Chapter 5

STRUCTURAL DESIGN CRITERIA

5.1 REFERENCE DOCUMENT:

The following reference document shall be used for loads other than earthquake and for combinations of loads as indicated in this chapter:

ASCE 7 *Minimum Design Loads for Buildings and Other Structures,* ASCE 7, 1998

5.2 DESIGN BASIS:

5.2.1 General: The seismic analysis and design procedures to be used in the design of *buildings* and other *structures* and their *components* shall be as prescribed in this chapter.

The *structure* shall include complete lateral- and vertical-force-resisting systems capable of providing adequate *strength*, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of *deformation* and *strength* demand. The design ground motions shall be assumed to occur along any direction of the *structure*. The adequacy of the structural systems shall be demonstrated through construction of a mathematical model and evaluation of this model for the effects of the design ground motions. Unless otherwise required, this evaluation shall consist of a linear elastic analysis in which design *seismic forces* are distributed and applied throughout the height of the *structure* in accordance with the procedures in Sec. 5.3 or Sec. 5.4. The corresponding structural *deformations* and internal forces in all members of the *structure* shall be determined and evaluated against acceptance criteria contained in the *Provisions*. Approved alternative procedure based on general principles of engineering mechanics and dynamics are permitted to be used to establish the *seismic forces* and their distribution. If an alternative procedure is used, the corresponding internal forces and *de-formations* in the members shall be determined using a model consistent with the procedure adopted.

Individual members shall be provided with adequate *strength* to resist the shears, axial forces, and moments determined in accordance with the *Provisions,* and connections shall develop the *strength* of the connected members or the forces indicated above. The *deformation* of the *structure* shall not exceed the prescribed limits.

A continuous load path, or paths, with adequate *strength* and stiffness shall be provided to transfer all forces from the point of application to the final point of resistance. The foundation shall be designed to accommodate the forces developed or the movements imparted to the *structure* by the design ground motions. In the determination of the foundation design criteria, special recognition shall be given to the dynamic nature of the forces, the expected ground motions, and the design basis for *strength* and energy dissipation capacity of the *structure*.

5.2.2 Basic Seismic-Force-Resisting Systems: The basic lateral and vertical *seismic-force-resisting system* shall conform to one of the types indicated in Table 5.2.2 subject to the limitations on height based on *Seismic Design Category* indicated in the table. Each type is subdivided by the types of vertical element used to resist lateral *seismic forces*. The appropriate response modification coefficient, R, system overstrength factor, Ω_0, and deflection amplification factor, C_d, indicated in Table 5.2.2 shall be used in determining the *base shear*, element design forces, and design *story* drift as indicated in the *Provisions*.

Seismic-force-resisting systems that are not contained in Table 5.2.2 shall be permitted if analytical and test data are submitted that establish the dynamic characteristics and demonstrate the lateral force resistance and energy dissipation capacity to be equivalent to the structural systems listed in Table 5.2.2 for equivalent response modification coefficient, R, system overstrength coefficient, Ω_0, and deflection amplification factor, C_d, values.

Special framing requirements are indicated in Sec. 5.2.6 and in Chapters 8, 9, 10, 11, and 12 for *structures* assigned to the various *Seismic Design Categories*.

5.2.2.1 Dual System: For a dual system, the *moment frame* shall be capable of resisting at least 25 percent of the design forces. The total seismic force resistance is to be provided by the combination of the *moment frame* and the *shear walls* or *braced frames* in proportion to their rigidities.

5.2.2.2 Combinations of Framing Systems: Different *seismic-force-resisting systems* are permitted along the two orthogonal axes of the *structure*. Combinations of *seismic-force-resisting systems* shall comply with the requirements of this section.

5.2.2.2.1 R and Ω_0 Factors: The response modification coefficient, R, in the direction under consideration at any *story* shall not exceed the lowest response modification factor, R, for the *seismic-force-resisting system* in the same direction considered above that *story* excluding penthouses. For other than dual systems where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of R used in that direction shall not be greater than the least value of any of the systems utilized in the same direction. If a system other than a dual system with a response modification coefficient, R, with a value of less than 5 is used as part of the *seismic-force-resisting system* in any direction of the *structure*, the lowest such value shall be used for the entire *structure*. The system overstrength factor, Ω_0, in the direction under consideration at any *story* shall not be less than the largest value of this factor for the *seismic-force-resisting system* in the same direction considered above that *story*.

Exceptions:

1. Supported structural systems with a weight equal to or less than 10 percent of the weight of the *structure*.
2. Detached one- and two-family dwellings of light-frame construction.

5.2.2.2.2 Combination Framing Detailing Requirements: The detailing requirements of Sec. 5.2.6 required by the higher response modification coefficient, R, shall be used for structural *components* common to systems having different response modification coefficients.

5.2.2.3 Seismic Design Categories B and C: The structural framing system for *structures* assigned to *Seismic Design Categories* B and C shall comply with the *structure* height and structural limitations in Table 5.2.2.

Table 5.2.2 Design Coefficients and Factors for Basic *Seismic-Force-Resisting* Systems

Basic *Seismic-Force-Resisting System*	**Detailing Reference Section**	**Response Modification Coefficient, R^a**	**System Overstrength Factor, Ω_0 [g]**	**Deflection Amplification Factor, C_d^b**	**System Limitations and Building Height Limitations (ft) by Seismic Design Category[c]**				
					B	**C**	**D [d]**	**E [e]**	**F [e]**
Bearing Wall Systems									
Ordinary steel concentrically braced frames -- Light framed wall	8.6	4	2	3½	NL	NL	65	65	65
Special *reinforced concrete shear walls*	9.3.2.4	5	2½	5	NL	NL	160	160	100
Ordinary *reinforced concrete shear walls*	9.3.2.3	4	2½	4	NL	NL	NP	NP	NP
Detailed *plain concrete shear walls*	9.3.2.2	2½	2½	2	NL	NL	NP	NP	NP
Ordinary *plain concrete shear walls*	9.3.2.1	1½	2½	1½	NL	NP	NP	NP	NP
Special reinforced masonry *shear walls*	11.11.5	3½	2½	3½	NL	NL	160	160	100
Intermediate reinforced masonry *shear walls*	11.11.4	2½	2½	2¼	NL	NL	NP	NP	NP
Ordinary reinforced masonry *shear walls*	11.11.3	2	2½	1¾	NL	NP	NP	NP	NP
Detailed plain masonry *shear walls*	11.11.2	2	2½	1¾	NL	160	NP	NP	NP
Ordinary plain masonry *shear walls*	11.11.1	1½	2½	1¼	NL	NP	NP	NP	NP
Light frame walls with *shear panels*	8.6, 12.3.4, 12.4	6½	3	4	NL	NL	65	65	65
Building Frame Systems									
Steel *eccentrically braced frames*, moment resisting, connections at columns away from links	AISC Seismic, Part I, Sec. 15	8	2	4	NL	NL	160	160	100
Steel *eccentrically braced frames*, nonmoment resisting, connections at columns away from links	AISC Seismic, Part I, Sec. 15	7	2	4	NL	NL	160	160	100

Basic *Seismic-Force-Resisting System*	Detailing Reference Section	Response Modification Coefficient, R^a	System Overstrength Factor, $\Omega_0{}^g$	Deflection Amplification Factor, $C_d{}^b$	System Limitations and Building Height Limitations (ft) by Seismic Design Category[c]				
					B	C	D[d]	E[e]	F[e]
Special steel *concentrically braced frames*	AISC Seismic, Part I, Sec. 13	6	2	5	NL	NL	160	160	100
Ordinary steel *concentrically braced frames*	8.4.4; AISC Seismic	5	2	4½	NL	NL	35[k]	35[k]	NP[k]
Special *reinforced concrete shear walls*	9.3.2.4	6	2½	5	NL	NL	160	160	100
Ordinary *reinforced concrete shear walls*	9.3.2.3	5	2½	4½	NL	NL	NP	NP	NP
Detailed *plain concrete shear walls*	9.3.2.2	3	2½	2½	NL	NL	NP	NP	NP
Ordinary *plain concrete shear walls*	9.3.2.1	2	2½	2	NL	NP	NP	NP	NP
Composite *eccentrically braced frames*	AISC Seismic, Part II, Sec. 14	8	2	4	NL	NL	160	160	100
Composite *concentrically braced frames*	AISC Seismic, Part II, Sec. 13	5	2	4½	NL	NL	160	160	100
Ordinary composite *braced frames*	AISC Seismic, Part II, Sec. 12	3	2	3	NL	NL	NP	NP	NP
Composite steel plate *shear walls*	AISC Seismic, Part II, Sec. 17	6½	2½	5½	NL	NL	160	160	100
Special composite *reinforced concrete shear walls* with steel elements	AISC Seismic, Part II, Sec. 16	6	2½	5	NL	NL	160	160	100
Ordinary composite *reinforced concrete shear walls* with steel elements	AISC Seismic. Part II, Sec. 15	5	2½	4½	NL	NL	NP	NP	NP
Special reinforced masonry *shear walls*	11.11.5	4½	2½	4	NL	NL	160	160	100
Intermediate reinforced masonry *shear walls*	11.11.4	3	2½	2½	NL	NL	NP	NP	NP
Ordinary reinforced masonry *shear walls*	11.11.3	2½	2½	2¼	NL	NP	NP	NP	NP

Basic *Seismic-Force-Resisting System*	Detailing Reference Section	Response Modification Coefficient, R^a	System Overstrength Factor, Ω_0 [g]	Deflection Amplification Factor, C_d^b	System Limitations and Building Height Limitations (ft) by Seismic Design Category[c]				
					B	C	D [d]	E [e]	F [e]
Detailed plain masonry *shear walls*	11.11.2	2½	2½	2¼	NL	160	NP	NP	NP
Ordinary plain masonry *shear walls*	11.11.1	1½	2½	1¼	NL	NP	NP	NP	NP
Light frame walls with *shear panels*	8.6, 12.3.4, 12.4	7	2½	4½	NL	NL	160	160	160
Moment Resisting Frame Systems									
Special steel *moment frames*	AISC Seismic, Part I, Sec. 9	8	3	5½	NL	NL	NL	NL	NL
Special steel truss *moment frames*	AISC Seismic, Part I, Sec. 12	7	3	5½	NL	NL	160	100	NP
Intermediate steel *moment frames*	AISC Seismic, Part I, Sec. 10	4½.	3	4	NL	NL	35[i]	NP[i,j]	NP[i,j]
Ordinary steel *moment frames*	AISC Seismic, Part I, Sec. 11	3½	3	3	NL	NL	NP[i,j]	NP [i,j]	NP [i,j]
Special reinforced concrete moment frames	9.3.1.3	8	3	5½	NL	NL	NL	NL	NL
Intermediate reinforced concrete moment frames	9.3.1.2	5	3	4½	NL	NL	NP	NP	NP
Ordinary reinforced concrete moment frames	9.3.1.1	3	3	2½	NL	NP	NP	NP	NP
Special composite *moment frames*	AISC Seismic, Part II, Sec. 9	8	3	5½	NL	NL	NL	NL	NL
Intermediate composite *moment frames*	AISC Seismic, Part II, Sec. 10	5	3	4½	NL	NL	NP	NP	NP
Composite partially restrained *moment frames*	AISC Seismic, Part II, Sec. 8	6	3	5½	160	160	100	NP	NP
Ordinary composite *moment frames*	AISC Seismic, Part II, Sec. 11	3	3	2½	NL	NP	NP	NP	NP

Basic *Seismic-Force-Resisting System*	Detailing Reference Section	Response Modification Coefficient, R^a	System Overstrength Factor, Ω_0 [g]	Deflection Amplification Factor, C_d^b	System Limitations and Building Height Limitations (ft) by Seismic Design Category[c]				
					B	C	D [d]	E [e]	F [e]
Special masonry moment frames	11.2	5½	3	5	NL	NL	160	160	100
Dual Systems with *Special Moment Frames* Capable of Resisting at Least 25% of Prescribed *Seismic Forces*									
Steel *eccentrically braced frames*, moment resisting connections, at columns away from links	AISC Seismic,Part I, Sec. 15	8	2½	4	NL	NL	NL	NL	NL
Steel *eccentrically braced frames*, non-moment resisting connections, at columns away from links	AISC Seismic, Part I, Sec. 15	7	2½	4	NL	NL	NL	NL	NL
Special steel *concentrically braced frames*	AISC Seismic, Part I, Sec. 13	8	2½	6½	NL	NL	NL	NL	NL
Special *reinforced concrete shear walls*	9.3.2.4	8	2½	6½	NL	NL	NL	NL	NL
Ordinary *reinforced concrete shear walls*	9.3.2.3	7	2½	6	NL	NL	NP	NP	NP
Composite *eccentrically braced frames*	AISC Seismic, Part II, Sec. 14	8	2½	4	NL	NL	NL	NL	NL
Composite *concentrically braced frames*	AISC Seismic, Part II, Sec. 13	6	2½	5	NL	NL	NL	NL	NL
Composite steel plate *shear walls*	AISC Seismic, Part II, Sec. 17	8	2½	6½	NL	NL	NL	NL	NL
Special composite *reinforced concrete shear walls* with steel elements	AISC Seismic, Part II, Sec. 16	8	2½	6½	NL	NL	NL	NL	NL
Ordinary composite *reinforced concrete shear walls* with steel elements	AISC Seismic, Part II, Sec. 15	7	2½	6	NL	NL	NP	NP	NP
Special reinforced masonry *shear walls*	11.11.5	7	3	6½	NL	NL	NL	NL	NL
Intermediate reinforced masonry *shear walls*	11.11.4	6½	3	5½	NL	NL	NL	NP	NP
Dual Systems with *Intermediate Moment Frames* Capable of Resisting at Least 25% of Prescribed *Seismic Forces*									

