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:


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 deformations 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, \( \Omega_p \), and deflection amplification factor, \( C_{db} \), 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, \( \Omega_p \), and deflection amplification factor, \( C_{db} \), 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 \( \Omega_p \) 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, \( \Omega_p \), 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.

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.
| Basic Seismic-Force-Resisting System | Detailing Reference Section | Response Modification Coefficient, \( R^a \) | System Overstrength Factor, \( \Omega^a \) | Deflection Amplification Factor, \( C_d^b \) | System Limitations and Building Height Limitations (ft) by Seismic Design Category
<table>
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<td>3½</td>
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<td>System Overstrength Factor, $Q^a$</td>
<td>Deflection Amplification Factor, $C_d^b$</td>
<td>System Limitations and Building Height Limitations (ft) by Seismic Design Category $^c$</td>
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<th>System Over-strength Factor, $Q'$</th>
<th>Deflection Amplification Factor, $C_{d}^{b}$</th>
<th>System Limitations and Building Height Limitations (ft) by Seismic Design Category $^{c}$</th>
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**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 | 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 | NL |
| Special steel concentrically braced frames | AISC Seismic, Part I, Sec. 13 | 8 | 2½ | 6½ | NL | NL | NL | NL | NL | NL |
| Special reinforced concrete shear walls | 9.3.2.4 | 8 | 2½ | 6½ | NL | NL | NL | NL | NL | NL |
| Ordinary reinforced concrete shear walls | 9.3.2.3 | 7 | 2½ | 6 | NL | NL | NP | NP | NP | NP |
| Composite eccentrically braced frames | AISC Seismic, Part II, Sec. 14 | 8 | 2½ | 4 | NL | NL | NL | NL | NL | NL |
| Composite concentrically braced frames | AISC Seismic, Part II, Sec. 13 | 6 | 2½ | 5 | NL | NL | NL | NL | NL | NL |
| Composite steel plate shear walls | AISC Seismic, Part II, Sec. 17 | 8 | 2½ | 6½ | NL | 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 | NL |
| Ordinary composite reinforced concrete shear walls with steel elements | AISC Seismic, Part II, Sec. 15 | 7 | 2½ | 6 | NL | NL | NP | NP | NP | NP |
| Special reinforced masonry shear walls | 11.11.5 | 7 | 3 | 6½ | NL | NL | NL | NL | NL | NL |
| Intermediate reinforced masonry shear walls | 11.11.4 | 6½ | 3 | 5½ | NL | NL | NL | NP | NP | NP |

**Dual Systems with Intermediate Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces**
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<th>Basic Seismic-Force-Resisting System</th>
<th>Detailing Reference Section</th>
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<th>Deflection Amplification Factor, $C_d^b$</th>
<th>System Limitations and Building Height Limitations (ft) by Seismic Design Category$^c$</th>
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</table>

$^a$ Response Modification Coefficient, $R^a$ is a factor used to adjust the expected response of a structure to seismic forces.

$^b$ System Overstrength Factor, $Q^b$ is a factor used to account for the additional strength provided by the structure.

$^c$ Deflection Amplification Factor, $C_d^b$ is a factor used to account for the amplification of deflections due to seismic forces.

$^d$ System Limitations and Building Height Limitations (ft) by Seismic Design Category:
- **B**: Basic Limitations
- **C**: Code Limitations
- **D**: Design Limitations
- **E**: Evaluation Limitations
- **F**: Feasibility Limitations

$^e$ Specific values may vary depending on the design category and local code requirements.

$^f$ Special steel concentrically braced frames are structural systems designed to resist seismic forces using steel elements in a concentric arrangement.

$^g$ Ordinary reinforced concrete shear walls are structural systems designed to resist seismic forces using reinforced concrete elements.

$^h$ Intermediate reinforced masonry shear walls are structural systems designed to resist seismic forces using masonry elements with intermediate reinforcement.

$^i$ Composite structures combine steel and concrete elements to enhance seismic performance.

