgment or auxiliary calculations.

The recommendation to carry out the analysis to at least 150% of the target displacement is meant to encourage the engineer to investigate likely building performance under extreme load conditions that exceed the design values. The engineer should recognize that the target displacement represents a mean displacement value for the design earthquake loading, and that there is considerable scatter about the mean. Estimates of the target displacement may be unconservative for buildings with low strength compared with the elastic spectral demands. Although data are lacking at the time of this writing, it is expected that 150% of the target displacement is approximately a mean plus one standard deviation displacement value for buildings with a lateral strength in excess of 25% of the elastic spectral strength.

As noted in Step 1 of the NSP, gravity loads need to be applied as initial conditions to the nonlinear procedure, and need to be maintained throughout the analysis. This is because superposition rules applicable to linear procedures do not, in general, apply to nonlinear procedures, and because the gravity loads may importantly influence the development of nonlinear response. The gravity-load combinations are the same as in the linear procedures. As noted previously, the use of more than one gravity-load combination will greatly increase the analysis effort in the NSP. It may be possible by inspection to determine that one of the two specified combinations will not be critical.

The mathematical model should be developed to be capable of identifying nonlinear action that may occur either at the component ends or along the length of the component. For example, a beam may develop a flexural plastic hinge along the span (rather than at the ends only), especially if the spans are long or the gravity loads are relatively high. In such cases, nodes should be inserted in the span of the beam to capture possible flexural yielding between the ends of the beam. This condition is illustrated in Figure C3-13 for a simple portal frame for increasing levels of earthquake load, namely, zero (part a) to E' (part b) to E'' (part c).

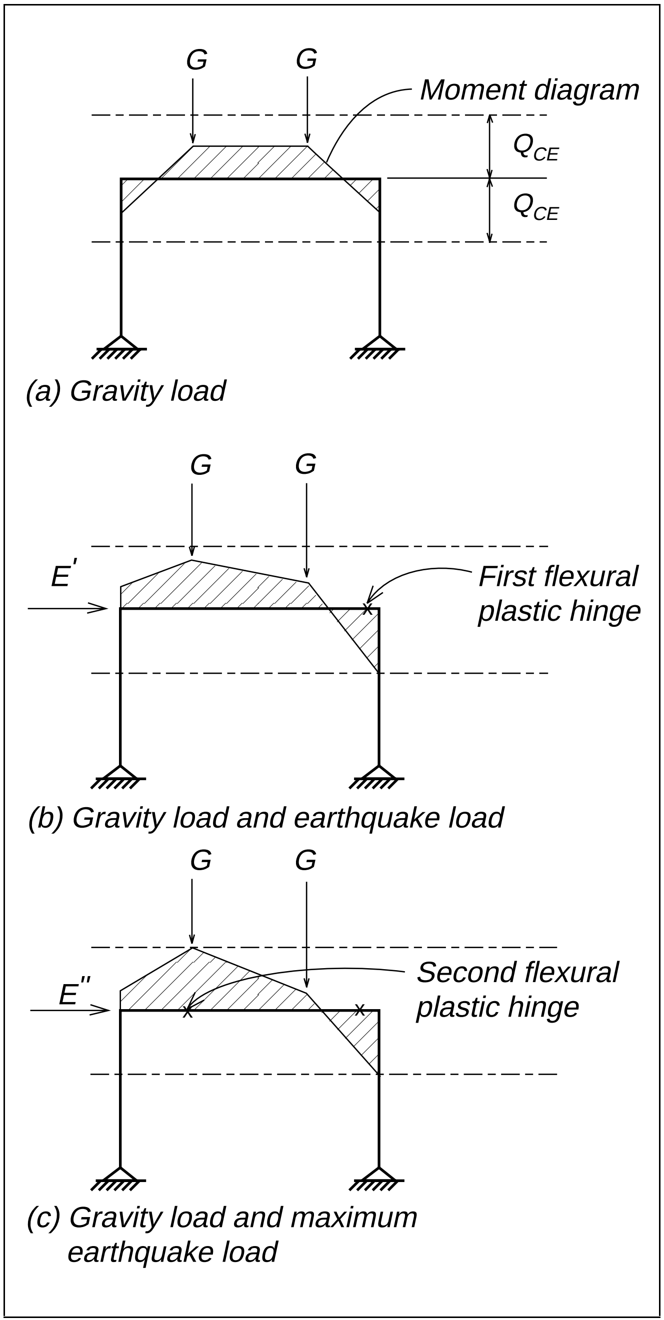


Figure C3-13 Identification of Potential Plastic Hinge Locations

B. Control Node

No commentary is provided for this section.

C. Lateral Load Patterns

The distribution of lateral inertia forces varies continuously during earthquake response. The extremes of the distribution will depend on the severity of earthquake shaking (or degree of nonlinear response), the frequency characteristics of the building and earthquake ground motion, and other aspects. The distribution of inertia forces determines relative magnitudes of shears, moments, and deformations. The loading profile that is critical for one design quantity may differ from that which is critical for another design quantity. Recognizing these aspects, design according to the NSP requires that at least two lateral load profiles be considered. With these two profiles it is intended that the range of design actions occurring during actual dynamic response will be approximately bound. Other load profiles, including adaptive load patterns, may be considered.

Some researchers have proposed adaptive load patterns, that is, patterns that change as the structure is displaced to larger amplitude. Different suggestions have been made in this regard, including the use of story forces that are proportional to the deflected shape of the structure (Fajfar and Fischinger, 1988), the use of load patterns based on mode shapes derived from secant stiffnesses at each load step (Eberhard and Sozen, 1993), and the use of patterns in which the applied story forces are proportional to story shear resistances at each step (Bracci et al., 1995). Because these alternatives require more analysis effort and their superiority to invariant load patterns has not been demonstrated, the use of adaptive load patterns is not required in the *Guidelines*. While these adaptive patterns are not specifically identified in the *Guidelines*, one of these may be substituted for one of the specified patterns in cases where it provides a more conservative bounding load distribution than the other patterns described in the *Guidelines*.

For the time being, only very simple invariant load patterns are specified in the *Guidelines*. The “uniform” load pattern is specified because it emphasizes demands in lower stories over demands in upper stories, and magnifies the relative importance of story shear forces compared with overturning moments. The load pattern based on the coefficient C_{vx} is an option presented for simplicity and consistency with the LSP. When higher mode effects are deemed to be important, a load pattern based on modal forces combined using either the SRSS or CQC methods should also be used. Such a pattern, developed using first and second mode information, is recommended for structures whose fundamental period exceeds 1.0 second. In this manner, credit is given at least to the elastic higher-mode effects.

D. Period Determination

As a structure responds inelastically to an earthquake, the apparent fundamental period changes with response amplitude. Some researchers have proposed to estimate design responses using a fundamental period corresponding to the secant stiffness at maximum displacement. It should be recognized, however, that elastic response spectra provide only an approximation of response once a structure has entered the nonlinear range, regardless of what reference period is used. For this reason, and to simplify the analysis process, the writers have adopted a reference period corresponding to the secant stiffness at 60% of the yield strength. Determination of this period requires that the structure first be loaded laterally to large deformation levels, and that the overall load-deformation relation be examined graphically.

It is not appropriate to use empirical code period equations for T, such as those given in Section 3.3.1.2. Such equations usually provide low estimates for fundamental periods. Low estimates are appropriate for the linear procedures, because they generally result in larger spectral design forces to be applied to the mathematical model, and therefore lead to more conservative results when used with the linear procedures. On the contrary, it is more conservative to use a high estimate of fundamental period for the NSP because it will usually result in a larger target displacement.

It is recommended to evaluate the use of secant stiffness at 60% of yield strength by considering its sensitivity to component verification. The intent of the specified secant stiffness is to approximate (within the structural displacement range of zero to target displacement) the nonlinear force-displacement relationship with a bilinear relationship. The best choice may be to have approximately equal area under both curves. Note that in most cases it is more conservative to use a lower yield displacement and a lower secant stiffness.

E. Analysis of Three-Dimensional Models

No commentary is provided for this section.

F. Analysis of Two-Dimensional Models

Three-dimensional analysis models are, in principle, more appropriate than two-dimensional analysis models. However, at the time of this writing, limitations in analysis software are such that three-dimensional analysis is likely to require significantly greater analysis effort, which may not be justified for relatively symmetric buildings. Therefore, two-dimensional models may be used. The use of three-dimensional models is encouraged wherever their use is feasible.

The procedure outlined in Section 3.3.3.2F for capturing the effects of torsion is only approximate, and cannot account for the effects of inelastic torsion. Three-dimensional analysis is recommended wherever possible for buildings with either low torsional stiffness, or substantial elastic torsional response.

The rule for multidirectional excitation is adapted from Section 3.2.7 for analysis of two-dimensional models.

C3.3.3.3 Determination of Actions and Deformations

Actions and deformations in components and elements are to be calculated at a predetermined displacement of the control node. The predetermined displacement is termed the target displacement.

A. Target Displacement

The *Guidelines* present one recognized procedure for calculating the target displacement. Other procedures also can be used. This commentary presents background information on two acceptable procedures. The first procedure, here termed Method 1, is that described in the *Guidelines*. The second procedure, here termed Method 2, and commonly referred to as the Capacity Spectrum Method, is described here but not in the *Guidelines*.