Basic *Seismic-Force-Resisting System*	Detailing Reference Section	Response Modification Coefficient, R^a	System Overstrength Factor, Ω_0 [g]	Deflection Amplification Factor, C_d^b	System Limitations and Building Height Limitations (ft) by Seismic Design Category[c]				
					B	C	D [d]	E [e]	F [e]
Special steel *concentrically braced frames* [f]	AISC Seismic, Part I, Sec. 13	4½	2½	4	NL	NL	35[i]	NP[i,j]	NP[i,j]
Special *reinforced concrete shear walls*	9.3.2.4	6	2½	5	NL	NL	160	100	100
Ordinary *reinforced concrete shear walls*	9.3.2.3	5½	2½	4½	NL	NL	NP	NP	NP
Ordinary reinforced masonry *shear walls*	11.11.3	3	3	2½	NL	160	NP	NP	NP
Intermediate reinforced masonry *shear walls*	11.11.4	5	3	4½	NL	NL	160	NP	NP
Composite *concentrically braced frames*	AISC Seismic, Part II, Sec. 13	5	2½	4½	NL	NL	160	100	NP
Ordinary composite *braced frames*	AISC Seismic, Part II, Sec. 12	4	2½	3	NL	NL	NP	NP	NP
Ordinary composite *reinforced concrete shear walls* with steel elements	AISC Seismic, Part II, Sec. 15	5½	2½	4½	NL	NL	NP	NP	NP
Inverted Pendulum Systems and Cantilevered Column Systems									
Special steel *moment frames*	AISC Seismic, Part I, Sec. 9	2½	2	2½	NL	NL	NL	NL	NL
Ordinary steel *moment frames*	AISC Seismic, Part I, Sec. 11	1¼	2	2½	NL	NL	NP	NP	NP
Special reinforced concrete moment frames	9.3.1.3	2½	2	1¼	NL	NL	NL	NL	NL
Structural Steel Systems Not Specifically Detailed for Seismic Resistance	AISC-ASD, AISC-LRFD, AISI	3	3	3	NL	NL	NP	NP	NP

NOTES FOR TABLE 5.2.2

[a] Response modification coefficient, *R*, for use throughout the *Provisions*.

[b] Deflection amplification factor, C_d, for use in Sec. 5.4.6.1 and 5.4.6.2.

[c] NL = not limited and NP = not permitted. If using metric units, 100 ft approximately equals 30 m and 160 ft approximately equals 50 m. Heights are measured from the base of the structure as defined in Sec. 2.1.

[d] See Sec. 5.2.2.4.1 for a description of *building* systems limited to *buildings* with a height of 240 ft (70 m) or less.

[e] See Sec. 5.2.2.5 for *building* systems limited to *buildings* with a height of 160 ft (50 m) or less.

[f] An *ordinary moment frame* is permitted to be used in lieu of an *Intermediate moment frame* in *Seismic Design Categories* B and C.

[g] The tabulated value of the *overstrength factor*, Ω_0, may be reduced by subtracting ½ for structures with flexible *diaphragms* but shall not be taken as less than 2 for any structure.

[i] Steel *ordinary moment frames* and *intermediate moment frames* are permitted in *single-story buildings* up to a height of 60 ft when the moment *joints* of field connections are constructed of bolted end plates and the *dead load* of the roof does not exceed 15 psf.

[j] Steel *ordinary moment frames* are permitted in *buildings* up to a height of 35 ft where the *dead load* of the walls, floors, and roofs does not exceed 15 psf.

[k] *Steel ordinary braced frames* are permitted in single-story buildings up to a height of 60 ft when the dead load of the roof does not exceed 15 psf and in penthouse structures.

5.2.2.4 Seismic Design Categories D and E: The structural framing system for a *structure* assigned to *Seismic Design Categories* D and E shall comply with Sec. 5.2.2.3 and the additional requirements of this section.

5.2.2.4.1 Limited Building Height: The height limit in Table 5.2.2 is permitted to be increased to 240 ft (70 m) in *buildings* that have steel *braced frames* or concrete cast-in-place *shear walls*. Such *buildings* shall be configured such that the *braced frames* or *shear walls* arranged in any one plane conform to the following:

1. The *braced frames* or cast-in-place special reinforced concrete *shear walls* in any one plane shall resist no more than 60 percent of the total *seismic forces* in each direction, neglecting torsional effects, and
2. The seismic force in any *braced frame* or *shear wall* resulting from torsional effects shall not exceed 20 percent of the total seismic force in that *braced frame* or *shear wall*.

5.2.2.4.2 Interaction Effects: *Moment frames* that are enclosed or adjoined by more rigid elements not considered to be part of the *seismic-force-resisting system* shall be designed so that the action or failure of those elements will not impair the vertical load and *seismic-force*-resisting capability of the frame. The design shall consider and provide for the effect of these rigid elements on the structural system at *structure* deformations corresponding to the design *story* drift, Δ, as determined in Sec. 5.4.6. In addition, the effects of these elements shall be considered when determining whether a *structure* has one or more of the irregularities defined in Sec. 5.2.3.

5.2.2.4.3 Deformational Compatibility: Every structural *component* not included in the *seismic-force-resisting system* in the direction under consideration shall be designed to be adequate for the vertical load-carrying capacity and the induced moments and shears resulting from the design *story* drift, Δ, as determined in accordance with Sec. 5.4.6 (also see Sec. 5.2.7).

> **Exception:** Beams and columns and their connections not designed as part of the lateral-force-resisting system but meeting the detailing requirements for either *intermediate moment frames* or *special moment frames* are permitted to be designed to be adequate for the vertical load-carrying capacity and the induced moments and shears resulting from the deformation of the *building* under the application of the design *seismic forces*.

When determining the moments and shears induced in *components* that are not included in the *seismic-force-resisting system* in the direction under consideration, the stiffening effects of adjoining rigid structural and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used.

5.2.2.4.4 Special Moment Frames: A *special moment frame* that is used but not required by Table 5.2.2 is permitted to be discontinued and supported by a more rigid system with a lower response modification coefficient, R, provided the requirements of Sec. 5.2.6.2.3 and 5.2.6.4.2 are met. Where a *special moment frame* is required by Table 5.2.2, the frame shall be continuous to the foundation.

5.2.2.5 Seismic Design Category F: The framing systems of *buildings* assigned to *Seismic Design Category* F shall conform to the requirements of Sec. 5.2.2.4 for *Seismic Design Categories* D and E and to the additional requirements and limitations of this section. The height limitation of Sec. 5.2.2.4.1 shall be reduced from 240 ft to 160 ft (70 to 50 m).

5.2.3 Structure Configuration: *Structures* shall be classified as regular or irregular based upon the criteria in this section. Such classification shall be based on the plan and vertical configuration.

5.2.3.1 Diaphragm Flexibility: *Diaphragms* constructed of untopped steel decking, *wood structural panels,* or similar panelized construction shall be considered flexible in *structures* having concrete or masonry *shear walls*. *Diaphragms* constructed of *wood structural panels* shall be considered rigid in light-frame *structures* using structural panels for lateral load resistance. *Diaphragms* of other types shall be considered flexible when the maximum lateral *deformation* of the *diaphragm* is more than two times the average *story* drift of the associated *story*. The loadings used for this calculation shall be those prescribed by Sec. 5.4

5.2.3.2 Plan Irregularity: *Structures* having one or more of the features listed in Table 5.2.3.2 shall be designated as having plan structural irregularity and shall comply with the requirements in the sections referenced in Table 5.2.3.2.

5.2.3.3 Vertical Irregularity: *Structures* having one or more of the features listed in Table 5.2.3.3 shall be designated as having vertical irregularity and shall comply with the requirements in the sections referenced in Table 5.2.3.3.

Exceptions:

1. Structural irregularities of Types 1a, 1b, or 2 in Table 5.2.3.3 do not apply where no *story drift ratio* under design lateral load is greater than 130 percent of the *story drift ratio* of the *story* immediately above. Torsional effects need not be considered in the calculation of *story* drifts for the purpose of this determination. The *story drift ratio* relationship for the top two *stories* of the *structure* are not required to be evaluated.

2. Irregularities Types 1a, 1b, and 2 of Table 5.2.3.3 are not required to be considered for one-*story structures* or for two-*story structures* in *Seismic Design Categories* A, B, C, or D.

TABLE 5.2.3.2 Plan Structural Irregularities

	Irregularity Type and Description	Reference Section	Seismic Design Category Application
1a	**Torsional Irregularity – to be considered when diaphragms are not flexible** Torsional irregularity shall be considered to exist when the maximum *story* drift, computed including accidental torsion, at one end of the *structure* transverse to an axis is more than 1.2 times the average of the *story* drifts at the two ends of the *structure*.	5.2.6.4.2 5.4.4	D, E, and F C, D, E, and F
1b	**Extreme Torsional Irregularity -- to be considered when diaphragms are not flexible** Extreme torsional irregularity shall be considered to exist when the maximum *story* drift, computed including accidental torsion, at one end of the *structure* transverse to an axis is more than 1.4 times the average of the *story* drifts at the two ends of the *structure*.	5.2.6.4.2 5.4.4 5.2.6.5.1	D, E, and F C, D, E, and F E and F
2	**Re-entrant Corners** Plan configurations of a *structure* and its lateral-force-resisting system contain re-entrant corners where both projections of the *structure* beyond a re-entrant corner are greater than 15 percent of the plan dimension of the *structure* in the given direction.	5.2.6.4.2	D, E, and F
3	**Diaphragm Discontinuity** Diaphragms with abrupt discontinuities or variations in stiffness including those having cutout or open areas greater than 50 percent of the gross enclosed diaphragm area or changes in effective diaphragm stiffness of more than 50 percent from one *story* to the next.	5.2.6.4.2	D, E, and F
4	**Out-of-Plane Offsets** Discontinuities in a lateral force resistance path such as out-of-plane offsets of the vertical elements.	5.2.6.4.2 5.2.6.2.10	D, E, and F B, C, D, E, and F
5	**Nonparallel Systems** The vertical lateral-force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force-resisting system.	5.2.5.2	C, D, E, and F

TABLE 5.2.3.3 Vertical Structural Irregularities

Irregularity Type and Description		Reference Section	Seismic Design Category Application
1a	**Stiffness Irregularity – Soft Story** A soft *story* is one in which the lateral stiffness is less than 70 percent of that in the *story* above or less than 80 percent of the average stiffness of the three stories above.	5.2.5.1	D, E, and F
1b	**Stiffness Irregularity--Extreme Soft Story** An extreme soft *story* is one in which the lateral stiffness is less than 60 percent of that in the *story* above or less than 70 percent of the average stiffness of the three stories above.	5.2.5.1 5.2.6.5.1	D, E, and F E and F
2	**Weight (Mass) Irregularity** Mass irregularity shall be considered to exist where the effective mass of any *story* is more than 150 percent of the effective mass of an adjacent *story*. A roof that is lighter than the floor below need not be considered.	5.2.5.1	D, E, and F
3	**Vertical Geometric Irregularity** Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any *story* is more than 130 percent of that in an adjacent *story*.	5.2.5.1	D, E, and F
4	**In-Plane Discontinuity in Vertical Lateral-Force Resisting Elements** An in-plane offset of the lateral-force-resisting elements greater than the length of those elements or a reduction in stiffness of the resisting element in the *story* below.	5.2.5.1 5.2.6.2.10 5.2.6.4.2	D, E, and F B, C, D, E, and F D, E, and F
5	**Discontinuity in Capacity – Weak Story** A weak *story* is one in which the *story* lateral *strength* is less than 80 percent of that in the *story* above. The *story strength* is the total *strength* of all seismic-resisting elements sharing the *story* shear for the direction under consideration.	5.2.6.2.3 5.2.5.1 5.2.6.5.1	B, C, D, E, and F D, E, and F E and F

5.2.4 Redundancy: A reliability factor, ρ, shall be assigned to all *structures* based on the extent of structural redundancy inherent in the lateral-force-resisting system.