$^j$ Special provisions may apply for specific seismic conditions or design criteria.
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, $Q_o$, 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

<table>
<thead>
<tr>
<th>Irregularity Type and Description</th>
<th>Reference Section</th>
<th>Seismic Design Category Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a Torsional Irregularity – to be considered when diaphragms are not flexible</td>
<td>5.2.6.4.2</td>
<td>D, E, and F</td>
</tr>
<tr>
<td>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.</td>
<td>5.4.4</td>
<td>C, D, E, and F</td>
</tr>
<tr>
<td>1b Extreme Torsional Irregularity -- to be considered when diaphragms are not flexible</td>
<td>5.2.6.4.2</td>
<td>D, E, and F</td>
</tr>
<tr>
<td>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.</td>
<td>5.4.4</td>
<td>C, D, E, and F</td>
</tr>
<tr>
<td>2 Re-entrant Corners</td>
<td>5.2.6.4.2</td>
<td>D, E, and F</td>
</tr>
<tr>
<td>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.</td>
<td>5.2.6.2.10</td>
<td>B, C, D, E, and F</td>
</tr>
<tr>
<td>3 Diaphragm Discontinuity</td>
<td>5.2.6.4.2</td>
<td>D, E, and F</td>
</tr>
<tr>
<td>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.</td>
<td>5.2.6.2.10</td>
<td>B, C, D, E, and F</td>
</tr>
<tr>
<td>4 Out-of-Plane Offsets</td>
<td>5.2.6.4.2</td>
<td>D, E, and F</td>
</tr>
<tr>
<td>Discontinuities in a lateral force resistance path such as out-of-plane offsets of the vertical elements.</td>
<td>5.2.6.2.10</td>
<td>B, C, D, E, and F</td>
</tr>
<tr>
<td>5 Nonparallel Systems</td>
<td>5.2.5.2</td>
<td>C, D, E, and F</td>
</tr>
<tr>
<td>The vertical lateral-force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force-resisting system.</td>
<td>5.2.5.2</td>
<td>C, D, E, and F</td>
</tr>
<tr>
<td>Irregularity Type and Description</td>
<td>Reference Section</td>
<td>Seismic Design Category Application</td>
</tr>
<tr>
<td>-----------------------------------------------------------------------------</td>
<td>-------------------</td>
<td>------------------------------------</td>
</tr>
<tr>
<td>1a Stiffness Irregularity – Soft Story</td>
<td>5.2.5.1</td>
<td>D, E, and F</td>
</tr>
<tr>
<td>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.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1b Stiffness Irregularity--Extreme Soft Story</td>
<td>5.2.5.1</td>
<td>D, E, and F</td>
</tr>
<tr>
<td>5.2.6.5.1</td>
<td>E and F</td>
<td></td>
</tr>
<tr>
<td>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.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 Weight (Mass) Irregularity</td>
<td>5.2.5.1</td>
<td>D, E, and F</td>
</tr>
<tr>
<td>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.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 Vertical Geometric Irregularity</td>
<td>5.2.5.1</td>
<td>D, E, and F</td>
</tr>
<tr>
<td>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.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 In-Plane Discontinuity in Vertical Lateral-Force Resisting Elements</td>
<td>5.2.5.1</td>
<td>D, E, and F</td>
</tr>
<tr>
<td>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.</td>
<td>5.2.6.2.10</td>
<td>B, C, D, E, and F</td>
</tr>
<tr>
<td>5.2.6.4.2</td>
<td>D, E, and F</td>
<td></td>
</tr>
<tr>
<td>5 Discontinuity in Capacity – Weak Story</td>
<td>5.2.6.2.3</td>
<td>B, C, D, E, and F</td>
</tr>
<tr>
<td>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.</td>
<td>5.2.5.1</td>
<td>D, E, and F</td>
</tr>
<tr>
<td>5.2.6.5.1</td>
<td>E and F</td>
<td></td>
</tr>
</tbody>
</table>
5.2.4 Redundancy: A reliability factor, $\rho$, 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 $\rho$ may be taken as 1.0.

5.2.4.2 Seismic Design Category D: For structures in Seismic Design Category D, $\rho$ shall be taken as the largest of the values of $\rho_x$ calculated at each story of the structure “x” in accordance with Eq. 5.2.4.2:

$$\rho_x = 2 - \frac{20}{r_{\text{max}_x} \sqrt{A_x}}$$

(5.2.4.2)

where:

$r_{\text{max}_x} =$ 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 $r_{\text{max}_x}$ is equal to the lateral force component in the most heavily loaded brace element divided by the story shear. For moment frames, $r_{\text{max}_x}$ 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, $r_{\text{max}_x}$ shall be taken equal to the maximum ratio, $r_{\text{ix}}$, 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, $r_{\text{max}_x}$ 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 $\rho$ 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 $\rho$ need not exceed 1.5, which is permitted to be used for any structure. The value of $\rho$ 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 $\rho$ calculated in accordance with this section does not exceed 1.25.