Method 1. This method is presented in the *Guidelines* for the NSP. It uses data from studies of SDOF systems to determine the target displacement for a multi-degree-of-freedom (MDOF) building. Baseline data used to estimate target displacements have been derived from statistical studies on bilinear and trilinear, non-strength-degrading SDOF systems with viscous damping equal to 5% of the critical value. In order to transform the response of an MDOF building into that of an equivalent SDOF system, the nonlinear force-deformation relation determined from the NSP must be replaced by a bilinear relationship. This transformation is illustrated in Figure C3-14. Additional details on the transformation from the MDOF building to the SDOF model are provided in the supplemental information at the end of this section.

The available SDOF and MDOF studies show that the maximum displacement response of a structure responding to an earthquake ground motion is governed by many parameters. Of primary importance is the effective stiffness of the structure, as represented by K_e in the NSP. The strength is mainly important for structures with a short fundamental vibration period relative to the predominant period of the ground motion; this parameter is represented in the NSP through the strength ratio *R*. Pinching and strength degradation can lead to increased displacements; these effects are difficult to characterize. As such, the effects of pinching and strength degradation (that is, the shape of the hysteresis loop) are lumped together and represented by the coefficient C_2. Post-yield stiffness tends to be important only if the stiffness approaches zero or becomes negative due to either

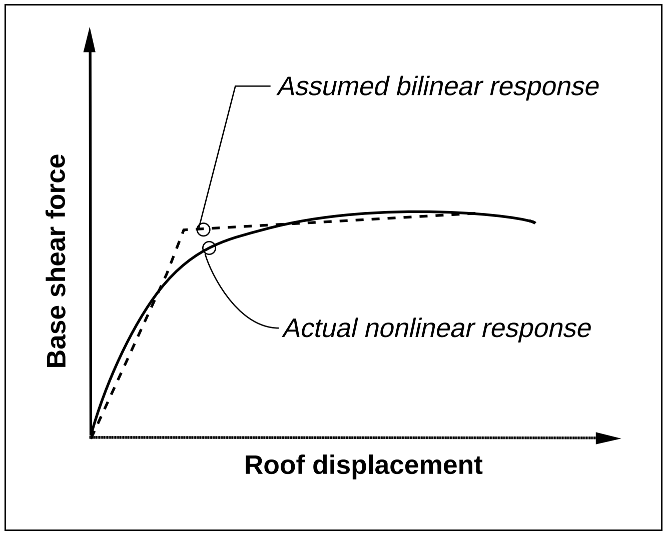


Figure C3-14 Base Shear Versus Displacement Relations

strength degradation of components or to P-Δ effects; these effects are captured approximately by coefficient C_3. The various coefficients in Equation 3-11 are discussed below.

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 (= $\Gamma_{1,r}$):

$$C_0 = \Gamma_{1,r} = \phi_{1,r}\frac{\{\phi_1\}^T[M]\{1\}}{\{\phi_1\}^T[M]\{\phi_1\}} \quad \text{(C3-3)}$$
$$= \phi_{1,r}\Gamma_1$$

where $[M]$ is a diagonal mass matrix, and Γ_1 is the first mode mass participation factor. Since the mass matrix is diagonal, Equation C3-3 can be rewritten as:

$$C_0 = \phi_{1,r}\frac{\Sigma_1^N m_i\phi_{i,n}}{\Sigma_1^N m_i\phi_{i,n}^2} \quad \text{(C3-4)}$$

where m_i is the mass at level *i*, and $\phi_{i,n}$ is the ordinate of mode shape *i* at level *n*. 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.

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

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 Equation 3-12. However, it is permitted to calculate this coefficient using the more approximate, and in some cases less conservative, procedure allowed for in the LSP. Limitation of the value of C_1 to the value used for the linear procedures is introduced so as not to penalize the use of the NSP. Additional discussion of this coefficient is in the commentary to Section 3.3.1.3.

Coefficient C_2. This coefficient adjusts design values based on the shape of the hysteresis characteristics of the building. See the commentary to Section 3.3.1.3 for additional discussion.

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 inter-story 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 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, calculated only for those buildings that exhibit negative post-yield stiffness, given by Equation 3-13, represents a substantial simplification and interpretation of much analysis data. For information, refer to Figure C3-15:the displacement amplification may become very large for bilinear small

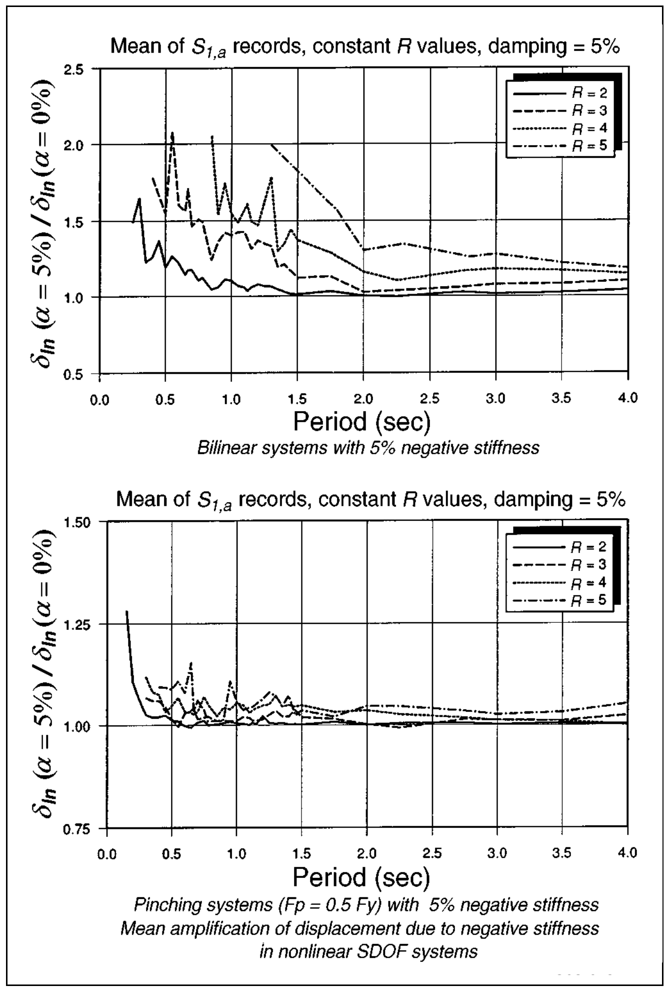


Figure C3-15 Effects of Negative Stiffness on Displacement Amplification

systems with short periods and low strength, even for values of negative stiffness (e.g., α = –0.05). The amplification is smaller for pinched hysteresis systems. Moreover, the mean results are erratic because of differences in the strong ground motions used for the analysis. The compromise offered in Equation 3-13 was to express the displacement amplification for bilinear systems by an approximate equation and to use half this value for coefficient C_3. This compromise is rationalized as follows. First, most buildings behave more like stiffness-degrading models than bilinear models. Second, in most buildings, the negative stiffness is not developed until after significant deformations have occurred. This decreases the P-Δ effects with respect to bilinear systems. However, negative stiffness in the base shear-roof displacement relation may not be representative of the negative stiffness in the critical story (likely the bottom story of the building). More work is needed in this subject area.

Method 2. Details of this procedure are not defined in the *Guidelines*, but it is considered an acceptable alternative procedure. In Method 1, the design displacement response is calculated using an initial effective stiffness. Method 2 determines maximum response based on the displacement corresponding to the intersection of the load-displacement relation (also known as the capacity curve) for the building and the spectral demand curve used to characterize the design seismic hazard. Method 2 uses initial effective stiffness and secant stiffness information to calculate the target displacement. Figure C3-16 illustrates the different stiffnesses used by the two methods, plotted in relation to the anticipated nonlinear load-displacement relation for the structure loaded to its design (target) displacement. Ideally, the two methods should produce the same design displacement. This is achieved for most cases by using different damping values for the two methods. Method 1 uses the damping effective for response near the yield level, typically 5% of the critical value. Method 2 uses a higher damping value, determined based on the shape of the hysteresis and the maximum deformation level.

This method is similar to the Capacity Spectrum Method. Further details on the Capacity Spectrum Method are in Army (1996), ATC (1982, 1996), Freeman et al. (1975), Freeman (1978), and Mahaney et al. (1993). The general procedure for using the method is similar to that for the NSP, described in the commentary on Section 3.3.3.2A. The procedure, including iterations that may be necessary, is described below.