5.2.4.1 Seismic Design Categories A, B, and C: For *structures* in *Seismic Design Categories* A, B and C, the value of ρ may be taken as 1.0.

5.2.4.2 Seismic Design Category D: For *structures* in *Seismic Design Category* D, ρ shall be taken as the largest of the values of ρ_x calculated at each *story* of the *structure* "x" in accordance with Eq. 5.2.4.2:

$$\rho_x = 2 - \frac{20}{r_{\max_x}\sqrt{A_x}} \tag{5.2.4.2}$$

where:

r_{max_x} = the ratio of the design *story shear* resisted by the single element carrying the most shear force in the *story* to the total *story shear* for a given direction of loading. For *braced frames*, the value of r_{max_x} is equal to the lateral force *component* in the most heavily loaded brace element divided by the *story shear*. For *moment frames*, r_{max_x} shall be taken as the maximum of the sum of the shears in any two adjacent columns in the plane of a *moment frame* divided by the *story shear*. For columns common to two bays with moment resisting connections on opposite sides at the level under consideration, 70 percent of the shear in that column may be used in the column shear summation. For *shear walls*, r_{max_x} shall be taken equal to the maximum ratio, r_{ix}, calculated as the shear in each *wall* or *wall* pier multiplied by $10/l_w$ (the metric coefficient is $3.3/l_w$), where l_w is the *wall* or *wall* pier length in feet (m) divided by the *story* shear and where the ratio $10/l_w$ need not be taken greater than 1.0 for buildings of light frame construction. For dual systems, r_{max_x} shall be taken as the maximum value as defined above considering all lateral-load-resisting elements in the *story*. The lateral loads shall be distributed to elements based on relative rigidities considering the interaction of the dual system. For dual systems, the value of ρ need not exceed 80 percent of the value calculated above.

A_x = the floor area in square feet of the *diaphragm* level immediately above the *story*.

The value of ρ need not exceed 1.5, which is permitted to be used for any *structure*. The value of ρ shall not be taken as less than 1.0.

> **Exception**: For *structures* with lateral-force-resisting systems in any direction comprised solely of *special moment frames*, the lateral-force-resisting system shall be configured such that the value of ρ calculated in accordance with this section does not exceed 1.25.

The metric equivalent of Eq. 5.2.4.2 is:

$$\rho_x = 2 - \frac{6.1}{r_{\max_x}\sqrt{A_x}}$$

where A_x is in square meters.

5.2.4.3 Seismic Design Categories E and F: For *structures* in *Seismic Design Categories* E and F, the value of ρ shall be calculated as indicated in Section 5.2.4.2, above.

> **Exception:** For *structures* with lateral-force-resisting systems in any direction comprised solely of *special moment frames*, the lateral-force-resisting system shall be configured such that the value of ρ calculated in accordance with Sec. 5.2.4.2 does not exceed 1.1.

5.2.5 Structural Analysis: A structural analysis conforming to one of the types permitted in Section 5.2.5.1 shall be made for all *structures*. Application of loading shall be as indicated in Sec. 5.2.5.2 and as required by the selected analysis procedure. All members of the *structure's seismic-force-resisting system* and their connections shall have adequate strength to resist the forces, Q_E, predicted by the analysis in combination with other loads as required by Sec. 5.2.7. Drifts predicted by the analysis shall be within the limits specified by Sec. 5.2.8. If a nonlinear analysis is performed, component deformation demands shall not exceed limiting values as indicated in Sec. 5.7.3.2.

> **Exception:** For *structures* in *Seismic Design Category* A, drift need not be evaluated.

5.2.5.1 Analysis Procedures: The structural analysis required by Sec. 5.2.5 shall consist of one of the types permitted in Table 5.2.5.1 based on the *structure*'s *Seismic Design Category*, structural system, dynamic properties, and regularity or, with the approval of the authority having jurisdiction, an alternative generally accepted procedure shall be permitted to be used.

5.2.5.2 Application of Loading: The directions of application of *seismic forces* used in the design shall be those that will produce the most critical load effects. It shall be permitted to satisfy this requirement using the procedures of Sec. 5.2.5.2.1 for *Seismic Design Category* A or B, Sec. 5.2.5.2.2 for *Seismic Design Category* C, and Sec. 5.2.5.2.3 for *Seismic Design Category* D, E, or F.

5.2.5.2.1 Seismic Design Category A or B: For *structures* assigned to *Seismic Design Category* A or B, the design seismic forces are permitted to be applied separately in each of two orthogonal directions and orthogonal interaction effects may be neglected.

TABLE 5.2.5.1 Permitted Analytical Procedures

Seismic Design Category	**Structural Characteristics**	**Index Force Analysis, Sec. 5.3**	**Equivalent Lateral Force Analysis, Sec. 5.4**	**Modal Response Spectrum Analysis, Sec. 5.5**	**Linear Response History Analysis, Sec. 5.6**	**Nonlinear Response History Analysis, Sec. 5.7**
A	Regular or irregular	P	P	P	P	P
B, C	Regular or irregular	NP	P	P	P	P
D, E, F	Regular *structures* with $T < 3.5T_s$ and all *structures* of light frame construction	NP	P	P	P	P
	Irregular *structures* with $T < 3.5T_s$ and having only plan irregularities Type 2, 3, 4, or 5 Table 5.2.3.2 or vertical irregularities Type 4 or 5 of Table 5.2.3.3.	NP	P	P	P	P
	Irregular *structures* with $T < 3.5T_s$ and having either plan irregularities Type 1a or 1b of Table 5.2.3.2 or vertical irregularities Type 1a or 1b, 2, or 3 of Table 5.2.3.3.	NP	NP	P	P	P
	All other structures	NP	NP	P	P	P

Notes: P indicates permitted; NP indicates not permitted.

5.2.5.2.2 Seismic Design Category C: Loading applied to *structures* assigned to *Seismic Design Category* C shall, as a minimum, conform to the requirements of Sec. 5.2.5.2.1 for *Seismic Design Categories* A and B and the requirements of this section. *Structures* that have plan structural irregularity Type 5 in Table 5.2.3.2 shall be analyzed for *seismic forces* using a three-dimensional representation and either of the following procedures:

a. The *structure* shall be analyzed using the equivalent lateral force analysis procedure of Sec. 5.4, the modal response spectrum analysis procedure of Sec. 5.5, or the linear response history analysis procedure of Sec. 5.6 as permitted under Sec. 5.2.5.1 with the loading applied independently in any two orthogonal directions. The most critical load effect due to direction of application of *seismic forces* on the *structure* may be assumed to be satisfied if *components* and their foundations are designed for the following combination of prescribed loads: 100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction; the combination requiring the maximum *component strength* shall be used.

b. The *structure* shall be analyzed using the linear response history analysis procedure of Sec. 5.6 or the nonlinear response history analysis procedure of Sec. 5.7 as permitted by Sec. 5.2.5.1 with simultaneous application of ground motion in each of two orthogonal directions.

5.2.5.2.3 Seismic Design Category D, E, or F: *Structures* assigned to *Seismic Design Category* D, E, or F shall be designed for the most critical load effect due to application of *seismic forces* in any direction. Either of the procedures of Sec. 5.2.5.2.2 shall be permitted to be used to satisfy this requirement. Two-dimensional analysis shall be permitted to be used where diaphragms are flexible and the structure does not have plan structural irregularity Type 5 of Table 5.2.3.2.

5.2.6 Design and Detailing Requirements: The design and detailing of the *components* of the *seismic-force-resisting system* shall comply with the requirements of this section. Foundation design shall conform to the applicable requirements of Chapter 7. The materials and the systems composed of those materials shall conform to the requirements and limitations of Chapters 8 through 12 for the applicable category.

5.2.6.1 Seismic Design Category A: The design and detailing of *structures* assigned to *Seismic Design Category* A shall comply with the requirements of this section.

5.2.6.1.1 Connections: All parts of the *structure* between separation *joints* shall be interconnected, and the connections shall be capable of transmitting the *seismic force*, F_p, induced by the parts being connected. Any smaller portion of the *structure* shall be tied to the remainder of the *structure* with elements having a *strength* of 0.133 times the short period design spectral response acceleration coefficient, S_{DS}, times the weight of the smaller portion or 5 percent of the portion's weight, whichever is greater.

A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder, or truss to its support. The connection shall have a minimum *strength* of 5 percent of the *dead load* and *live load* reaction.

5.2.6.1.2 Anchorage of Concrete or Masonry Walls: Concrete and masonry *walls* shall be anchored to the roof and all floors and to members that provide lateral support for the *wall* or which are supported by the *wall*. The anchorage shall provide a direct connection between the *walls* and the roof or floor construction. The connections shall be capable of resisting a seismic lateral force, F_p, induced by the *wall* of 400 times the short period design spectral response acceleration coefficient, S_{DS}, in pounds per lineal ft (5840 times S_{DS} in N/m) of *wall* multiplied by the *occupancy importance factor, I*. *Walls* shall be designed to resist bending between anchors where the anchor spacing exceeds 4 ft (1.2 m).

5.2.6.2 Seismic Design Category B: *Structures* assigned to *Seismic Design Category* B shall conform to the requirements of Sec. 5.2.6.1 for *Seismic Design Category* A and the requirements of this section.

5.2.6.2.1 P-Delta Effects: *P-delta effects* shall be included as required by Sec. 5.4.6.2

5.2.6.2.2 Openings: Where openings occur in *shear walls*, *diaphragms* or other plate-type elements, reinforcement at the edges of the openings shall be designed to transfer the stresses into the structure. The edge reinforcement shall extend into the body of the *wall* or *diaphragm* a distance sufficient to develop the force in the reinforcement.

5.2.6.2.3 Discontinuities in Vertical System: *Structures* with a discontinuity in lateral capacity, vertical irregularity Type 5 as defined in Table 5.2.3.3, shall not be over 2 stories or 30 ft (9 m) in height where the "weak" *story* has a calculated *strength* of less than 65 percent of the strength of the *story* above.

> **Exception:** The height limitation shall not apply when the "weak" *story* is capable of resisting a total *seismic force* equal to 75 percent of the deflection amplification factor, C_d, times the design force prescribed in Sec. 5.3.

5.2.6.2.4 Nonredundant Systems: The design of a *structure* shall consider the potentially adverse effect that the failure of a single member, connection, or *component* of the *seismic-force-resisting system* would have on the stability of the *structure*.

5.2.6.2.5 Collector Elements: Collector elements shall be provided that are capable of transferring the *seismic forces* originating in other portions of the *structure* to the element providing the resistance to those forces.

5.2.6.2.6 Diaphragms: The deflection in the plane of the *diaphragm*, as determined by engineering analysis, shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be a deflection that permits the attached element to maintain its structural integrity under the individual loading and to continue to support the prescribed loads.

Floor and roof *diaphragms* shall be designed to resist the following *seismic forces*: A minimum force equal to 20 percent of the short period design spectral response acceleration, S_{DS}, times the weight of the *diaphragm* and other elements of the *structure* attached thereto plus the portion of the seismic shear force at that level, V_x, required to be transferred to the *components* of the vertical *seismic-force-resisting system* because of offsets or changes in stiffness of the vertical *components* above and below the *diaphragm*.

Diaphragms shall provide for both the shear and bending stresses resulting from these forces. *Diaphragms* shall have ties or struts to distribute the *wall* anchorage forces into the *diaphragm*. *Diaphragm* connections shall be positive, mechanical, or welded type connections.

5.2.6.2.7 Bearing Walls: Exterior and interior *bearing walls* and their anchorage shall be designed for a force equal to 40 percent of the short period design spectral response acceleration, S_{DS}, times the weight of *wall*, W_c, normal to the surface, with a minimum force of 10 percent of the weight of the *wall*. Interconnection of *wall* elements and connections to supporting framing systems shall have sufficient ductility, rotational capacity, or *strength* to resist shrinkage, thermal changes, and differential foundation settlement when combined with *seismic forces*.

5.2.6.2.8 Inverted Pendulum-Type Structures: Supporting columns or piers of *inverted pendulum-type structures* shall be designed for the bending moment calculated at the *base* determined using the procedures given in Sec. 5.3 and varying uniformly to a moment at the top equal to one-half the calculated bending moment at the *base*.

5.2.6.2.9 Anchorage of Nonstructural Systems: When required by Chapter 6, all portions or *components* of the *structure* shall be anchored for the *seismic force*, F_p, prescribed therein.