The metric equivalent of Eq. 5.2.4.2 is:
\[
\rho_x = 2 - \frac{6.1}{r_{\text{max}} \sqrt{A_x}}
\]

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 \( \rho \) 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 \( \rho \) 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

<table>
<thead>
<tr>
<th>Seismic Design Category</th>
<th>Structural Characteristics</th>
<th>Index Force Analysis, Sec. 5.3</th>
<th>Equivalent Lateral Force Analysis, Sec. 5.4</th>
<th>Modal Response Spectrum Analysis, Sec. 5.5</th>
<th>Linear Response History Analysis, Sec. 5.6</th>
<th>Nonlinear Response History Analysis, Sec. 5.7</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Regular or irregular</td>
<td>P</td>
<td>P</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>B, C</td>
<td>Regular or irregular</td>
<td>NP</td>
<td>P</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>D, E, F</td>
<td>Regular <em>structures</em> with $T &lt; 3.5T_s$ and all <em>structures</em> of light frame construction</td>
<td>NP</td>
<td>P</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td></td>
<td>Irregular <em>structures</em> with $T &lt; 3.5T_s$ 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.</td>
<td>NP</td>
<td>P</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td></td>
<td>Irregular <em>structures</em> with $T &lt; 3.5T_s$ 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.</td>
<td>NP</td>
<td>NP</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td></td>
<td>All other structures</td>
<td>NP</td>
<td>NP</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
</tbody>
</table>

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, \( S_{dps} \), 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, $C_p$, 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, $V_s$, 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 $Q_{EI}$ 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:

$$F_p = 1.2 S_{DS} W_p$$

(5.2.6.3.2)
where:

\[ F_p \quad = \quad \text{the design force in the individual anchors}, \]
\[ S_{DS} \quad = \quad \text{the design spectral response acceleration at short periods in accordance with Sec. 4.1.2.5}, \]
\[ I \quad = \quad \text{the occupancy importance factor in accordance with Sec. 1.4}, \]
\[ W_p \quad = \quad \text{the weight of the wall tributary to the anchor}. \]

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 \( \Omega_{SE} \) 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:

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

(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 0.4$S_{D3} w_{px}$ but shall not be less than 0.2$S_{D3} w_{px}$. 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:

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

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,
- $\rho$ = 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:

$$E = \rho Q_E - 0.2 S_{DS} D$$  \hspace{1cm} (5.2.7-2)

where $E$, $\rho$, $Q_E$, $S_{DS}$, 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:

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

$$E = \Omega_0 Q_E - 0.2 S_{DS} D$$  \hspace{1cm} (5.2.7.1-2)
where $E$, $Q_E$, $S_{DS}$, and $D$ are as defined above and $\Omega_0$ is the system overstrength factor as given in Table 5.2.2. The term $\Omega_0Q_E$ 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, $\Delta$, as determined in Sec. 5.3.7 or 5.4.6 shall not exceed the allowable story drift, $\Delta_a$, 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, $\delta_x$, as determined in Sec. 5.3.7.1.

**TABLE 5.2.8 Allowable Story Drift, $\Delta_a$ (in. or mm)**

<table>
<thead>
<tr>
<th>Structure</th>
<th>Seismic Use Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>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</td>
<td>I</td>
</tr>
<tr>
<td>Masonry cantilever shear wall structures $^c$</td>
<td>0.025 $h_{sx}$ $^b$</td>
</tr>
<tr>
<td>Other masonry shear wall structures</td>
<td>0.007 $h_{sx}$</td>
</tr>
<tr>
<td>Masonry wall frame structures</td>
<td>0.013 $h_{sx}$</td>
</tr>
<tr>
<td>All other structures</td>
<td>0.020 $h_{sx}$</td>
</tr>
</tbody>
</table>

$^a$ $h_{sx}$ 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:

\[ F_x = 0.01 w_x \]  

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/m²) 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:

\[ V = C_s W \]  

where:

- \( C_s \) = the seismic response coefficient determined in accordance with Sec. 5.4.1.1 and
$W = \text{the total dead load and applicable portions of other loads as defined in Sec. 5.3.}$