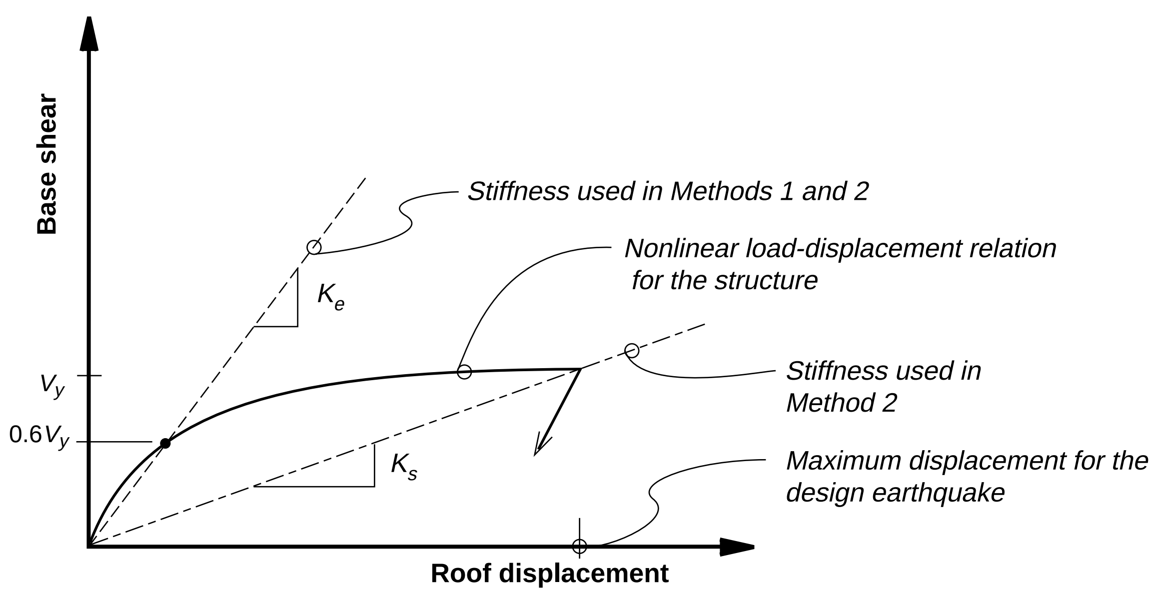


Figure C3-16 Stiffness Calculations for Estimating Building Response

Steps 1–7. These steps are identical to those described in Section C3.3.3.2A.

Step 8. The target displacement is estimated, based on either an initial assumption or information obtained from previous iterations in the procedure. Given this target displacement, an effective initial stiffness K_e is determined using procedures described in Section 3.3.3.2D. The secant stiffness K_s is defined by the slope of a line from the origin to the nonlinear load-deformation relation at the point corresponding to the target displacement. The corresponding global displacement ductility is defined as $\mu_{\delta g} = K_e / K_s$.

Step 9. The equivalent viscous damping is determined as a function of the global displacement ductility and the expected shape of the hysteresis relation for response at that ductility level using either explicit calculation (ATC, 1996) or tabulated data for different seismic framing systems (Army, 1996).

Step 10. Given the equivalent viscous damping determined as described above, a design response spectrum for that damping is constructed. As described in Section 2.6.1.5, this can be achieved by first constructing the general acceleration response spectrum for 5% damping, and then modifying it by the coefficients in Table 2-15 for different levels of damping. The acceleration response spectrum can be converted to a displacement response spectrum by multiplying the acceleration response spectrum ordinates by the factor $T^2/(4\pi^2)$. Figure C3-17 illustrates the effect of different damping levels on a typical acceleration and displacement response spectrum.

Step 11. Compare the displacement response amplitude calculated for the assumed secant stiffness and damping with the displacement amplitude assumed in Step 8. If the values differ by more than about 10%, iterate the process beginning with Step 8.

As noted in Step 10, the spectral acceleration and spectral displacement spectra are related by the factor $T^2/(4\pi^2)$. Therefore, it is possible to plot both the spectral acceleration and the spectral displacement on a single graph. Figure C3-18 plots an example for a range of equivalent viscous damping. The radial lines correspond to lines of constant period. This form of the design loading is convenient because it can be compared directly with the nonlinear load-deformation relation for the building, normalized with respect to the equivalent SDOF coordinates as described in the Supplemental Information on the NSP below. Using this format, the target displacement for the equivalent SDOF system is at the intersection of the load-deformation envelope with the response spectrum for the appropriate damping level. Note that the target displacement for the equivalent SDOF system in general is not the same as the target displacement at the roof level; to arrive at the roof level target displacement requires transformation back to the MDOF system.

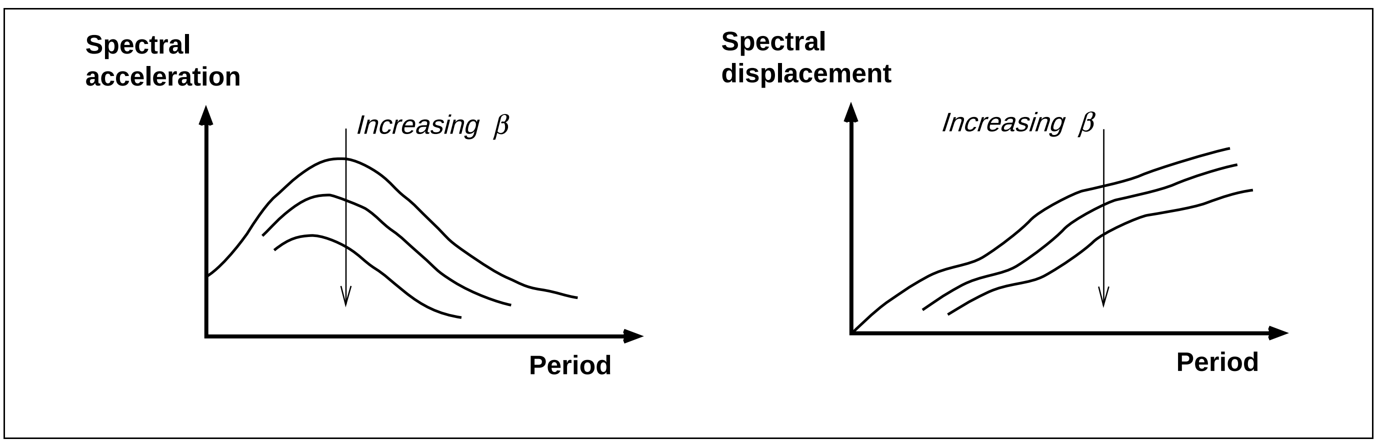


Figure C3-17 Spectral Acceleration and Displacement Curves

Supplemental Information on the NSP. The NSP is based in part on the assumption that the response of a building can be related to the response of an equivalent SDOF system. This implies that response is controlled

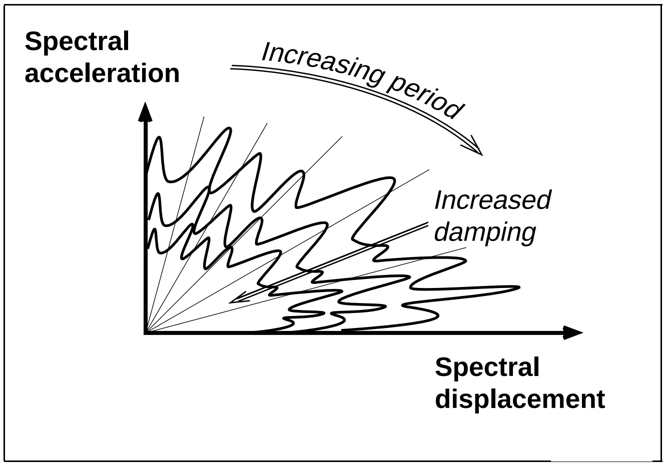


Figure C3-18 Spectral Demand Curves

by a single mode, and that the shape of this mode remains essentially constant throughout the response history. Although both assumptions are incorrect, pilot studies (Saiidi and Sozen, 1981; Fajfar and Fischinger, 1988; Qi and Moehle, 1991; Miranda, 1991; Lawson et al., 1994) have indicated that these assumptions lead to reasonable predictions of the maximum seismic response of MDOF buildings, provided response is dominated by the first mode.

The formulation of the equivalent SDOF system assumes that the deflected shape of the MDOF system can be represented by a shape vector, $\{\Phi\}$, that remains constant throughout the response history, regardless of the level of deformation. The choice of the shape vector is discussed at the end of this section. The transformation of the MDOF system to an equivalent SDOF system is derived below.

The governing differential equation of the MDOF system is:

$$[M]\{\ddot{X}\} + [C]\{\dot{X}\} + \{Q\} = -[M]\{1\}\ddot{x}_g \qquad \text{(C3-5)}$$

where $[M]$ and $[C]$ are the mass and damping matrices, $\{X\}$ is the relative displacement vector, and $\ddot{x}_g$ is the ground acceleration history. Vector $\{Q\}$ denotes the story force vector. Let the assumed shape vector $\{\Phi\}$ be normalized with respect to the roof displacement, x_t; that is, $\{X\} = \{\Phi\}x_t$. Substituting this expression for $\{X\}$ in Equation C3-5 yields:

$$\begin{aligned}&[M]\{\Phi\}\ddot{x}_t + [C]\{\Phi\}\dot{x}_t + \{Q\}\\ &= -[M]\{1\}\ddot{x}_g\end{aligned} \qquad \text{(C3-6)}$$

Define the SDOF reference displacement x^r as:

$$x^r = \frac{\{\Phi\}^T[M]\{\Phi\}}{\{\Phi\}^T[M]\{1\}}x_t \qquad \text{(C3-7)}$$

Pre-multiplying Equation C3-6 by $\{\Phi\}^T$ and substituting for x_t using Equation C3-7 results in the governing differential equation for the response of the equivalent SDOF system:

$$M^r\ddot{x}^r + C^r\dot{x}^r + Q^r = -M^r\ddot{x}_g \qquad \text{(C3-8)}$$

where:

$$M^r = \{\Phi\}^T[M]\{1\} \qquad \text{(C3-9)}$$

$$Q^r = \{\Phi\}^T\{Q\} \qquad \text{(C3-10)}$$

$$C^r = \{\Phi\}^T[C]\{\Phi\}\frac{\{\Phi\}^T[M]\{1\}}{\{\Phi\}^T[M]\{\Phi\}} \qquad \text{(C3-11)}$$

The force-displacement relation of the equivalent SDOF system can be determined from the results of an NSP of the MDOF structure (Figure 3-1) using the shape vector established above. To identify global strength and displacement quantities, the multilinear relation is represented by a bilinear relationship that is defined by a yield strength, an average elastic stiffness ($K_e = V_y/\delta_y$), and a softening stiffness, K_s ($=\alpha K_e$). For reference, the force versus displacement relations for the MDOF system and the equivalent SDOF system are presented in Figure C3-19.