5.2.6.2.10 Columns Supporting Discontinuous Walls or Frames: Columns supporting discontinuous *walls* or frames of *structures* having plan irregularity Type 4 of Table 5.2.3.2 or vertical irregularity Type 4 of Table 5.2.3.3 shall have the *design strength* to resist the maximum axial force that can develop in accordance with the special combination of loads of Sec. 5.2.7.1.

5.2.6.3 Seismic Design Category C: *Structures* assigned to *Seismic Design Category* C shall conform to the requirements of Sec. 5.2.6.2 for *Seismic Design Category* B and to the requirements of this section.

5.2.6.3.1 Collector Elements: Collector elements shall be provided that are capable of transferring the *seismic forces* originating in other portions of the *structure* to the element providing the resistance to those forces. Collector elements, splices, and their connections to resisting elements shall resist the of Sec. 5.2.7.1.

> **Exception:** In *structures* or portions thereof braced entirely by light frame *shear walls*, collector elements, splices, and connections to resisting elements are permitted to be designed to resist forces in accordance with Eq. 5.2.6.4.4.

The quantity $\Omega_0 E$ in Eq. 5.2.7.1-1 need not exceed the maximum force that can be transferred to the collector by the *diaphragm* and other elements of the lateral-force-resisting system.

5.2.6.3.2 Anchorage of Concrete or Masonry Walls: Concrete or masonry *walls* shall be anchored to all floors, roofs, and members that provide out-of-plane lateral support for the *wall* or that are supported by the *wall*. The anchorage shall provide a positive direct connection between the *wall* and the floor, roof, or supporting member capable of resisting the horizontal forces specified in this section for *structures* with flexible *diaphragms* or of Sec. 6.1.3 for *structures* with *diaphragms* that are not flexible.

Anchorage of *walls* to flexible *diaphragms* shall have the *strength* to develop the out-of-plane force given by Eq. 5.2.6.3.2:

$$F_p = 1.2 S_{DS} I W_p \qquad (5.2.6.3.2)$$

where:

F_p = the design force in the individual anchors,

S_{DS} = the design spectral response acceleration at short periods in accordance with Sec. 4.1.2.5,

I = the *occupancy importance factor* in accordance with Sec. 1.4, and

W_p = the weight of the *wall* tributary to the anchor.

Diaphragms shall be provided with continuous ties or struts between *diaphragm* chords to distribute these anchorage forces into the *diaphragms*. Added chords are permitted to be used to form *subdiaphragms* to transmit the anchorage forces to the main continuous cross-ties. The maximum length to width ratio of the structural *subdiaphragm* shall be 2-1/2 to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the *diaphragm* and the attached *components*. Connections shall extend into the *diaphragm* a sufficient distance to develop the force transferred into the *diaphragm*.

In wood *diaphragms*, the continuous ties shall be in addition to the diaphragm sheathing. Anchorage shall not be accomplished by use of toe nails or nails subject to withdrawal nor shall wood ledgers of framing be used in cross-grain bending or cross-grain tension. The *diaphragm* sheathing shall not be considered as effectively providing the ties or struts required by this section.

In metal deck *diaphragms*, the metal deck shall not be used as the continuous ties required by this section in the direction perpendicular to the deck span.

Diaphragm-to-*wall* anchorage using embedded straps shall be attached to or hooked around the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

5.2.6.4 Seismic Design Category D: *Structures* assigned to *Seismic Design Category* D shall conform to the requirements of Sec. 5.2.6.3 for *Seismic Design Category* C and to the requirements of this section.

5.2.6.4.1 Collector Elements: Collector elements shall be provided that are capable of transferring the *seismic forces* originating in other portions of the *structure* to the element providing the resistance to those forces. Collector elements, splices, and their connections to resisting elements shall resist the forces determined in accordance with Eq. 5.2.6.4.4. In addition, collector elements, splices, and their connections to resisting elements shall have the *design strength* to resist the earthquake loads defined in the special load combination of Sec. 5.2.7.1.

> **Exception:** In *structures* or portions thereof braced entirely by light *shear walls*, collector elements, splices, and connections to resisting elements are permitted to be designed to resist forces in accordance with Eq. 5.2.5.4.

The quantity $\Omega_0 E$ in Eq. 5.2.7.1-1 need not exceed the maximum force that can be transferred to the collector by the *diaphragm* and other elements of the lateral-force-resisting system.

5.2.6.4.2 Plan or Vertical Irregularities: The design shall consider the potential for adverse effects when the ratio of the *strength* provided in any *story* to the *strength* required is significantly less than that ratio for the *story* immediately above and the *strengths* shall be adjusted to compensate for this effect.

For *structures* having a plan structural irregularity of Type 1a, 1b, 2, 3, or 4 in Table 5.2.3.2 or a vertical structural irregularity of Type 4 in Table 5.2.3.3, the design forces determined from Sec. 5.4.1 shall be increased 25 percent for connections of *diaphragms* to vertical elements and to collectors and for connections of collectors to the vertical elements. Collectors also shall be designed for these increased forces unless subject to the requirements of Sec. 5.2.6.4.1 or Sec. 8.6.2.

5.2.6.4.3 Vertical Seismic Forces: The vertical *component* of earthquake ground motion shall be considered in the design of horizontal cantilever and horizontal prestressed *components*. The load combinations used in evaluating such *components* shall include *E* as defined by Eq. 5.2.7-1 and 5.2.7-2. Horizontal cantilever structural *components* shall be designed for a minimum net upward force of 0.2 times the *dead load* in addition to the applicable load combinations of Sec. 5.2.7.

5.2.6.4.4 Diaphragms: *Diaphragms* shall be designed to resist design *seismic forces* determined in accordance with Eq. 5.2.6.4.4 as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad (5.2.6.4.4)$$

where:

F_{px} = the *diaphragm* design force,

F_i = the design force applied to Level *i*,

w_i = the weight tributary to Level *I*, and

w_{px} = the weight tributary to the *diaphragm* at Level *x*.

The force determined from Eq. 5.2.6.4.4 need not exceed $0.4S_{DS}Iw_{px}$ but shall not be less than $0.2S_{DS}Iw_{px}$. When the *diaphragm* is required to transfer design *seismic forces* from the vertical-resisting elements above the *diaphragm* to other vertical-resisting elements below the *diaphragm* due to offsets in the placement of the elements or to changes in relative lateral stiffness in the vertical elements, these forces shall be added to those determined from Eq. 5.2.6.4.4.

5.2.6.5 Seismic Design Categories E and F: *Structures* assigned to *Seismic Design Categories* E and F shall conform to the requirements of Sec. 5.2.6.4 for *Seismic Design Category* D and to the requirements of this section.

5.2.6.5.1 Plan or Vertical Irregularities: *Structures* having plan irregularity Type 1b of Table 5.2.3.2 or vertical irregularities Type 1b or 5 of Table 5.2.3.3 shall not be permitted.

5.2.7 Combination of Load Effects: The effects on the *structure* and its *components* due to *gravity loads* and *seismic forces* shall be combined in accordance with the factored load combinations as presented in ASCE7- 98 except that the effect of seismic loads, *E*, shall be as defined herein.

The effect of seismic load *E* shall be defined by Eq. 5.2.7-1 as follows for load combinations in which the effects of *gravity loads* and seismic loads are additive:

$$E = \rho Q_E + 0.2 S_{DS} D \qquad (5.2.7\text{-}1)$$

where:

E = the effect of horizontal and vertical earthquake-induced forces,

S_{DS} = the design spectral response acceleration at short periods obtained from Sec. 4.1.2.5.

D = the effect of dead load,

ρ = the reliability factor, and

Q_E = the effect of horizontal *seismic forces*.

The effect of seismic load *E* shall be defined by Eq. 5.2.7-2 as follows for load combinations in which the effects of gravity counteract seismic load:

$$E = \rho Q_E - 0.2 S_{DS} D \qquad (5.2.7\text{-}2)$$

where E, ρ, Q_E, S_{DS}, and D are as defined above.

5.2.7.1 Special Combination of Loads: When specifically required by the *Provisions*, the design *seismic force* on *components* sensitive to the effects of structural overstrength shall be as defined by Eq. 5.2.7.1-1 and 5.2.7.1-2 when seismic load is, respectively, additive or counteractive to the gravity forces as follows:

$$E = \Omega_0 Q_E + 0.2 S_{DS} D \qquad (5.2.7.1\text{-}1)$$

$$E = \Omega_0 Q_E - 0.2 S_{DS} D \qquad (5.2.7.1\text{-}2)$$

where E, Q_E, S_{DS}, and D are as defined above and Ω_0 is the system overstrength factor as given in Table 5.2.2. The term $\Omega_0 Q_E$ calculated in accordance with Eq. 5.2.7.1-1 and 5.2.7.1-2 need not exceed the maximum force that can develop in the element as determined by a rational plastic mechanism analysis or nonlinear response analysis utilizing realistic expected values of material *strengths*.

Exception: The special load combination of Eq. 5.2.7.1-1 need not apply to the design of *components* in *structures* in *Seismic Design Category* A.

5.2.8 Deflection and Drift Limits: The design *story* drift, Δ, as determined in Sec. 5.3.7 or 5.4.6 shall not exceed the allowable *story* drift, Δ_a, as obtained from Table 5.2.8 for any *story*. For *structures* with significant torsional deflections, the maximum drift shall include torsional effects. All portions of the *structure* shall be designed and constructed to act as an integral unit in resisting *seismic forces* unless separated structurally by a distance sufficient to avoid damaging contact under total deflection, δ_x, as determined in Sec. 5.3.7.1.

TABLE 5.2.8 Allowable Story Drift, Δ_a [a] (in. or mm)

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

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

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

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

5.3 INDEX FORCE ANALYSIS PROCEDURE: An index force analysis shall consist of the application of static lateral index forces to a linear mathematical model of the *structure* independently in each of two orthogonal directions. For purposes of analysis, the *structure* shall be considered to be fixed at the base. The lateral index forces shall be as given by Eq. 5.3 and shall be applied simultaneously at each floor level:

$$F_x = 0.01 w_x \tag{5.3}$$

where:

F_x = the design lateral force applied at *Story x*,

w_x = the portion of the total gravity load of the *structure*, *W*, located or assigned to Level *x*, and

W = the total *dead load* and applicable portions of other loads listed below:

1. In areas used for storage, a minimum of 25 percent of the floor *live load* shall be applicable. Floor *live load* in public garages and open parking *structures* is not applicable.
2. Where an allowance for *partition* load is included in the floor load design, the actual *partition* weight or a minimum weight of 10 psf (500 Pa/m^2) of floor area, whichever is greater, shall be applicable.
3. Total operating weight of permanent equipment.
4. In areas where the design flat roof snow load does not exceed 30 pounds per square ft, the effective snow load is permitted to be taken as zero. In areas where the design snow load is greater than 30 pounds per square ft and where siting and load duration conditions warrant and when approved by the authority having jurisdiction, the effective snow load is permitted to be reduced to not less than 20 percent of the design snow load.

5.4 EQUIVALENT LATERAL FORCE PROCEDURE: An equivalent lateral force analysis shall consist of the application of equivalent static lateral forces to a linear mathematical model of the *structure*. The directions of application of lateral forces shall be as indicated in Sec. 5.2.5.2. The lateral forces applied in each direction shall sum to a total seismic base shear given by Sec. 5.4.1 and shall be distributed vertically in accordance with Sec. 5.4.3. For purposes of analysis, the *structure* shall be considered fixed at the *base*.

5.4.1 Seismic Base Shear: The seismic *base shear*, *V*, in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \tag{5.4.1}$$

where:

C_s = the *seismic response coefficient* determined in accordance with Sec. 5.4.1.1 and

W = the total *dead load* and applicable portions of other loads as defined in Sec. 5.3.

5.4.1.1 Calculation of Seismic Response Coefficient: The *seismic response coefficient*, C_s, shall be determined in accordance with the following equation:

$$C_s = \frac{S_{DS}}{R/I} \qquad (5.4.1.1\text{-}1)$$

where:

S_{DS} = the design spectral response acceleration in the short period range as determined from Sec. 4.1.2.5,

R = the response modification factor from Table 5.2.2, and

I = the *occupancy importance factor* determined in accordance with Sec. 1.4.

The value of the *seismic response coefficient* computed in accordance with Eq. 5.4.1.1-1 need not exceed the following:

$$C_s = \frac{S_{D1}}{T(R/I)} \qquad (5.4.1.1\text{-}2)$$

where I and R are as defined above and

S_{D1} = the design spectral response acceleration at a period of 1.0 second as determined from Sec. 4.1.2.5,

T = the fundamental period of the *structure* (sec) determined in Sec. 5.4.2, and

S_1 = the mapped *maximum considered earthquake* spectral response acceleration determined in accordance with Sec. 4.1.