### 5.4.1.1 Calculation of Seismic Response Coefficient:

The seismic response coefficient, $C_s$, shall be determined in accordance with the following equation:

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

where:

- $S_{DS} = \text{the design spectral response acceleration in the short period range as determined from Sec. 4.1.2.5,}$
- $R = \text{the response modification factor from Table 5.2.2, and}$
- $I = \text{the occupancy importance factor determined in accordance with Sec. 1.4.}$

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

$$C_s = \frac{S_{DI}}{T(R/I)} \quad (5.4.1.1-2)$$

where $I$ and $R$ are as defined above and

- $S_{DI} = \text{the design spectral response acceleration at a period of 1.0 second as determined from Sec. 4.1.2.5,}$
- $T = \text{the fundamental period of the structure (sec) determined in Sec. 5.4.2, and}$
- $S_1 = \text{the mapped maximum considered earthquake spectral response acceleration determined in accordance with Sec. 4.1.}$

$C_s$ shall not be taken less than:

$$C_s = 0.044 IS_{DS} \quad (5.4.1.1-3)$$

For structures in Seismic Design Categories E and F, the value of the seismic response coefficient, $C_s$, shall not be taken less than:

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

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 \( S_J \).

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**

<table>
<thead>
<tr>
<th>Design Spectral Response Acceleration at 1 Second, ( S_{pl} )</th>
<th>Coefficient ( C_u )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Greater than or equal to 0.4</td>
<td>1.4</td>
</tr>
<tr>
<td>0.3</td>
<td>1.4</td>
</tr>
<tr>
<td>0.2</td>
<td>1.5</td>
</tr>
<tr>
<td>0.15</td>
<td>1.6</td>
</tr>
<tr>
<td>0.1</td>
<td>1.7</td>
</tr>
<tr>
<td>Less than or equal to 0.05</td>
<td>1.7</td>
</tr>
</tbody>
</table>

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

\[
T_a = C_r h_n^x
\]  

(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 $x$

<table>
<thead>
<tr>
<th>Structure Type</th>
<th>$C_{OT}$</th>
<th>$x$</th>
</tr>
</thead>
<tbody>
<tr>
<td>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.</td>
<td>0.028 (metric 0.0724)</td>
<td>0.8</td>
</tr>
<tr>
<td>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.</td>
<td>0.016 (metric 0.0466)</td>
<td>0.9</td>
</tr>
<tr>
<td>Eccentrically braced steel frames</td>
<td>0.03 (metric 0.0731)</td>
<td>0.75</td>
</tr>
<tr>
<td>All other structural systems</td>
<td>0.02 (metric 0.0488)</td>
<td>0.75</td>
</tr>
</tbody>
</table>

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):

$$T_a = 0.1N$$  \hspace{1cm} (5.4.2.1-2)

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:

$$T_a = \frac{0.0019}{\sqrt{C_w}} h_n T_a = \frac{0.0062}{\sqrt{C_w}} h_n$$  \hspace{1cm} (5.4.2.1-3)

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:

$$C_w = \frac{100}{A_R} \sum_{i=1}^{n} \left( \frac{h_n}{h_i} \right) \frac{A_i}{1 + 0.83 \left( \frac{h_n}{D} \right)^2}$$  \hspace{1cm} (5.4.2.1-4)
where:

\[ A_B = \text{the base area of the structure (ft}^2\text{ or m}^2\text{)}, \]

\[ A_i = \text{the area of shear wall } i \text{ (ft}^2\text{ or m}^2\text{)}, \]

\[ D_i = \text{the length of shear wall } i \text{ (ft or m)}, \]

\[ h_i = \text{the height of shear wall } i \text{ (ft or m), and} \]

\[ n = \text{the number of shear walls in the building effective in resisting lateral forces in the direction under consideration}. \]

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:

\[ F_x = C_{vx} V \quad (5.4.3-1) \]

and

\[ C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \quad (5.4.3-2) \]

where:

\[ C_{vx} = \text{vertical distribution factor}, \]

\[ V = \text{total design lateral force or shear at the base of the structure (kip or kN)}, \]

\[ w_i \text{ and } w_x = \text{the portion of the total gravity load of the structure, } W, \text{ located or assigned to Level } i \text{ or } x, \]

\[ h_i \text{ and } h_x = \text{the height (ft or m) from the base to Level } i \text{ or } x, \text{ and} \]

\[ k = \text{an exponent related to the structure period as follows:} \]

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:

\[ V_x = \sum_{i=x}^{n} F_i \quad (5.4.4) \]
where \( F_i \) = the portion of the seismic \textit{base shear}, \( V \) (kip or kN), induced at Level \( i \).