The base shear force at yield (V_y) and the corresponding roof displacement ($x_{t,y}$) from Figure C3-19 are used together with Equations C3-7 and C3-10 to compute the force-displacement relationship for the equivalent SDOF system as follows. The initial period of the equivalent SDOF system (T_{eq}) can be computed as:

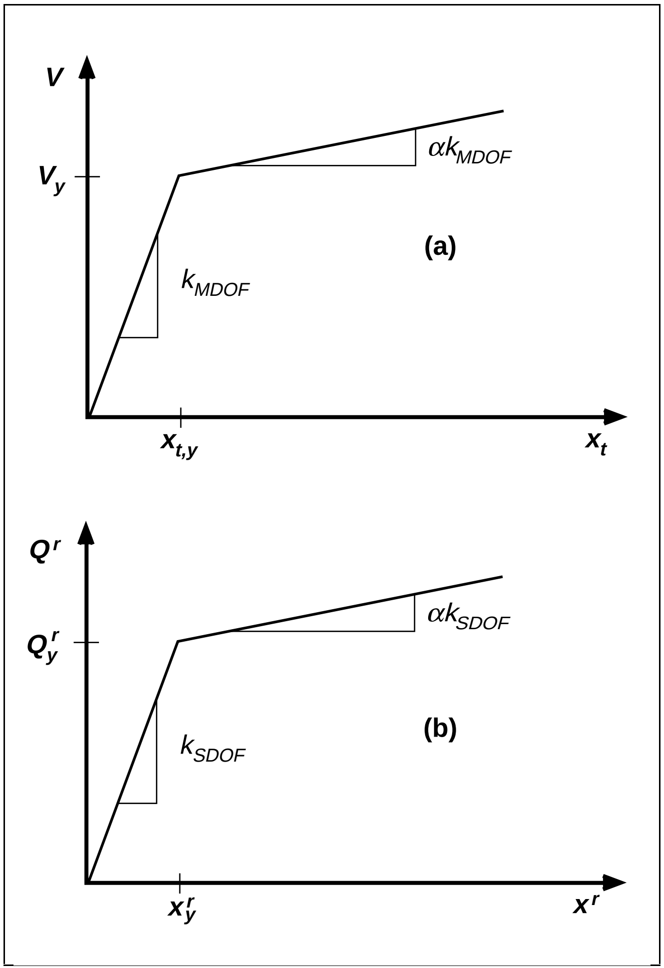


Figure C3-19 Force-Displacement Relations of MDOF Building and Equivalent SDOF System

$$T_{eq} = 2\pi\left[\frac{x^r_y M^r}{Q^r_y}\right]^{1/2} \tag{C3-12}$$

where the reference SDOF yield displacement x^r_y is calculated as:

$$x^r_y = \frac{\{\Phi\}^T[M]\{\Phi\}}{\{\Phi\}^T[M]\{\Phi\}}x_{t,y} \tag{C3-13}$$

and the reference SDOF yield force, Q^r_y, is calculated as:

$$Q^r_y = \{\Phi\}^T\{Q_y\} \tag{C3-14}$$

where $\{Q_y\}$ is the story force vector at yield, namely, $V_y = \{1\}^T\{Q_y\}$.

The strain-hardening ratio (α) of the force-displacement curve of the MDOF structure will define the strain-hardening ratio of the bilinear force-displacement curve of the equivalent SDOF system.

Using the above information, the equivalent SDOF system is now characterized. The next step in the analysis process is the calculation of the target displacement for the purpose of performance evaluation. The properties of the equivalent SDOF system, together with spectral information for inelastic SDOF systems, provide the information necessary to estimate the target displacement.

For elastic SDOF systems, the spectral displacement can be obtained directly from the design ground motion spectrum. If spectral accelerations are given, the spectral displacements S_d can be calculated as $S_aT^2/(4\pi^2)$ where T is the period of the elastic SDOF system.

Displacements of nonlinear (inelastic) SDOF systems differ from those of linearly-elastic SDOF systems, particularly in the short-period range (see Figure C3-9). In the short-period range, the ratio of inelastic to elastic displacement depends strongly on the inelastic deformation demand for the system, which is expressed in terms of the ductility ratio. The relation between the ductility ratio and the ratio of elastic to inelastic strength demands can be expressed by relationships (see Figure C3-9), which have been developed recently by several investigators (Miranda and Bertero, 1994).

Thus, to calculate a target displacement, the ductility demand for the equivalent SDOF system must be calculated. This last step requires the engineer to estimate of the ratio of elastic strength demand to yield strength of the equivalent SDOF system. Since inelastic spectra are usually obtained for unit mass systems, it is convenient to divide Equation C3-8 by M^r to obtain the differential equation for the unit mass equivalent SDOF system:

$$\ddot{x}^r + \frac{C^r}{M^r}\dot{x}^r + \frac{Q^r}{M^r} = -\ddot{x}_g \qquad \text{(C3-15)}$$

Equation C3-15 describes the response of a unit mass SDOF system with period T_{eq} and yield strength $F_{y,\,eq}$ given as

$$F_{y,\,eq} = \frac{Q_y^r}{M^r} \qquad \text{(C3-16)}$$

If the elastic response spectrum is known, the elastic strength demand of the unit mass equivalent SDOF system can be computed as:

$$F_{e,\,eq} = S_a(T_{eq}) \qquad \text{(C3-17)}$$

where the term on the right-hand side of the equation is the spectral acceleration ordinate. The strength reduction factor R can then be obtained from the relationship

$$R = \frac{F_{e,\,eq}}{F_{y,\,eq}} = \frac{S_a(T_{eq})M^r}{Q_y^r} \qquad \text{(C3-18)}$$

The ductility demand of the equivalent SDOF system can now be obtained from published $R - \mu - T$ relationships.

Note that the published data presents mean results; for essential and other important structures, the reader is encouraged to use mean plus one standard deviation displacement demands in lieu of mean displacement demands.

Since the ductility demands of the equivalent SDOF system and the MDOF structure are assumed to be equal, the target displacement of the MDOF system, $x_{t,\,t}$, is given by

$$x_{t,\,t} = \mu x_{t,\,y} \qquad \text{(C3-19)}$$

Further modifications to the target displacement may be needed to account for local soil effects, effects of strength and stiffness degradation, second-order effects, and other factors that may significantly affect displacement response.

The two key quantities needed to compute the target displacement are the period (T_{eq}) and the yield strength ($F_{y,\,eq}$) of the equivalent SDOF system. These quantities depend on the shape vector $\{\phi\}$, the story force vector $\{Q\}$, and the mass distribution over the height of the building. The need for a simplified approach makes necessary the use of readily available parameters to estimate these quantities. The first mode period (T_1) and the first mode participation factor (PF_1) are suitable for this purpose. Given the substantial variations in the shape vector, the following assumptions are made:

$$T_{eq} = T_1 \qquad \text{(C3-20)}$$

$$F_{y,\,eq} = \frac{V_y}{W}PF_1 \qquad \text{(C3-21)}$$

The accuracy of these assumptions was investigated in a sensitivity study using a triangular story force vector, equal masses at each floor, and shape vectors for wall structures—ranging from an elastic deflected shape to a straight line deflected shape (representing plastic hinging at the base and no elastic deformations). The plastic component of the roof displacement is described by the parameter p as shown in Figure C3-20. The results of the study are presented in Figure C3-21. The lower plot demonstrates the accuracy of Equation C3-21.

This study, and a companion study using shape vectors representing framed structures with story mechanisms, indicate that T_{eq} and $F_{y,\,eq}$ are insensitive to the choice of shape vector. Accordingly, the expression for the strength ratio R given by Equation 3-12 is likely adequate.