C_s shall not be taken less than:

$$C_s = 0.044 I S_{DS} \qquad (5.4.1.1\text{-}3)$$

For *structures* in *Seismic Design Categories* E and F, the value of the seismic response co-efficient, C_s, shall not be taken less than:

$$C_s = \frac{0.5 S_1}{R/I} \qquad (5.4.1.1\text{-}4)$$

For regular *structures* 5 *stories* or less in height and having a period, T, of 0.5 seconds or less, the *seismic response coefficient*, C_s, shall be permitted to be calculated using values of 1.5 and 0.6, respectively, for the mapped *maximum considered earthquake* spectral response accelerations, S_S and S_1.

A soil-*structure* interaction reduction is permitted when determined using Sec. 5.8 or other generally accepted procedures approved by the authority having jurisdiction.

5.4.2 Period Determination: The fundamental period of the *building*, T, in the direction under consideration shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The fundamental period, T, so calculated, shall not exceed the product of the coefficient for the upper limit on calculated period, C_u, from Table 5.4.2 and the approximate fundamental period, T_a, calculated in accordance with Sec. 5.4.2.1. The approximate period formulae of Sec. 5.4.2.1 is permitted to be used directly as an alternative to performing an analysis to determine the fundamental period of the *building*, T.

TABLE 5.4.2 Coefficient for Upper Limit on Calculated Period

Design Spectral Response Acceleration at 1 Second, S_{D1}	**Coefficient C_u**
Greater than or equal to 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
0.1	1.7
Less than or equal to 0.05	1.7

5.4.2.1 Approximate Fundamental Period: The approximate fundamental period, T_a, in seconds, shall be determined from the following equation:

$$T_a = C_r h_n^x \tag{5.4.2.1-1}$$

where h_n is the height (ft or m) above the *base* to the highest level of the *structure* and the values of C_r and x shall be determined from Table 5.4.2.1.

TABLE 5.4.2.1 Values of Approximate Period Parameters C_r and x

Structure Type	C_{OT}	x
Moment resisting frame systems of steel in which the frames resist 100 percent of the required *seismic force* and are not enclosed or adjoined by more rigid *components* that will prevent the frames from deflecting when subjected to *seismic forces*.	0.028 (metric 0.0724)	0.8
Moment resisting frame systems of *reinforced concrete* in which the frames resist 100 percent of the required *seismic force* and are not enclosed or adjoined by more rigid *components* that will prevent the frames from deflecting when subjected to *seismic forces*.	0.016 (metric 0.0466)	0.9
Eccentrically braced steel frames	0.03 (metric 0.0731)	0.75
All other structural systems	0.02 (metric 0.0488)	0.75

Alternatively, the approximate fundamental period, T_a, in seconds, is permitted to be determined from the following equation for concrete and steel moment resisting frame *structures* not exceeding 12 *stories* in height and having a minimum *story* height of 10 ft (3 m):

$$T_a = 0.1N \qquad (5.4.2.1\text{-}2)$$

where N = number of stories.

The approximate fundamental period, T_a, in seconds, for masonry or concrete shear wall *structures* is permitted to be determined from the following equation:

$$T_a = \frac{0.0019}{\sqrt{C_w}} h_n \; T_a = \frac{0.0062}{\sqrt{C_W}} h_n \qquad (5.4.2.1\text{-}3)$$

where C_w is a coefficient related to the effective shear wall area and h_n is as defined above. The coefficient C_w shall be calculated from the following equation:

$$C_w = \frac{100}{A_B} \sum_{i=1}^{n} \left(\frac{h_n}{h_i}\right) \frac{A_i}{\left[1 + 0.83\left(\frac{h_n}{D}\right)^2\right]} \qquad (5.4.2.1\text{-}4)$$

where:

A_B = the base area of the structure (ft^2. or m^2),

A_i = the area of shear wall *i* (ft^2. or m^2),

D_i = the length of shear wall *i* (ft or m),

h_i = the height of shear wall *i* (ft or m), and

n = the number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

5.4.3 Vertical Distribution of Seismic Forces: The lateral force, F_x (kip or kN), induced at any level shall be determined from the following equations:

$$F_x = C_{vx} V \tag{5.4.3-1}$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{5.4.3-2}$$

where:

C_{vx} = vertical distribution factor,

V = total design lateral force or shear at the *base* of the *structure* (kip or kN),

w_i and w_x = the portion of the total *gravity load* of the *structure*, *W*, located or assigned to Level *i* or *x*,

h_i and h_x = the height (ft or m) from the *base* to Level *i* or *x*, and

k = an exponent related to the *structure* period as follows:

For *structures* having a period of 0.5 seconds or less, $k = 1$

For *structures* having a period of 2.5 seconds or more, $k = 2$

For *structures* having a period between 0.5 and 2.5 seconds, *k* shall be 2 or shall be determined by linear interpolation between 1 and 2

5.4.4 Horizontal Shear Distribution: The seismic design *story shear* in any *story*, V_x (kip or kN), shall be determined from the following equation:

$$V_x = \sum_{i=x}^{n} F_i \tag{5.4.4}$$

where F_i = the portion of the seismic *base shear*, V (kip or kN), induced at Level i.

The seismic design *story shear*, V_x (kip or kN), shall be distributed to the various vertical elements of the *seismic-force-resisting system* in the *story* under consideration based on the relative lateral stiffnesses of the vertical-resisting elements and the *diaphragm*.

5.4.4.1 Inherent Torsion: The distribution of lateral forces at each level shall consider the effect of the inherent torsional moment, M_t (kip·ft or kN·m), resulting from eccentric location of the masses.

5.4.4.2. Accidental Torsion: In addition to the inherent torsional moment, the distribution of lateral forces also shall include accidental torsional moments, M_{ta} (kip·ft or kN·m), caused by an assumed *displacement* of the mass each way from its actual location by a distance equal to 5 percent of the dimension of the *structure* perpendicular to the direction of the applied forces.

5.4.4.3 Dynamic Amplification of Torsion: For *structures* of *Seismic Design Categories* C, D, E and F where Type 1a or 1b torsional irregularity exists as defined in Table 5.2.3.1, the effects of torsional irregularity shall be accounted for by multiplying the sum of M_t plus M_{ta} at each level by a torsional amplification factor, A_x, determined from the following equation:

$$A_x = \left(\frac{\delta_{max}}{1.2\,\delta_{avg}} \right)^2 \tag{5.4.4.3-1}$$

where:

δ_{max} = the maximum *displacement* at Level x (in. or mm) and

δ_{avg} = the average of the *displacements* at the extreme points of the *structure* at Level x (in. or mm).

The torsional amplification factor, A_x, is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

5.4.5 Overturning: The *structure* shall be designed to resist overturning effects caused by the *seismic forces* determined in Sec. 5.3.4. At any *story*, the increment of overturning moment in the *story* under consideration shall be distributed to the various vertica- force-resisting elements in the same proportion as the distribution of the horizontal shears to those elements.

The overturning moments at Level x, M_x (kip·ft or kN·m), shall be determined from the following equation:

$$M_x = \sum_{i-x}^{n} F_i (h_i - h_x) \tag{5.4.5}$$

where:

F_i = the portion of the seismic *base shear*, V, induced at Level i and

h_i and h_x = the height (ft or m) from the *base* to Level i or x,

The foundations of *structures*, except *inverted pendulum-type structures*, shall be permitted to be designed for three-fourths of the foundation overturning design moment, M_f (kip·ft or kN·m), determined using Eq. 5.4.5 at the foundation-soil interface.

5.4.6 Drift Determination and *P*-Delta Effects: *Story* drifts and, where required, member forces and moments due to *P-delta effects* shall be determined in accordance with this section. Determination of *story* drifts shall be based on the application of the design *seismic forces* to a mathematical model of the physical *structure*. The model shall include the stiffness and *strength* of all elements that are significant to the distribution of forces and *deformations* in the *structure* and shall represent the spatial distribution of the mass and stiffness of the *structure*. In addition, the model shall comply with the following:

1. Stiffness properties of *reinforced concrete* and masonry elements shall consider the effects of cracked sections and
2. For steel *moment resisting frame* systems, the contribution of panel zone *deformations* to overall *story* drift shall be included.

5.4.6.1 Story Drift Determination: The design *story* drift, Δ, shall be computed as the difference of the deflections at the center of mass at the top and bottom of the *story* under consideration.

> **Exception:** For *structures* of *Seismic Design Categories* C, D, E and F having plan irregularity Type 1a or 1b of Table 5.4.3.2-2, the design *story* drift, Δ, shall be computed as the largest difference of the deflections along any of the edges of the *structure* at the top and bottom of the *story* under consideration.

The deflections of Level x, δ_x (in. or mm), shall be determined in accordance with following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad (5.4.6.1)$$

where:

C_d = the deflection amplification factor in Table 5.2.2,

δ_{xe} = the deflections determined by an elastic analysis (in. or mm), and

I = the *occupancy importance factor* determined in accordance with Sec. 1.4.

The elastic analysis of the *seismic-force-resisting system* shall be made using the prescribed seismic design forces of Sec. 5.4.3. For the purpose of this section, the value of the base shear, V, used in Eq. 5.3.2 need not be limited by the value obtained from Eq. 5.3.2.1-3.

For determining compliance with the *story* drift limitation of Sec. 5.2.8, the deflections of Level x, δ_x (in. or mm), shall be calculated as required in this section. For purposes of this drift analysis only, it is permissible to use the computed fundamental period, T (secs), of the *structure* without the upper bound limitation specified in Sec. 5.4.2 when determining drift level seismic design forces.

Where applicable, the design *story* drift, Δ (in. or mm), shall be increased by the incremental factor relating to the *P-delta effects* as determined in Sec. 5.4.6.2.

5.4.6.2 *P*-Delta Effects: *P-delta effects* on *story shears* and moments, the resulting member forces and moments, and the *story* drifts induced by these effects are not required to be considered when the stability coefficient, θ, as determined by the following equation is equal to or less than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad (5.4.6.2\text{-}1)$$

where:

P_x = the total vertical design load at and above Level x (kip or kN). When calculating the vertical design load for purposes of determining P-delta, the individual load factors need not exceed 1.0.

Δ = the design *story* drift occurring simultaneously with V_x (in. or mm).

V_x = the seismic shear force acting between Level x and x - 1 (kip or kN).

h_{sx} = the *story* height below Level x (in. or mm).

C_d = the deflection amplification factor in Table 5.2.2.

The stability coefficient, θ, shall not exceed θ_{max} determined as follows:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \qquad (5.4.6.2\text{-}2)$$

where β is the ratio of shear demand to shear capacity for the *story* between Levels x and x - 1. This ratio is permitted to be conservatively taken as 1.0.

When the stability coefficient, θ, is greater than 0.10 but less than or equal to θ_{max}, the incremental factor related to *P-delta effects*, a_d, shall be determined by rational analysis (see Part 2, *Commentary*). To obtain the *story* drift for including the *P-delta effects*, the design *story* drift determined in Sec. 5.4.6.1 shall be permitted to be multiplied by 1.0/(1 - θ).

When θ is greater than θ_{max}, the *structure* is potentially unstable and shall be redesigned.

5.5 MODAL RESPONSE SPECTRUM ANALYSIS PROCEDURE: A modal response spectrum analysis shall consist of the analysis of a linear mathematical model of the *structure* to determine the maximum accelerations, forces, and displacements resulting from the dynamic response to ground shaking represented by the design response spectrum. The analysis shall be performed in accordance with the requirements of this section. For purposes of analysis, the *structure* shall be permitted to be considered to be fixed at the *base* or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness of foundations. The symbols used in this section have the same meaning as those for similar terms used in Sec. 5.4 but with the subscript *m* denoting quantities relating to the m^{th} mode.

5.5.1 Modeling: A mathematical model of the *structure* shall be constructed that represents the spatial distribution of mass and stiffness throughout the *structure*. For regular *structures* with independent orthogonal *seismic-force-resisting systems*, independent two-dimensional models are permitted to be constructed to represent each system. For irregular *structures* or *structures* without independent orthogonal systems, a three-dimensional model incorporating a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis shall be included at each level of the *structure*. Where the *diaphragms* are not rigid compared to the vertical elements of the lateral-force-resisting system, the model should include representation of the diaphragm's flexibility and such additional dynamic degrees of freedom as are required to account for the participation of the *diaphragm* in the *structure's* dynamic response. In addition, the model shall comply with the following:

1. Stiffness properties of concrete and masonry elements shall consider the effects of cracked sections and

2. The contribution of panel zone *deformations* to overall *story* drift shall be included for steel moment frame resisting systems.