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

\textbf{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.

\textbf{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 \textit{displacement} of the mass each way from its actual location by a distance equal to 5 percent of the dimension of the \textit{structure} perpendicular to the direction of the applied forces.

\textbf{5.4.4.3 Dynamic Amplification of Torsion:} For \textit{structures} of \textit{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:

\[
A_x = \left( \frac{\delta_{\text{max}}}{1.2 \delta_{\text{avg}}} \right)^2 \quad (5.4.4.3-1)
\]

where:

\( \delta_{\text{max}} \) = the maximum \textit{displacement} at Level \( x \) (in. or mm) and

\( \delta_{\text{avg}} \) = the average of the \textit{displacements} at the extreme points of the \textit{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.

\textbf{5.4.5 Overturning:} The \textit{structure} shall be designed to resist overturning effects caused by the \textit{seismic forces} determined in Sec. 5.3.4. At any \textit{story}, the increment of overturning moment in the \textit{story} under consideration shall be distributed to the various vertical-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:

\[
M_x = \sum_{i=x}^{n} F_i (h_i - h_x) \quad (5.4.5)
\]

where:
$F_i = \text{the portion of the seismic base shear, } V, \text{ induced at Level } i \text{ and}$

$h_i \text{ and } h_x = \text{the height (ft or m) from the base to Level } i \text{ or } x,$

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, $\Delta_i$, 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, $\Delta_i$, 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 $x$, $\delta_x$ (in. or mm), shall be determined in accordance with following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \tag{5.4.6.1}$$

where:

$C_d = \text{the deflection amplification factor in Table 5.2.2},$

$\delta_{xe} = \text{the deflections determined by an elastic analysis (in. or mm)},$ and

$I = \text{the occupancy importance factor determined in accordance with Sec. 1.4}.$

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 \(x\), \(\delta_x\) (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, \(\Delta\) (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, \(\theta\), as determined by the following equation is equal to or less than 0.10:

\[
\theta = \frac{P_x \Delta}{V_x h_{xx} C_d} \tag{5.4.6.2-1}
\]

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.
- \(\Delta\) = 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).
- \(h_{xx}\) = the *story* height below Level \(x\) (in. or mm).
- \(C_d\) = the deflection amplification factor in Table 5.2.2.

The stability coefficient, \(\theta\), shall not exceed \(\theta_{\text{max}}\) determined as follows:

\[
\theta_{\text{max}} = \frac{0.5}{\beta C_d} \leq 0.25 \tag{5.4.6.2-2}
\]

where \(\beta\) 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, \(\theta\), is greater than 0.10 but less than or equal to \(\theta_{\text{max}}\), 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 \(1.0/(1 - \theta)\).

When \(\theta\) is greater than \(\theta_{\text{max}}\), 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 \( m^{\text{th}} \) 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 \( \phi \), 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 \( m^{\text{th}} \) mode, \( V_m \), shall be determined from the following equations:

\[
V_m = C_{sm} \overline{W}_m \quad \text{(5.5.4-1)}
\]

\[
\overline{W}_m = \frac{\left( \sum_{i=1}^{n} w_i \phi_{im} \right)^2}{\sum_{i=1}^{n} w_i \phi_{im}^2} \quad \text{(5.5.4-2)}
\]
where:

\[ C_{sm} = \text{the modal seismic response coefficient as determined by Eq. 5.5.4-3,} \]
\[ W_m = \text{the effective modal gravity load including portions of the live load as defined in Sec. 5.3,} \]
\[ w_i = \text{the portion of the total gravity load of the structure at Level } i, \]
\[ \phi_{im} = \text{the displacement amplitude at the } i^{\text{th}} \text{ level of the structure when vibrating in its } m^{\text{th}} \text{ mode.} \]