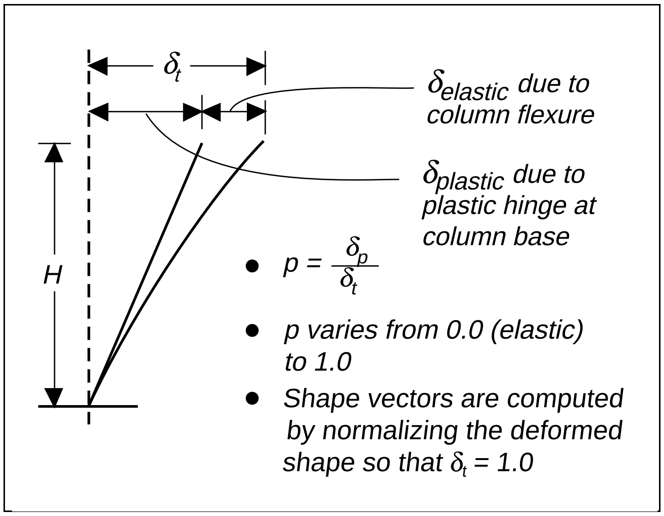


Figure C3-20 Shape Vectors used in Sensitivity Study

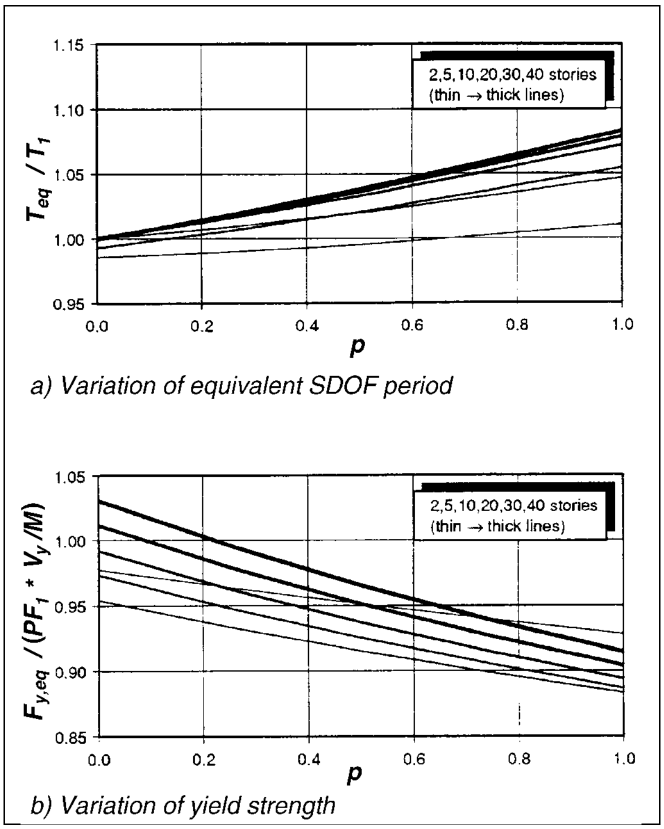


Figure C3-21 Sensitivity to the Choice of Shape Vector

B. Floor Diaphragms

Floor diaphragms shall be designed to transfer the inertia forces calculated using either of the linear procedures (Sections 3.3.1.3D or 3.3.2.3B) plus the horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm.

Other rational procedures may be used to calculate the inertia forces at each floor level for the purpose of diaphragm design.

C3.3.4 Nonlinear Dynamic Procedure (NDP)

C3.3.4.1 Basis of the Procedure

No commentary is provided for this section.

C3.3.4.2 Modeling and Analysis Assumptions

A. General

The modeling and analysis considerations described in Section C3.3.3.2 apply to the NDP unless superseded by provisions in Section 3.3.4.1. All masses in the building must be represented in the mathematical model and located so as to adequately capture horizontal and vertical inertial effects.

Diaphragms may be assumed to behave in the elastic range to simplify the nonlinear model. However, if the diaphragm represents the primary nonlinear element in the structural system, the mathematical model should include the nonlinear force-deformation characteristics of the diaphragm (Kunnath et al., 1994).

B. Ground Motion Characterization

Ground motion time-histories are required for the NDP. Such histories (or pairs thereof) shall be developed according to the requirements of Section 2.6.1.

C. Time-History Method

See Section C3.3.2.2D for pertinent information.

C3.3.4.3 Determination of Actions and Deformations

A. Modification of Demands

The element and component deformations and actions used for evaluation shall be established using the results of the NDP.

C3.4 Acceptance Criteria

C3.4.1 General Requirements

No commentary is provided for this section.

C3.4.2 Linear Procedures

These acceptance criteria apply for both the LSP and the LDP. (See Section C3.4.2.2A for supplemental information on linear procedures acceptance criteria and Equation 3-18.)

C3.4.2.1 Design Actions

This section defines the actions (forces and moments), including gravity and earthquake effects, for which the evaluation is carried out.

A. Deformation-Controlled Actions

Equation 3-14 defines the deformation-controlled actions for design. This equation states the design actions in force terms, although the intent is to provide an indirect (albeit very approximate) measure of the deformations that the structural component or element experiences for the combination of design gravity loading plus design earthquake loading. Because of possible anticipated nonlinear response of the structure, the design actions as represented by this equation may exceed the actual strength of the component or element to resist these actions. The acceptance criteria of Section 3.4.2.2A 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.

B. Force-Controlled Actions

The basic approach for calculating force-controlled actions for design differs from that used for deformation-controlled actions. The reason is that, whereas nonlinear deformations may be associated with deformation-controlled actions, nonlinear deformations associated with force-controlled actions are not permitted. Therefore, force demands for force-controlled actions must not exceed the force capacity (strength).

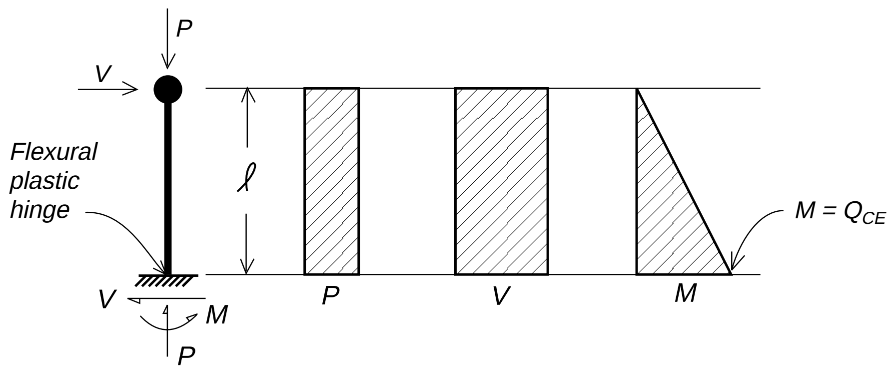
Ideally, an inelastic mechanism for the structure will be identified, and the force-controlled actions Q_{UF} for design will be determined by limit analysis using that mechanism. This approach will always produce a conservative estimate of the design actions, even if an incorrect mechanism is selected. 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 Equations 3-15 and 3-16. Additional discussion of both approaches is provided below.

Limit analysis to determine force-controlled design actions is relatively straightforward for some components and some structures. The concept is illustrated in a series of structural idealizations in Figure C3-22. Each of these cases is discussed briefly in the paragraphs below.

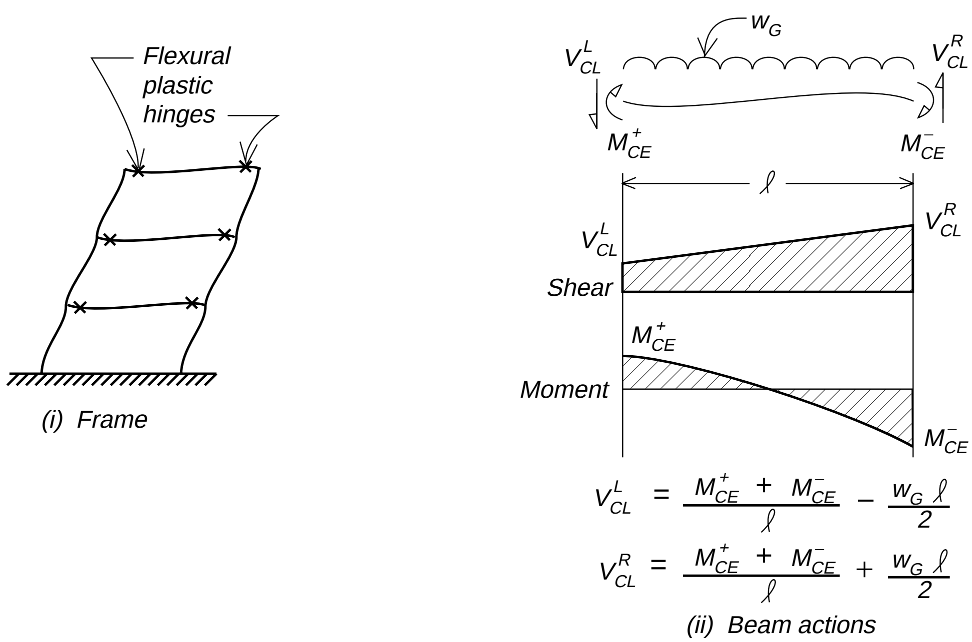
Figure C3-22(a) illustrates a structure consisting of a single cantilever column with a mass at the top. The deformation-controlled action is flexure at the column base. Force-controlled actions include axial load and shear force. Assuming a nonlinear mechanism involving flexure at the base of the column, and using the expected moment strength Q_{CE} at that location, the design shear force is calculated from equilibrium to be equal to Q_{CE}/l, where l is the column length. Because earthquake loading produces no axial force in this column, the design axial force is equal to the gravity level value.

Figure C3-22(b) illustrates a multistory frame. Considering a typical beam, the deformation-controlled actions are flexural moment at the beam ends, and the force-controlled action of interest is the beam shear. Assuming a nonlinear mechanism involving flexure at the beam ends, and using the expected moment strengths Q_{CE} at those locations, the design shear force at various locations along the beam can be calculated from equilibrium of a free-body diagram loaded by the expected moment strengths and gravity loads. This same approach can be used to determine the design shear force in columns of frames.