5.5.2 Modes: An analysis shall be conducted to determine the natural modes of vibration for the *structure* including the period of each mode, the modal shape vector ϕ, the modal participation factor, and modal mass. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual mass in each of two orthogonal directions.

5.5.3 Modal Properties: The required periods, mode shapes, and participation factors of the *structure* shall be calculated by established methods of structural analysis for the fixed-*base* condition using the masses and elastic stiffnesses of the *seismic-force-resisting system*.

5.5.4 Modal Base Shear: The portion of the *base shear* contributed by the m^{th} mode, V_m, shall be determined from the following equations:

$$V_m = C_{sm} \overline{W_m} \qquad (5.5.4\text{-}1)$$

$$\overline{W_m} = \frac{\left(\sum_{i=1}^{n} w_i \phi_{im} \right)^2}{\sum_{i=1}^{n} w_i \phi_{im}^2} \qquad (5.5.4\text{-}2)$$

where:

C_{sm} = the modal *seismic response coefficient* as determined by Eq. 5.5.4-3,

$\overline{W}_m$ = the effective modal *gravity load* including portions of the *live load* as defined in Sec. 5.3,

w_i = the portion of the total *gravity load* of the *structure* at Level *i*, and

ϕ_{im} = the *displacement* amplitude at the i^{th} level of the *structure* when vibrating in its m^{th} mode.

The modal *seismic response coefficient,* C_{sm}, shall be determined in accordance with the following equation:

$$C_{sm} = \frac{S_{am}}{R/I} \qquad (5.5.4\text{-}3)$$

where:

S_{am} = The design spectral response acceleration at period T_m determined from either the general design response spectrum of Sec. 4.1.2.5 or a site-specific response spectrum determined in accordance with Sec. 4.1.3,

R = the response modification factor determined from Table 5.2.2,

I = the *occupancy importance factor* determined in accordance with Sec. 1.4, and

T_m = the modal period of vibration (in seconds) of the m^{th} mode of the *structure*.

Exceptions:

1. When the general design response spectrum of Sec. 4.1.2.6 is used for *structures* on *Site Class* D, E or F soils, the modal seismic design coefficient, C_{sm}, for modes other than the fundamental mode that have periods less than 0.3 seconds is permitted to be determined by the following equation:

   where S_{DS} is as defined in Sec. 4.1.2.5 and R, I, and T_m are as defined above.

$$C_{sm} = \frac{0.4 S_{DS}}{(R/I)}(1.0 + 5.0 T_m) \qquad (5.5.4\text{-}4)$$

2. When the general design response spectrum of Sec. 4.1.2.6 is used for *structures* where any modal period of vibration, T_m, exceeds 4.0 seconds, the modal seismic design coefficient, C_{sm}, for that mode is permitted to be determined by the following equation:

$$C_{sm} = \frac{4S_{D1}}{(R/I)T_m^2} \qquad (5.5.4\text{-}5)$$

where R, I, and T_m are as defined above and and S_{D1} is the design spectral response acceleration at a period of 1 second as determined in Sec. 4.1.2.5.

The reduction due to soil-*structure* interaction as determined in Sec. 5.8.3 shall be permitted to be used.

5.5.5 Modal Forces, Deflections, and Drifts: The modal force, F_{xm}, at each level shall be determined by the following equations:

$$F_{xm} = C_{vxm} V_m \qquad (5.5.5\text{-}1)$$

and

$$C_{vxm} = \frac{w_x \phi_{xm}}{\sum_{i=1}^{n} w_i \phi_{im}} \qquad (5.5.5\text{-}2)$$

where:

C_{vxm}	=	the vertical distribution factor in the m^{th} mode,
V_m	=	the total design lateral force or shear at the *base* in the m^{th} mode,
w_i, w_x	=	the portion of the total *gravity load*, W, located or assigned to Level i or x,
ϕ_{xm}	=	the *displacement* amplitude at the x^{th} level of the *structure* when vibrating in its m^{th} mode, and
ϕ_{im}	=	the *displacement* amplitude at the i^{th} level of the *structure* when vibrating in its m^{th} mode.

The modal deflection at each level, δ_{xm}, shall be determined by the following equations:

$$\delta_{xm} = \frac{C_d \delta_{xem}}{I} \qquad (5.5.5\text{-}3)$$

and

$$\delta_{xem} = \left(\frac{g}{4\pi^2}\right)\left(\frac{T_m^2 F_{xm}}{w_x}\right) \quad (5.5.5\text{-}4)$$

where:

C_d = the deflection amplification factor determined from Table 5.2.2,

δ_{xem} = the deflection of Level x in the m^{th} mode at the center of the mass at Level x determined by an elastic analysis,

g = the acceleration due to gravity (ft/s^2 or m/s^2),

I = the *occupancy importance factor* determined in accordance with Sec. 1.4,

T_m = the modal period of vibration, in seconds, of the m^{th} mode of the *structure*,

F_{xm} = the portion of the seismic *base shear* in the m^{th} mode, induced at Level x, and

w_x = the portion of the total *gravity load* of the *structure*, W, located or assigned to Level x.

The modal drift in a *story*, Δ_m, shall be computed as the difference of the deflections, δ_{xm}, at the top and bottom of the *story* under consideration.

5.5.6 Modal Story Shears and Moments: The *story shears*, *story* overturning moments, and the shear forces and overturning moments in vertical elements of the structural system at each level due to the *seismic forces* determined from the appropriate equation in Sec. 5.5.5 shall be computed for each mode by linear static methods.

5.5.7 Design Values: The design value for the modal *base shear*, V_t; each of the *story shear*, moment, and drift quantities; and the deflection at each level shall be determined by combining their modal values as obtained from Sec. 5.5.5 and 5.5.6. The combination shall be carried out by taking the square root of the sum of the squares of each of the modal values or by the complete quadratic combination technique. The complete quadratic combination shall be used where closely spaced periods in the translational and torsional modes will result in cross-correlation of the modes.

A *base shear*, V, shall be calculated using the equivalent lateral force procedure in Sec. 5.4. For the purpose of this calculation, the fundamental period of the *structure*, T (sec), shall not exceed the coefficient for upper limit on the calculated period, C_u, times the approximate fundamental period of the *structure*, T_a. Where the design value for the modal *base shear*, V_t, is less than 85 percent of the calculated *base shear*, V, using the equivalent lateral force procedure, the design *story shears*, moments, drifts, and floor deflections shall be multiplied by the following modification factor:

$$0.85\frac{V}{V_t} \quad (5.5.7.1)$$

where:

V = the equivalent lateral force procedure *base shear* calculated in accordance with Sec. 5.4 and

V_t = the modal *base shear* calculated in accordance with this section.

Where soil-*structure* interaction in accordance with Sec. 5.8 is considered, the reduced value of V calculated in accordance with that section may be used for V in Eq. 5.5.7.1.

5.5.8 Horizontal Shear Distribution: The horizontal distribution of shear shall be in accordance with the requirements of Sec. 5.4.4 except that amplification of torsion per Sec. 5.4.4.1.3 is not required for that portion of the torsion included in the dynamic analysis model.

5.5.9 Foundation Overturning: The foundation overturning moment at the foundation-soil interface shall be permitted to be reduced by 10 percent.

5.5.10 P-Delta Effects: The *P-delta effects* shall be determined in accordance with Sec. 5.4.6. The *story* drifts and *story shears* shall be determined in accordance with Sec. 5.4.6.1.

5.6 LINEAR RESPONSE HISTORY ANALYSIS PROCEDURE: A linear response history analysis shall consist of an analysis of a linear mathematical model of the *structure* to determine its response, through methods of numerical integration, to suites of ground motion acceleration histories compatible with the design response spectrum for the site. The analysis shall be performed in accordance with the provisions of this section. For the purposes of analysis, the *structure* shall be permitted to be considered to be fixed at the *base* or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness of foundations.

5.6.1 Modeling: Mathematical models shall conform to the requirements of Sec. 5.5.1.

5.6.2 Ground Motion: A suite of not less than three appropriate ground motions shall be used in the analysis. Ground motion shall conform to the requirements of this section.

5.6.2.1 Two-Dimensional Analysis: When two-dimensional analyses are performed, each ground motion shall consist of a horizontal acceleration history selected from an actual recorded event. Appropriate acceleration histories shall be obtained from records of events having magnitudes, fault distances, and source mechanisms that are consistent with those that control the *maximum considered earthquake*. Where the required number of appropriate recorded ground motion records are not available, appropriate simulated ground motion records shall be used to make up the total number required. The ground motions shall be scaled such that the average value of the 5 percent damped response spectra for the suite of motions is not less than the design response spectrum for the site determined in accordance with Sec. 4.1.3 for periods ranging from $0.2T$ to $1.5T$ seconds where T is the natural period of the *structure* in the fundamental mode for the direction of response being analyzed.

5.6.2.2 Three-Dimensional Analysis: When three-dimensional analysis is performed, ground motions shall consist of pairs of appropriate horizontal ground motion acceleration components that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distances, and source mechanisms that are consistent with those that control the *maximum considered earthquake*. Where the required number of recorded ground motion pairs are not available, appropriate simulated ground motion pairs shall be used to make up the total number required. For each pair of horizontal ground motion components, the square root of the sum of the squares (SRSS) of the 5 percent damped response spectrum of the scaled horizontal components shall be constructed. Each pair of motions shall be scaled such that the average value of the SRSS spectra from all horizontal component pairs is not less than 1.3 times the 5 percent damped design response spectrum determined in accordance with Sec. 4.1.3 for periods ranging from $0.2T$ to $1.5T$ seconds where T is the natural period of the fundamental mode of the *structure*.

5.6.3 Response Parameters: For each ground motion analyzed, the individual response parameters shall be scaled by the quantity I/R where I is the occupancy importance factor determined in accordance with Sec. 1.4 and R is the response modification coefficient selected in accordance with Sec. 5.2.2. The maximum value of the base shear, V_j, member forces, Q_{Ej}, and the interstory drifts, δ_{ij}, at each *story* scaled as indicated above shall be determined. When the maximum scaled base shear predicted by the analysis, V_j, is less than given by Eq. 5.4.1.1-3 or, in *Seismic Design Categories* E and F, Eq. 5.4.1.1-4, the scaled member forces, Q_{Ej}, shall be additionally scaled by the factor V/V_j where V is the minimum base shear determined in accordance with Eq. 5.4.1.1-3 or, for *structures* in Seismic Design Category E or F, Eq. 5.4.1.1-4.

If at least seven ground motions are analyzed, the design member forces, Q_E, used in the load combinations of Sec. 5.2.7 and the design interstory drift, Δ, used in the evaluation of drift in accordance with Sec. 5.2.8 shall be permitted to be taken, respectively, as the average of the scaled Q_{Ej} and δ_{ij} values determined from the analyses and scaled as indicated above. If less than seven ground motions are analyzed, the design member forces, Q_E, and the design interstory drift, Δ, shall be taken as the maximum value of the scaled Q_{Ej} *and* δ_{ij} values determined from the analyses.

Where the *Provisions* require the consideration of the special load combinations of Sec. 5.2.7.1, the value of $\Omega_0 Q_E$ need not be taken larger than the maximum of the unscaled value, Q_{Ej}, obtained from the suite of analyses.

5.7 NONLINEAR RESPONSE HISTORY ANALYSIS PROCEDURE : A nonlinear response history analysis shall consist of an analysis of a mathematical model of the *structure* that directly accounts for the nonlinear hysteretic behavior of the structure's *components* to determine its response, through methods of numerical integration, to suites of ground motion acceleration histories compatible with the design response spectrum for the site. The analysis shall be performed in accordance with the requirements of this section.

5.7.1 Modeling: A mathematical model of the *structure* shall be constructed that represents the spatial distribution of mass throughout the *structure*. The hysteretic behavior of elements shall be modeled consistent with suitable laboratory test data and shall account for all significant yielding, strength degradation, stiffness degradation, and hysteretic pinching indicated by such test data. Strength of elements shall be based on expected values considering material overstrength, strain hardening, and hysteretic strength degradation. Linear properties consistent with the provisions of Sec. 5.5.1 shall be permitted to be used for those elements demonstrated by the analysis to remain within their linear range of response. The *structure* shall be assumed to have a fixed *base* or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness and load carrying characteristics of the foundations consistent with site-specific soils data and rational principles of engineering mechanics.

For regular *structures* with independent orthogonal *seismic-force-resisting systems*, independent two-dimensional models shall be permitted to be constructed to represent each system. For *structures* having plan irregularity Type 1a, 1b, 4, or 5 of Table 5.2.3.2 or *structures* without independent orthogonal systems, a three-dimensional model incorporating a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis at each level of the *structure* shall be used. Where the *diaphragms* are not rigid compared to the vertical elements of the lateral-force-resisting system, the model shall include representation of the diaphragm's flexibility and such additional dynamic degrees of freedom as are required to account for the participation of the *diaphragm* in the *structure's* dynamic response.

5.7.2 Ground Motion and Other Loading: Ground motion shall conform to the requirements of Sec. 5.6.2. The *structure* shall be analyzed for the effects of these ground motions simultaneously with the effects of dead load in combination with not less than 25 percent of the required live loads.

5.7.3 Response Parameters: For each ground motion analyzed, individual response parameters consisting of the maximum value of the individual member forces, Q_{Ej}, member inelastic deformations, γ_j, and interstory drifts, δ_{ij}, at each story shall be determined.

If at least seven ground motions are analyzed, the design values of member forces, Q_E, member inelastic deformations, γ_i, and interstory drift, Δ , shall be taken, respectively, as the average of the scaled Q_{Ej}, γ_i, and δ_i values determined from the analyses. If less than seven ground motions are analyzed, the design member forces, Q_E, design member inelastic deformations, γ_i and the design interstory drift, Δ, shall be taken as the maximum value of the scaled Q_{Ej}, γ_j, and δ_{ij} values determined from the analyses.

5.7.3.1 Member Strength: The adequacy of members to resist the load combinations of Sec 5.2.7 need not be evaluated.

> **Exception:** Where the *Provisions* requires the consideration of the special load combinations of Sec. 5.2.7.1, the maximum value of Q_{Ej} obtained from the suite of analyses shall be taken in place of the quantity $\Omega_0 Q_E$.

5.7.3.2 Member Deformation:. The adequacy of individual members and their connections to withstand the design deformation values, γ_i, predicted by the analyses shall be evaluated based on laboratory test data for similar components. The effects of gravity and other loads on member deformation capacity shall be considered in these evaluations. Member deformation shall not exceed two thirds of the value that results in loss of ability to carry gravity loads or that results in deterioration of member strength to less than the 67 percent of the peak value.

5.7.3.3 Interstory Drift: The design interstory drift obtained from the analyses shall not exceed 125 percent of the drift limit specified in Sec. 5.2.8.

5.7.4 Design Review: A design review of the *seismic-force-resisting system* and the structural analysis shall be performed by an independent team of *registered design professionals* in the appropriate disciplines and others experienced in seismic analysis methods and the theory and application of nonlinear seismic analysis and structural behavior under extreme cyclic loads. The design review shall include, but not be limited to, the following:

1. Review of any site-specific seismic criteria employed in the analysis including the development of site-specific spectra and ground motion time histories,
2. Review of acceptance criteria used to demonstrate the adequacy of structural elements and systems to withstand the calculated force and deformation demands, together with that laboratory and other data used to substantiate these criteria,
3. Review of the preliminary design including the determination of the target displacement of the structure and the margins remaining beyond these displacements, and
4. Review of the final design of the entire structural system and all supporting analyses.

5.8 SOIL-STRUCTURE INTERACTION EFFECTS:

5.8.1 General: The requirements set forth in this section are permitted to be used to incorporate the effects of soil-*structure* interaction in the determination of the *design earthquake* forces and the corresponding *displacements* of the *structure*. The use of these requirements will decrease the design values of the *base shear*, lateral forces, and overturning moments but may increase the computed values of the lateral *displacements* and the secondary forces associated with the *P*-delta effects.

The requirements for use with the equivalent lateral force procedure are given in Sec. 5.8.2 and those for use with the modal analysis procedure are given in Sec. 5.8.3.

5.8.2 Equivalent Lateral Force Procedure: The following requirements are supplementary to those presented in Sec. 5.4.

5.8.2.1 Base Shear: To account for the effects of soil-*structure* interaction, the *base shear*, *V*, determined from Eq. 5.4.1-1 may be reduced to:

$$\tilde{V} = V - \Delta V \qquad (5.8.2.1\text{-}1)$$

The reduction, ΔV, shall be computed as follows:

$$\Delta V = \left[C_s - \tilde{C}_s \left(\frac{0.05}{\tilde{\beta}} \right)^{0.4} \right] \overline{W} \qquad (5.8.2.1\text{-}2)$$

where:

C_s = the *seismic response coefficient* computed from Eq. 5.4.1.1-1 using the fundamental natural period of the fixed-*base structure* (T or T_a) as specified in Sec.5.4.2,

$\tilde{C}_s$ = the *seismic response coefficient* computed from Eq. 5.4.1.1-1 using the fundamental natural period of the flexibly supported *structure* ($\tilde{T}$) defined in Sec. 5.8.2.1.1,

$\tilde{B}$ = the fraction of critical damping for the *structure*-foundation system determined in Sec. 5.8.2.1.2, and

$\overline{W}$ = the effective *gravity load* of the *structure*, which shall be taken as 0.7*W*, except that for *structures* where the *gravity load* is concentrated at a single level, it shall be taken equal to *W*.

The reduced *base shear,* $\tilde{V}$, shall in no case be taken less than 0.7*V*.

5.8.2.1.1 Effective Building Period: The effective period, $\tilde{T}$, shall be determined as follows:

$$\tilde{T} = T \sqrt{1 + \frac{\bar{k}}{K_y}\left(1 + \frac{K_y \overline{h}^2}{K_\theta}\right)} \qquad (5.8.2.1.1\text{-}1)$$

where:

T = the fundamental period of the *structure* as determined in Sec. 5.4.2;

$\bar{k}$ = the stiffness of the *structure* when fixed at the *base*, defined by the following:

$$\bar{k} = 4\pi^2 \left(\frac{\overline{W}}{g T^2}\right) \qquad (5.8.2.1.1\text{-}2)$$

$\bar{h}$ = the effective height of the *structure*, which shall be taken as 0.7 times the total height, h_n, except that for *structures* where the *gravity load* is effectively concentrated at a single level, it shall be taken as the height to that level;

K_y = the lateral stiffness of the foundation defined as the horizontal force at the level of the foundation necessary to produce a unit deflection at that level, the force and the deflection being measured in the direction in which the *structure* is analyzed;

K_θ = the rocking stiffness of the foundation defined as the moment necessary to produce a unit average rotation of the foundation, the moment and rotation being measured in the direction in which the *structure* is analyzed; and

g = the acceleration of gravity.

The foundation stiffnesses, K_y and K_θ, shall be computed by established principles of foundation mechanics (see the *Commentary*) using soil properties that are compatible with the soil strain levels associated with the design earthquake motion. The average shear modulus, *G*, for the soils beneath the foundation at large strain levels and the associated shear wave velocity, v_s, needed in these computations shall be determined from Table 5.8.2.1.1 where:

v_{so} = the average shear wave velocity for the soils beneath the foundation at small strain levels (10^{-3} percent or less),

G_o = $\gamma v^2_{so}/g$ = the average shear modulus for the soils beneath the foundation at small strain levels, and

γ = the average unit weight of the soils.

TABLE 5.8.2.1.1 Values of G/G_o and v_s/v_{so}

	Peak Ground Acceleration, (g)			
	≤ 0.10	**≤ 0.15**	**0.20**	**≥ 0.30**
Value of G/G_o	0.81	0.64	0.49	0.42
Value of v_s/v_{so}	0.90	0.80	0.70	0.65

Alternatively, for *structures* supported on mat foundations that rest at or near the ground surface or that are embedded in such a way that the side *wall* contact with the soil cannot be considered to remain effective during the design ground motion, the effective period of the *structure* may be determined from:

$$\tilde{T} = T\sqrt{1 + \frac{25\alpha r_a \bar{h}}{v_s^2 T^2}\left(1 + \frac{1.12 r_a \bar{h}^2}{\alpha_\theta r_m^3}\right)} \tag{5.8.2.1.1-3}$$

where:

α = the relative weight density of the *structure* and the soil defined by:

$$\alpha = \frac{\bar{W}}{\gamma A_o \bar{h}} \tag{5 8.2.1.1-4}$$

r_a and r_m = characteristic foundation lengths defined by:

$$r_a = \sqrt{\frac{A_o}{\pi}} \tag{5.8.2.1.1-5}$$

and

$$r_m = \sqrt[4]{\frac{4 I_o}{\pi}} \tag{5.8.2.1.1-6}$$

where:

A_o = the area of the foundation,

I_o = the static moment of the foundation about a horizontal centroidal axis normal to the direction in which the *structure* is analyzed, and

5.8.2.1.2 Effective Damping: The *effective damping* factor for the *structure*-foundation system, $\tilde{\beta}$, shall be computed as follows:

$$\tilde{\beta} = \beta_o + \frac{0.05}{\left(\frac{\tilde{T}}{T}\right)^3} \qquad (5.8.2.1.2\text{-}1)$$

where β_o = the foundation damping factor as specified in Figure 5.8.2.1.2.

The values of β_o corresponding to S_{DS} = 0.375 in Figure 5.8.2.1.2 shall be determined by averaging the results obtained from the solid lines and the dashed lines.

The quantity r in Figure 5.8.2.1.2 is a characteristic foundation length that shall be determined as follows:

For $\bar{h}/L_o \leq 0.5$,

$$r = r_a = \sqrt{\frac{A_o}{\pi}} \qquad (5.8.2.1.2\text{-}2)$$

For $\bar{h}/L_o \geq 1$,

$$r = r_m = \sqrt[4]{\frac{4I_o}{\pi}} \qquad (5.8.2.1.2\text{-}3)$$

where:

L_o = the overall length of the side of the foundation in the direction being analyzed,

A_o = the area of the load-carrying foundation, and

I_o = the static moment of inertia of the load-carrying foundation.

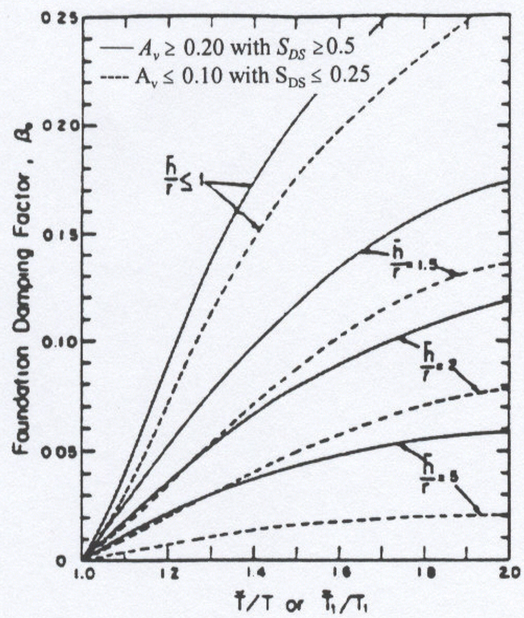

FIGURE 5.8.2.1.2 Foundation damping factor.

For intermediate values of $\bar{h}/L_0$, the value of *r* shall be determined by linear interpolation.

> **Exception:** For *structures* supported on point bearing piles and in all other cases where the foundation soil consists of a soft stratum of reasonably uniform properties underlain by a much stiffer, rock-like deposit with an abrupt increase in stiffness, the factor β_o in Eq. 5.8.2.1.2-1 shall be replaced by:
>
> $$\beta_o' = \left(\frac{4D_s}{V_s\tilde{T}}\right)^2 \beta_o \qquad (5.8.2.1.2\text{-}4)$$
>
> if $4D_s/v_s\tilde{T} < 1$ where D_s is the total depth of the stratum.

The value of $\tilde{\beta}$ computed from Eq. 5.8.2.1.2-1, both with or without the adjustment represented by Eq. 5.8.2.1.2-4, shall in no case be taken as less than $\tilde{\beta}$ = 0.05 or greater than $\tilde{\beta}$ = 0.20.

5.8.2.2 Vertical Distribution of Seismic Forces: The distribution over the height of the *structure* of the reduced total *seismic force,* $\tilde{V}$, shall be considered to be the same as for the *structure* without interaction.

5.8.2.3 Other Effects: The modified *story shears*, overturning moments, and torsional effects about a vertical axis shall be determined as for *structure*s without interaction using the reduced lateral forces.

The modified deflections, $\tilde{\delta}_x$, shall be determined as follows:

$$\tilde{\delta}_x = \frac{\tilde{V}}{V}\left(\frac{M_o h_x}{K_\theta} + \delta_x\right) \qquad (5.8.2.3)$$

where:

M_o = the overturning moment at the *base* determined in accordance with Sec. 5.4.5 using the unmodified *seismic forces* and not including the reduction permitted in the design of the foundation,

h_x = the height above the *base* to the level under consideration, and

δ_x = the deflections of the fixed-*base structure* as determined in Sec. 5.4.6.1 using the unmodified *seismic forces*.