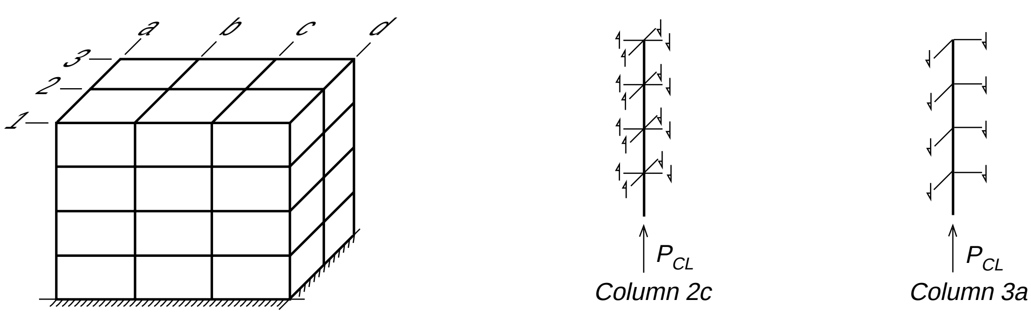
Note that beam flexural moment along the length of the beam may also be assumed to be a force-controlled action because flexural yielding is not desired away from the beam ends. The beam moment diagram, determined from equilibrium of the free-body diagram, identifies the appropriate moments to be checked against the beam moment strength along the beam span.



a) Cantilever column supporting concentrated mass



b) Beam with flexural plastic hinges at beam ends



c) Column axial load determined as the sum of the beam shears. Note that in the limit, the beam shear is equal to the shear corresponding to beam flexural hinging (see part b).

Figure C3-22 Checking for Force-Controlled Actions

Figure C3-22(c) illustrates a multistory frame. Considering interior and exterior columns, the deformation-controlled actions are flexural moment at the column ends, and the force-controlled actions of interest are column shear and axial load. Assume for this example that we are interested in identifying the column axial load for design. A mechanism suitable for obtaining design axial loads is shown. A free-body diagram of each column is made by making a cut at the intersection with each beam framing into the column, and replacing the beam by the internal forces (moment and shear) that would be acting in the beam at that location (these actions were discussed in the previous example). Note that beams may be framing into the column from two orthogonal framing directions, and that the actions from each beam should be considered. This aspect is especially important for corner columns of frames.

Limit analysis can be used for a broad range of other cases, and specialized mechanisms can be identified that may result in reductions in the design actions that need to be considered.

Equations 3-15 and 3-16 are recommended only for those cases where it is not feasible to determine force-controlled actions for design using limit analysis, or for cases where significant levels of nonlinear action are not anticipated for the design loading.

Equations 3-15 and 3-16 are conservative and can be used to calculate all force-controlled actions. Equation 3-15 can be used to calculate actions that result from forces delivered by yielding components. For instance, it could be used to calculate the axial forces in Columns 2c and 3a (Figure C3-22) wherein the seismic axial forces are delivered by beams yielding in flexure. However, if some of the beams framing into Column 2c do not yield, Equation 3-15 cannot be used and either limit analysis or Equation 3-16 must be used to calculate the design axial force. Other examples of this condition could include pier and spandrel components in pierced shear walls, secondary components and elements, and joints and columns in slab-column framing systems.

The writers recognize that Equation 3-15 is a relatively crude estimator of actual expected forces, and therefore the equation has been defined to produce conservative results in most cases. The rationale used to develop Equation 3-15 follows.

The coefficient J in Equation 3-15 was the subject of much debate in the development of the *Guidelines*, and the final result may not be appropriate in certain cases. According to Equation 3-17, for zones of high seismicity, the value of J may equal 2.0. The result in Equation 3-17 is that the force-controlled action for design is equal to the gravity load action plus half the seismic action calculated by the linear procedures, implying that the structure has sufficient strength to resist only about half the design lateral forces. It is anticipated that most structures in regions of low seismicity will be able to resist the design seismic forces without significant yielding. Therefore, Equation 3-17 has been written so that J will reduce to unity as the spectral acceleration reduces.

Coefficient C_1 in Equations 3-15 and 3-16 is the same coefficient introduced in Equation 3-6. It was introduced in Equation 3-6 to amplify the design base shear to achieve a better estimate of the maximum displacement for short-period buildings responding in the nonlinear range. Of course, for nonlinear response, the base shear will decrease rather than increase. Thus, in most cases it is reasonable to divide this component back out of the force estimate when seeking forces using Equations 3-15 and 3-16. Coefficients C_2 and C_3 in Equations 3-15 and 3-16 were introduced in Equation 3-6 to increase the pseudo lateral load to capture the effects on maximum displacement response due to pinching and strength degradation, and second-order effects, respectively. None of these three effects will increase the base shear force. As such, these coefficients are divided back out of the seismic force estimate of Equations 3-15 and 3-16.

C3.4.2.2 Acceptance Criteria for Linear Procedures

A. Deformation-Controlled Actions

In the linear procedures of Sections 3.3.1 and 3.3.2, a linearly-elastic model of the structure is loaded by lateral forces that will displace the model to displacements expected in the building as it responds to the design earthquake. If the building responds nonlinearly, as is often the case, the lateral forces and corresponding internal forces will exceed yielding values. The degree to which the calculated internal forces exceed the component strengths is used as a measure of the extent of nonlinear deformations that develop in the component. The acceptance criteria for deformation-controlled actions, as expressed by

Equation 3-18, are based on this concept. In Equation 3-18, the design actions Q_{UD} may exceed the actual strength of the component, Q_{CE}. The modifier *m* in Equation 3-18 provides a measure of the ductility capacity of the component associated with the expected inelastic deformation mode. Figure C3-23 illustrates the *m* factor for a moment-rotation (M-ϕ) deformation-controlled action on a component or element. (Note: The *m* factor is also applicable for axial and shear deformations.) M_e (or in the notation of Equation 3-14, Q_{UD}) is the design moment (action) due to gravity loads and earthquake loads that the component or element would experience if the component or element were to remain elastic. $M_{CE} = Q_{CE}$ is the expected strength of the component or element at the expected deformation of the component or element. Thus, $m = Q_{UD}/Q_{CE}$ or $mQ_{CE} = Q_{UD}$. In Chapters 4 through 8, *m* factors are given for determining the acceptability of various soil foundation, steel, concrete, masonry, and wood components or elements. Chapter 8 also includes *m* factors for wood connections. The derivation of Equation 3-18 is provided below.

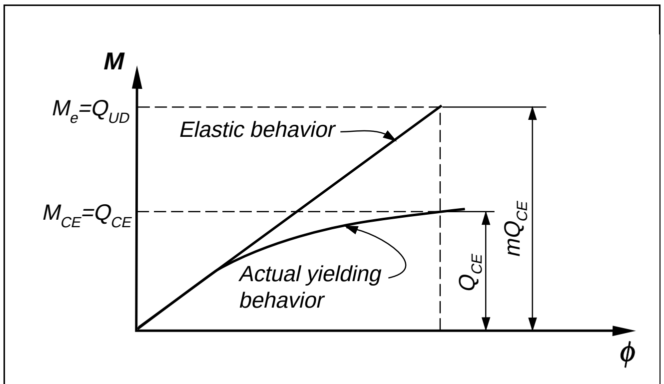


Figure C3-23 Basis for m Factor (using M as Representative of a Deformation-Controlled Action)

The expected strength of the component or element, Q_{CE}, should be calculated as the largest resistance obtained for deformations up to and including the maximum deformations to be experienced by the component for the design earthquake loading. Its calculation should take into consideration actual material properties, including strain hardening, and actual cross sections, including composite action with interconnected materials where appropriate. Procedures for calculation of Q_{CE} are specified in Chapters 5 through 8.

Note that all secondary components and elements, which are required to be excluded from the mathematical model when using the linear procedures, must be checked to ensure that they have adequate deformation capacity. This can either be done directly for each component or element where drift capacities are known, or alternately, a secondary mathematical model can be constructed that includes the secondary components. This model is subjected to the design displacements obtained for the linear procedure. All deformation-controlled actions are then checked according to Equation 3-18.

Supplemental Information on Linear Procedure Acceptance Criteria and Equation 3-18.
Equation 3-18 sets the acceptance criterion for deformation-controlled actions. This equation is a displacement-based check that is expressed in force units for ease of implementation. In Equation 3-14, the gravity force actions (Q_G) calculated using Equations 3-2 and 3-3 are combined with the seismic force actions (Q_E), calculated using either Equation 3-6 for the LSP or Section 3.3.3 for the LDP. The resulting action is then compared with the expected capacity of the component that is increased by a component demand modifier, *m*.

Figures C3-24 and C3-25 illustrate the intent of Equation 3-18. The subject frame in these figures is a one-bay portal frame. It is assumed that gravity loads are applied to the beam only, and seismic inertial loads are only developed at the level of the beam. The internal actions in the beam and columns, resulting from the application of the gravity and seismic loads, are indicated in Figures C3-24 and C3-25, respectively. The following formulation assumes a statistical relation between inelastic and elastic displacements.

First, the beam is considered. Assumed loads and actions, and key response histories are indicated in Figure C3-24. It is assumed that flexure in the beam is designated as deformation-controlled. Shear and axial load effects are to be ignored. The history of the beam (and the frame) begins at point “a”. Under gravity loads (loading to point “b”), the lateral displacement of the beam is zero, while the moment at the beam end increases from zero to M_G. The beam flexural

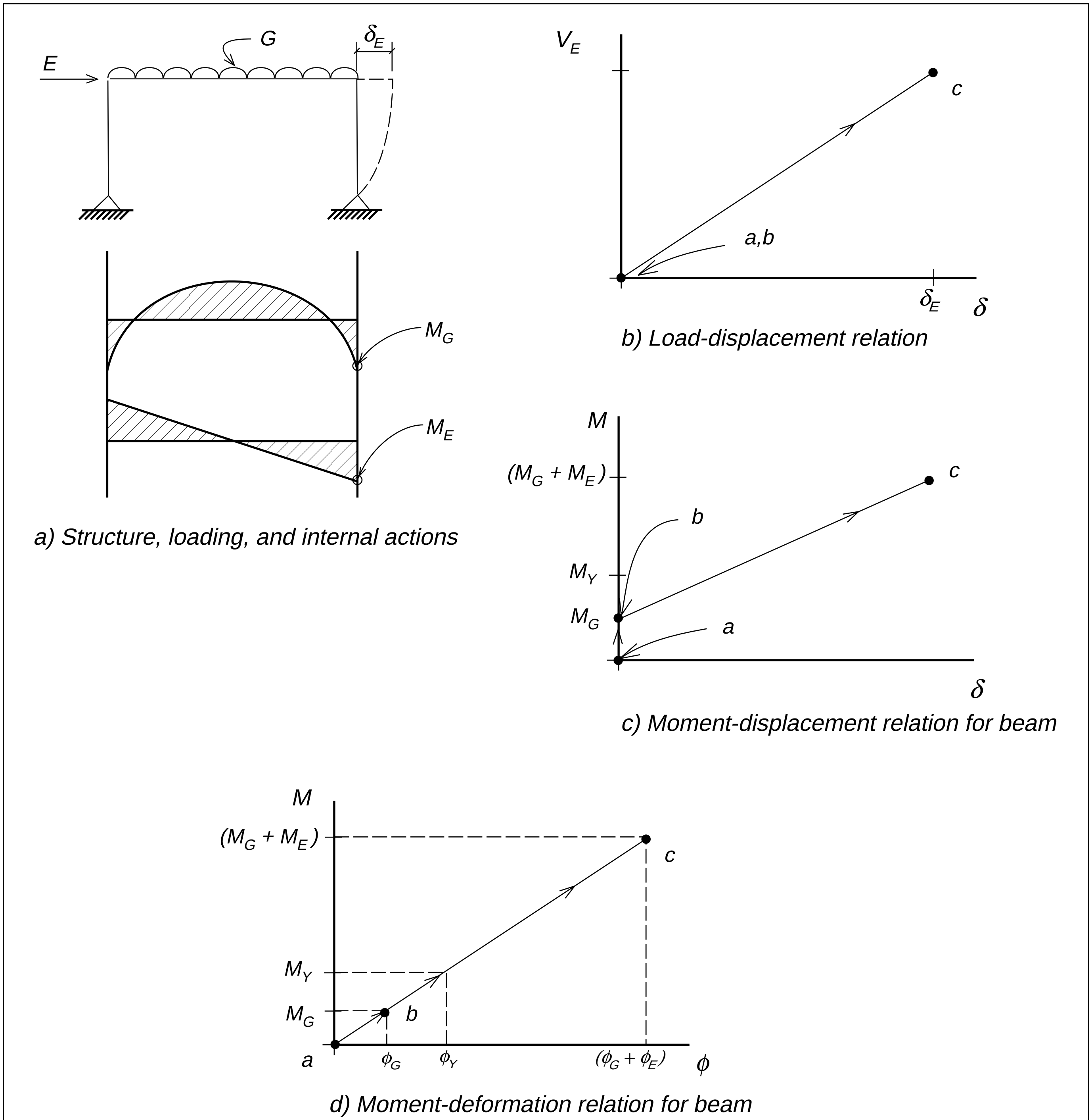


Figure C3-24 Frame Evaluation - Beam Information

deformation, expressed in the figure as ϕ, increases from zero to ϕ_G. Under lateral earthquake loading (loading to point "c"), the moment at the beam end increases from M_G to $(M_G + M_E)$. The beam deformation increases from ϕ_G to $(\phi_G + \phi_E)$. Assuming that the beam deformation increases linearly from zero to $\phi_G + \phi_E$—which will likely not quite happen because of the difference in the curvature distributions for gravity and seismic loading—it can be written that:

$$\mu_\phi = \frac{\phi_G + \phi_E}{\phi_Y} = \frac{M_G + M_E}{M_Y} \qquad \text{(C3-22)}$$

where μ_ϕ is component ductility expressed in terms of ϕ, and ϕ_Y and M_Y refer to component yield deformation and force, respectively. Reorganizing the terms in Equation C3-22 results in:

$$M_G + M_E = \mu_\phi M_Y \qquad \text{(C3-23)}$$

Equation C3-23 is essentially Equation 3-18 with μ_ϕ replacing the component demand modifier, *m*, and Q_{UD} replacing the sum of M_G and M_E.

Second, the columns in the sample frame are considered. Assumed loads and actions, and key response histories are indicated in Figure C3-25. It is assumed that flexure in the column is designated as a deformation-controlled action. The history of the column begins at point "a". Under gravity loads (loading to point "b"), the lateral displacement is zero, while the moment at the column end increases from zero to M_G, and the axial load increases from zero to P_G. The column deformation, expressed as ϕ, increases from zero to ϕ_G. Under lateral earthquake loading (loading to point "c"), the moment at the beam end increases from M_G to $(M_G + M_E)$. The column deformation increases from ϕ_G to $(\phi_G + \phi_E)$. It is clear that column deformation and column moment follow a similar path to those of the beam described above. As such, Equation 3-18 applies to the column moment. However, this equation may not apply to the axial load—a quantity that is needed to calculate M_Y. Rather, axial load may follow a very different path, and a different procedure is required to calculate it.

Equations 3-15 and 3-16 are an attempt to provide a simple and conservative estimate of the forces that occur in a component under gravity and earthquake loading. These equations should be used unless the engineer carries out limit analysis of the frame to calculate the axial load that exists when the frame is displaced to cause yielding of all actions contributing to the axial force in the members—the preferred solution method. Refer to Figure C3-26, which considers both an interior column and an exterior column. The history of this frame begins at point "a". Under gravity loads (loading to point "b"), the lateral displacement is zero, while the axial force increases to P_G. Under the application of the equivalent base shear (equal to V for the LSP), the lateral displacement increases to δ_e (loading to point "c"). The axial load in the interior column remains constant, while the axial load in the exterior column computed from the linear elastic model increases to $(P_G + P_E)$. However, because of yielding in the building frame, the maximum base shear is unlikely to reach the equivalent base shear. As the frame begins to yield, it is also likely that the axial load in the exterior column will increase at a decreasing rate. Without carrying out a detailed analysis, it is virtually impossible to pinpoint what will be the axial load in the exterior column. Meanwhile, the interior column is probably carrying the gravity axial load (P_G).

Considering the interior column, it is apparent that Equation 3-18 does not apply to the axial load on the column. In the absence of limit analysis data, Equations 3-15 and 3-16 should be used to establish the axial force coexisting with the column moment for checking the acceptability of the column.

B. Force-Controlled Actions

The lower-bound strength of the component or element, Q_{CL}, should be calculated as a mean minus one standard deviation level of resistance, taking into consideration degradation that might occur over the range of deformation cycles to which the component or element may be subjected. Procedures for calculation of Q_{CL} are specified in Chapters 5 through 8.