The modified *story* drifts and *P-delta effects* shall be evaluated in accordance with the requirements of Sec. 5.4.6.2 using the modified *story shears* and deflections determined in this section.

5.8.3 Modal Analysis Procedure: The following requirements are supplementary to those presented in Sec. 5.5.

5.8.3.1 Modal Base Shears: To account for the effects of soil-*structure* interaction, the *base shear* corresponding to the fundamental mode of vibration, V_1, is permitted to be reduced to:

$$\tilde{V}_1 = V_1 - \Delta V_1 \qquad (5.8.3.1\text{-}1)$$

The reduction, ΔV_1, shall be computed in accordance with Eq. 5.8.2.1-2 with $\overline{W}_1$ taken as equal to the *gravity load* $\overline{W}_1$ defined by Eq. 5.5.4-2, C_s computed from Eq. 5.5.4-3 using the fundamental period of the fixed-*base structure*, T_1, and $\tilde{C}_s$ computed from Eq. 5.5.4-3 using the fundamental period of the elastically supported *structure*, $\tilde{T}_1$.

The period $\tilde{T}_1$ shall be determined from Eq. 5.8.2.1.1-1, or from Eq. 5.8.2.1.1-3 when applicable, taking $T = \tilde{T}_1$, evaluating $\overline{K}$ from Eq. 5.8.2.1.1-2 with $\overline{W} = \overline{W}_1$, and computing $\overline{h}$ as follows:

$$\bar{h} = \frac{\sum_{i=1}^{n} w_i \phi_{i1} h_i}{\sum_{i=1}^{n} w_i \phi_{i1}} \tag{5.8.3.1-2}$$

The above designated values of $\overline{W}, \bar{h}$, T, and $\tilde{T}$ also shall be used to evaluate the factor α from Eq. 5.8.2.1.1-4 and the factor β_o from Figure 5.8.2.1.2. No reduction shall be made in the shear *components* contributed by the higher modes of vibration. The reduced *base shear*, $\tilde{V}_1$, shall in no case be taken less than $0.7V_1$.

5.8.3.2 Other Modal Effects: The modified modal *seismic forces*, *story shears*, and overturning moments shall be determined as for *structures* without interaction using the modified *base shear*, $\tilde{V}_1$, instead of V_1. The modified modal deflections, $\tilde{\delta}_{xm}$, shall be determined as follows:

$$\tilde{\delta}_{xm} = \frac{\tilde{V}_1}{V_1}\left[\frac{M_{o1} h_x}{K_\theta} + \delta_{xl}\right] \tag{5.8.3.2-1}$$

and

$$\tilde{\delta}_{xm} = \delta_x \qquad \text{for } m = 2, 3, \ldots\ldots \tag{5.8.3.2-2}$$

where:

M_{o1} = the overturning *base* moment for the fundamental mode of the fixed-*base structure,* as determined in Sec. 5.5.6 using the unmodified modal *base shear* V_1, and

δ_{xm} = the modal deflections at Level x of the fixed-*base structure* as determined in Sec. 5.5.5 using the unmodified modal shears, V_m.

The modified modal drift in a *story*, Δ_m, shall be computed as the difference of the deflections, δ_{xm}, at the top and bottom of the *story* under consideration.

5.8.3.3 Design Values: The design values of the modified shears, moments, deflections, and *story* drifts shall be determined as for *structures* without interaction by taking the square root of the sum of the squares of the respective modal contributions. In the design of the foundation, the overturning moment at the foundation-soil interface determined in this manner may be reduced by 10 percent as for *structures* without interaction.

The effects of torsion about a vertical axis shall be evaluated in accordance with the requirements of Sec. 5.5.8 and the *P-delta effects* shall be evaluated in accordance with the requirements of Sec. 5.4.6.2, using the *story shears* and drifts determined in Sec. 5.8.3.2.

Appendix to Chapter 5

NONLINEAR STATIC ANALYSIS

PREFACE: This appendix introduces nonlinear static analysis, a new seismic analysis procedure sometimes known as pushover analysis, for review and comment and for later adoption into the body of the *NEHRP Recommended Provisions*.

Although nonlinear static analysis has not previously been included in design provisions for new building construction, the procedure itself is not new and has been used for many years in both research and design applications. For example, nonlinear static analysis has been used for many years as a standard methodology in the design of offshore platform structures. It also has been adopted in several standard methodologies for the seismic evaluation and retrofit of building structures, including the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA 273) and *Methodolgies for Post-earthquake Evaluation and Repair of Concrete and Masonry Buildings* (ATC 40). Nonlinear static analysis also forms the basis for earthquake loss estimation procedures contained in HAZUS, FEMA's nationally applicable earthquake loss estimation model. Finally, although it does not explicitly appear in the *NEHRP Recommended Provisions*, the nonlinear static analysis methodology forms the basis for the equivalent lateral force procedures contained in the *Provisions* for base-isolated structures and proposed for inclusion for energy-dissipated structures.

One of the key controversies surrounding the introduction of this methodology into the *Provisions* relates to the determination of the limit *deformation*, sometimes also called a target *displacement*. Several methodologies for estimating the amount of *deformation* induced in a *structure* by the *design earthquake* have been proposed and are included in various adoptions of the procedure. The approach presented in this appendix is based on statistical correlations of the *displacements* predicted by linear and nonlinear dynamic analyses of *structures* similar, but not identical, to the approach contained in FEMA 273.

A second controversy relates to the lack of consensus-backed acceptance criteria to be used to determine the adequacy of a design once the forces and *deformations* produced by *design earthquake* ground shaking are estimated. It should be noted that this same lack of acceptance criteria applies equally to the nonlinear response history approach, which already has been adopted into building codes.

Nonlinear static analysis provides a simplified method of directly evaluating nonlinear response of *structures* to strong earthquake ground shaking that can be an attractive alternative to the more complex procedures of nonlinear response history analysis. It is hoped that exposure of this approach through inclusion in this

appendix will allow the necessary consensus to be developed to permit later integration into the *Provisions* as such.

Users of this appendix also should consult the *Commentary* for guidance. Please direct all feedback on this appendix and its commentary to the BSSC.

5A.1 NONLINEAR STATIC ANALYSIS: A nonlinear static analysis shall consist of an analysis of a mathematical model of the *structure* that directly accounts for the nonlinear behavior of the *structure's* components under an incrementally increased pattern of lateral forces. In this procedure, a mathematical model of the *structure* is incrementally displaced to a target *displacement* through application of a series of lateral forces or until the *structure* collapses and the resulting internal forces, Q_{Ej}, and member *deformations*, γ_I, at each increment of loading are determined. At the target displacement for the *structure*, the resulting internal forces and deflections should be less than the capacity of each element calculated according to the applicable acceptance criteria in Sec. 5A.1.3. The analysis shall be performed in accordance with this section.

5A.1.1 Modeling: A mathematical model of the *structure* shall be constructed to represent the spatial distribution of mass and stiffness of the structural system considering the effects of component nonlinearity at *deformation* levels that exceed their elastic limit.

The nonlinear force-*deformation* characteristics of *components* shall be represented by suitable multilinear models. Unless analysis indicates that a *component* remains elastic, as a minimum a bilinear model shall be used for each *component*. The multilinear force-*deformation* characteristics for each *component*, termed a backbone, should include representation of the linear stiffness of the *component* before onset of yield, the yield strength, and the stiffness properties of the *component* after yield at various levels of *deformation*. These properties shall be consistent with principles of mechanics or laboratory data. Linear properties representing component behavior before yield shall be consistent with the provisions of Sec. 5.5.1. Strength of elements shall be based on expected values considering material overstrength and strain hardening. The properties of elements and components after yielding should account for strength and stiffness degradation due to softening or fracture as indicated by principles of mechanics or test data. The model for columns should reflect the influence of axial load when axial loads exceed 15 percent of the buckling load. The *structure* shall be assumed to have a fixed *base* or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness and load-carrying characteristics of the foundations, consistent with site-specific soil data and rational principles of engineering mechanics.

For regular *structures* with independent orthogonal *seismic-force-resisting systems*, independent two-dimensional models shall be permitted to be constructed to represent each system. For *structures* having plan irregularities Types 4 and 5 of Table 5.2.3.2 or *structures* without independent orthogonal systems, a three-dimensional model incorporating a minimum of three degrees of freedom, consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis to each level of the *structure*, shall be used. Where the *diaphragms* are not rigid compared to the vertical elements of the lateral-force-resisting system, the model should include representation of the diaphragm's flexibility. A control point shall be selected for each model. This control point normally shall be taken as the center of mass of the highest level of the *structure*. For *structures* with penthouses, the control point shall be taken as the center of mass of the level at the base of the penthouse. This level shall be termed the control level.

5A.1.2 Lateral Loads: A pattern of lateral loads shall be applied incrementally at the mass centroid of each level *I*. The pattern of lateral loads applied in each direction should follow the distribution obtained from a modal analysis for the fundamental mode of response in the direction under consideration as given by Sec. 5.5.5.

At each increment of lateral loading, *k*, the total force applied to the model shall be characterized by the base shear, V_k. The base shear at the initial increment of load, V_1, shall be taken as the design base shear calculated in accordance with Sec. 5.4.1. The base shear, *V*, should be incremented in steps that are sufficiently small to permit significant changes in individual *component* behavior, such as yielding, buckling or failure, to be detected. The *structure* shall be analyzed for these lateral forces simultaneously with the effects of *dead load* in combination with not less than 25 percent of the required *live loads*, reduced as permitted for the area of a single floor.

Loading shall be applied independently in each of two directions. At each load step, the total applied force, V_k, the lateral displacement of the control point, Δ_k, and the forces and deformations in each component shall be recorded.

5A.1.3 Limit Deformation: The incremental nonlinear analysis should be continued by increasing the base shear until the deflection at the control point exceeds 150 percent of the inelastic deflection. The expected inelastic *deformation* of the control panel shall be taken as the deflection predicted for the control point from a modal response spectrum analysis using a 5 percent damped design level response spectrum, considering only the fundamental mode of response in the direction under consideration, and factored by the coefficient C_i. When the ratio for the period, T_s, as defined in Sec. 4.1.2.6, to the fundamental period of the *structure* in the direction under consideration, T_1, is less than or equal to a value of 1.0, the coefficient C_i shall be taken as having a value of 1.0. Otherwise, the value of the coefficient C_i shall be calculated from the following equation:

$$C_i = \frac{(1 - T_s/T_1)}{R_d} + (T_s/T_1) \qquad \text{(5A.1.3-1)}$$

where T_s and T_i are as defined above and R_d is given by the following equation:

$$R_d = \frac{1.5R}{\Omega_0} \qquad \text{(5A.1.3-2)}$$

where R and Ω_0 are, respectively, the response modification and overstrength coefficients from Table 5.2.2.

5A.1.4 Design Response Parameters: For each lateral force analyzed, the design response parameters including interstory drift and member force and deformation shall be taken as the value obtained from the analysis at the expected inelastic displacement.

5A.1.4.1 Member Strength: The adequacy of members to resist the load combinations of Sec. 5.2.7 need not be evaluated.

> **Exception:** Where the *Provisions* require the consideration of the special load combinations of Sec. 5.2.7.1, the value of Ω_{Ei} obtained from the analysis at the expected inelastic deformation, as calculated from Sec. 5A.1.3, shall be taken in place of the quantity $\Omega_0 Q_E$.

5A.1.4.2 Member Deformation: The adequacy of individual members and their connections to withstand the design deformation values, γ_i, predicted by the analyses shall be evaluated based on laboratory test data for similar *components*. The effects of gravity and other loads on member deformation capacity shall be considered in these evaluations. Member deformation shall not exceed two thirds of a value that results in loss of ability to carry gravity loads or that results in deterioration of member strength to less than 67 percent of the peak value.

5A.1.4.3 Interstory Drift: The design interstory drift obtained from the analysis shall not exceed 125 percent of the drift limit specified in Sec. 5.2.8.

5A.1.5 Design Review: When the nonlinear static analysis method is used to design the *structure*, a design review of the *seismic-force-resisting system* and the structural analysis shall be performed by an independent team of *registered design professionals* in the appropriate disciplines and others experienced in seismic analysis methods and the theory and application of nonlinear seismic analysis and structural behavior under extreme cyclic loads. The design review shall include, but not be limited to, the following:

1. Review of any site-specific seismic criteria employed in the analysis including the development of site-specific spectra.,
2. Review of acceptance criteria used to demonstrate the adequacy of structural elements and systems to withstand the calculated force and deformation demands togther with that laboratory and other data used to substantiate these criteria,
3. Review the preliminary design including the determination of the expected inelastic displacement of the *structure* and the margins remaining beyond these *displacements*, and
4. Review of the final design of the entire structural system and all supporting analyses.