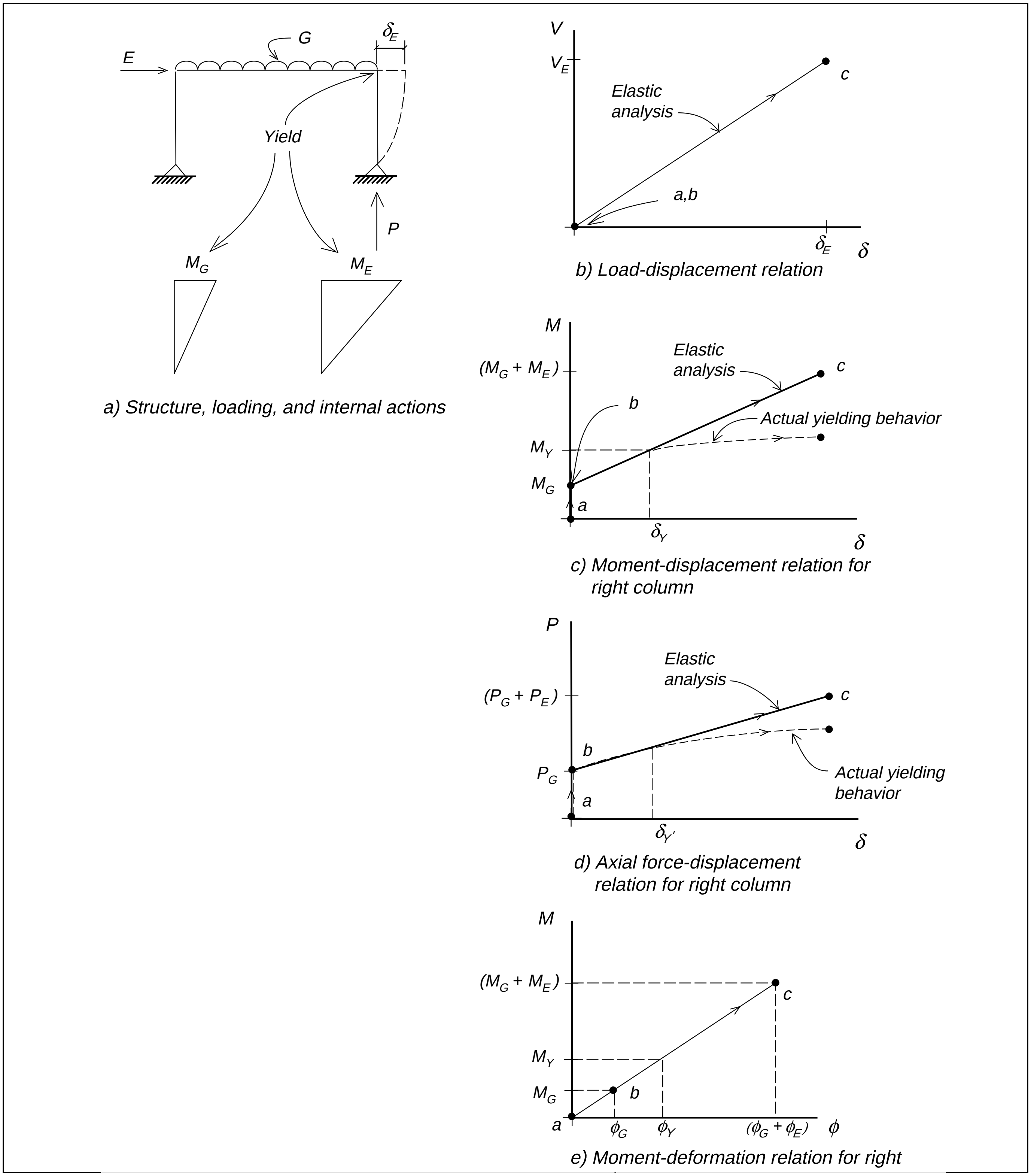


Figure C3-25 Frame Evaluation - Column Information

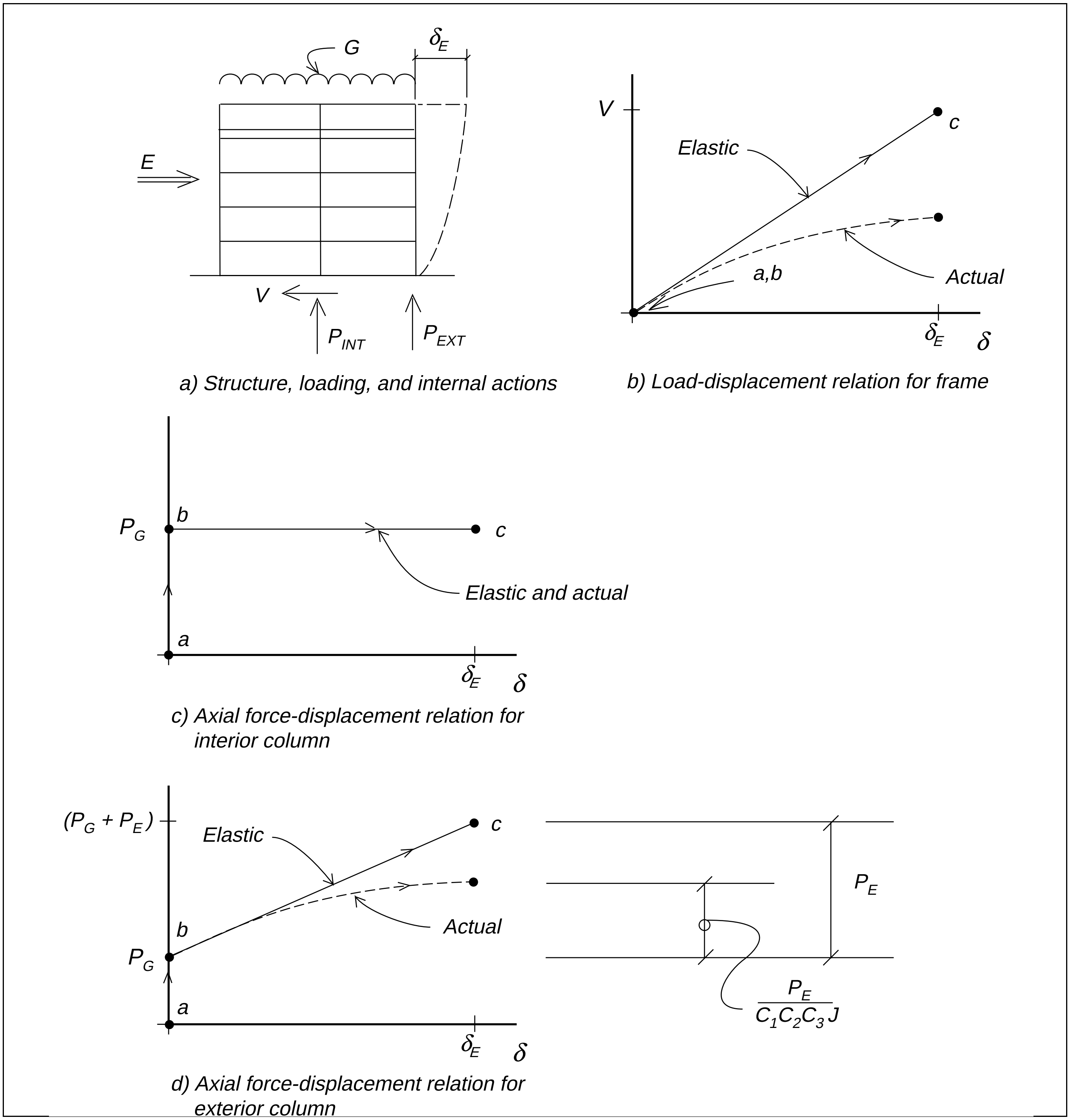


Figure C3-26 Evaluation of a Multibay Frame

Note that all secondary components and elements, which are required to be excluded from the mathematical model when using the linear procedures, must be checked to ensure that they have adequate deformation capacity. This can be done either directly for each component or element where drift capacities are known, or alternatively, a secondary mathematical model can be constructed that includes the secondary components. This model is subjected to the design displacements obtained for the linear procedure. All force-controlled actions are then checked according to Equation 3-19.

C. Verification of Design Assumptions

A primary goal of this section is to ensure that the engineer checks design actions and associated strengths at all locations within a component, rather than just at end points or at nodes used to define the component in a computer model of the building. For example, it is inadequate to check for flexural strength only at the ends of a beam; it is also necessary to check flexural design actions against flexural strengths at other locations on the beam.

For beams evaluated or designed using the linear procedures, it is required that inelastic flexural action be restricted to component ends. This is because the linear procedures can lead to nonconservative results, and may completely misrepresent actual behavior, when flexural yielding occurs along the span length. To check for this case, construct a free-body diagram of the beam loaded at its ends with the expected moment strengths Q_{CE} and along its length with the design gravity loads (Figure C3-13). The moment diagram along the length of the beam can then be constructed from equilibrium principles. The moments along the length of the beam are then compared with the strengths at all locations. For this purpose, the strength may be calculated as Q_{CE} (that is, assuming expected strength rather than lower-bound strength). Where this comparison indicates that flexural strength may be reached at locations more than one beam depth from the beam ends, either the beam should be rehabilitated to prevent inelastic action along the length, or the design should be based on one of the nonlinear procedures (Section 3.3.3 or 3.3.4).

C3.4.3 Nonlinear Procedures

These acceptance criteria apply for both the NSP and the NDP.

C3.4.3.1 Design Actions and Deformations

The NSP and the NDP both provide direct information on force and deformation demands that are associated with the specified design loading. Therefore, it is not necessary to define design forces and deformations for deformation-controlled actions and force-controlled actions using the procedures described for the linear procedures.

C3.4.3.2 Acceptance Criteria for Nonlinear Procedures

Performance evaluation consists of a capacity/demand evaluation of relevant parameters (actions and deformations). Demands are determined directly from the nonlinear procedure. Procedures for determining force and deformation capacities are specified in Chapters 5 through 8.

It must be recognized that capacity may take on a different meaning for different Performance Levels and different deformation levels. In general, strength capacities are calculated according to procedures in Chapters 4 through 8, taking into consideration the deformation level experienced by the component. Different deformation levels are permitted depending on the Performance Level.

Deformation capacities in Chapters 5 through 8 are specified in tabular form in terms of quantities that are commonly available from nonlinear analysis computer programs. At the component level, these deformations are specified in absolute terms, as plastic hinge rotation capacity, shear distortion capacity, and inter-story drift capacity. Ductility ratios are not generally used, since it may be more difficult to interpret the output data from most computer programs in these terms.

It must be recognized that at the time of this writing, neither deformation demands nor deformation capacities can be predicted accurately using the nonlinear procedures, although these procedures are generally believed to be far superior to the linear procedures in this regard. The inability to make accurate predictions may not be a major drawback, because accurate predictions usually are not critical, particularly for components that deteriorate in a gradual manner. Collapse and life-safety hazards are caused primarily by brittle failure modes in components and connections that are important parts of the gravity and lateral load paths. Thus the emphasis (with a focus on the Life Safety Performance Level) needs to be on verification of the following:

1. A complete and adequate load path exists.

2. The load path remains sound at the deformations associated with the target displacement level.

3. Critical connections remain capable of transferring loads between the components that form part of the load path.

4. Individual components that may fail in a brittle mode and that are important parts of the load path are not overlooked (where multiple failure modes are possible, ensuring that each is identified).

5. Localized failures (should they occur) do not violate the goals of the Performance Level; in particular, it must be verified that the loads tributary to the failed components can be transferred safely to other components and that the failed component itself does not pose an unacceptable hazard.

6. Finally, there should be verification of reasonable deformation control. Story drift quantities indicated in Table 2-4 may be used for reference.

C3.5 Definitions

No commentary is provided for this section.

C3.6 Symbols

No commentary is provided for this section.

C3.7 References

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