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

\[ C_{sm} = \frac{S_{am}}{R/I} \]  \hspace{1cm} (5.5.4-3)

where:

\[ S_{am} = \text{The design spectral response acceleration at period } T_m \text{ determined from either the general design response spectrum of Sec. 4.1.2.5 or a site-specific response spectrum determined in accordance with Sec. 4.1.3,} \]
\[ R = \text{the response modification factor determined from Table 5.2.2,} \]
\[ I = \text{the occupancy importance factor determined in accordance with Sec. 1.4,} \]
\[ T_m = \text{the modal period of vibration (in seconds) of the } m^{\text{th}} \text{ mode of the structure.} \]

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:

\[ C_{sm} = \frac{0.4 S_{DS}}{(R/I)} \left(1.0 + 5.0 T_m\right) \]  \hspace{1cm} (5.5.4-4)

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:
\[ C_{sm} = \frac{4 S_{D1}}{(R/I) T_m^2} \] (5.5.4-5)

where \( R, I, \) and \( T_m \) are as defined above 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:

\[ F_{xm} = C_{vxm} V_m \] (5.5.5-1)

and

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

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 \( x \),
- \( \phi_{xm} \) = the displacement amplitude at the \( x^{th} \) level of the structure when vibrating in its \( m^{th} \) mode, and
- \( \phi_{im} \) = the displacement amplitude at the \( i^{th} \) level of the structure when vibrating in its \( m^{th} \) mode.

The modal deflection at each level, \( \delta_{xm} \), shall be determined by the following equations:

\[ \delta_{xm} = \frac{C_d \delta_{xem}}{I} \] (5.5.5-3)

and
\[ \delta_{sem} = \left( \frac{g}{4\pi^2} \right) \left( \frac{T_m^2 F_{xm}}{w_x} \right) \]  
(5.5.5-4)

where:

- \( C_d \) = the deflection amplification factor determined from Table 5.2.2,
- \( \delta_{sem} \) = 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, \( \Delta_m \), shall be computed as the difference of the deflections, \( \delta_{sem} \), 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:

\[ 0.85 \frac{V}{V_t} \]  
(5.5.7.1)
where:

\[ V = \text{the equivalent lateral force procedure } base\ shear \text{ calculated in accordance with Sec. 5.4} \]

\[ V_t = \text{the modal } base\ shear \text{ calculated in accordance with this section.} \]

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 0.2\( T \) to 1.5\( T \) 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, \(\delta_j\), 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, \(\Delta\), 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 \(\delta_j\) 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, \(\Delta\), shall be taken as the maximum value of the scaled \(Q_{Ej}\) and \(\delta_j\) 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 \(Q_0Q_E\) 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 overstrenth, 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, $\gamma_j$, and interstory drifts, $\delta_{ij}$, 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, $\gamma_i$, and interstory drift, $\Delta$, shall be taken, respectively, as the average of the scaled $Q_{Ej}$, $\gamma_i$, and $\delta_{ij}$ values determined from the analyses. If less than seven ground motions are analyzed, the design member forces, $Q_E$, design member inelastic deformations, $\gamma_i$, and the design interstory drift, $\Delta$, shall be taken as the maximum value of the scaled $Q_{Ej}$, $\gamma_i$, and $\delta_{ij}$ 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 $Q_E$ obtained from the suite of analyses shall be taken in place of the quantity $Q_dQ_E$.

5.7.3.2 Member Deformation: The adequacy of individual members and their connections to withstand the design deformation values, $\gamma_i$, 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:

$$\tilde{V} = V - \Delta V$$

(5.8.2.1-1)

The reduction, $\Delta V$, shall be computed as follows:

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

(5.8.2.1-2)

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

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

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:

\[ \tilde{T} = T \sqrt{1 + \frac{\overline{K}}{K_y} \left(1 + \frac{K_y h^2}{K_{\theta}}\right)} \]  

(5.8.2.1.1-1)

where:

\( T \) = the fundamental period of the structure as determined in Sec. 5.4.2;

\( \overline{K} \) = the stiffness of the structure when fixed at the base, defined by the following:

\[ \overline{K} = 4\pi^2 \left(\frac{\overline{W}}{g T^2}\right) \]  

(5.8.2.1.1-2)

\( \overline{h} \) = 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;

\( K_{\theta} \) = 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 \( K_{\theta} \), 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, \( G_o \), 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),

\( G_o = \gamma v_{so}^2 / g \) = the average shear modulus for the soils beneath the foundation at small strain levels, and
\[ \gamma = \text{the average unit weight of the soils.} \]

### TABLE 5.8.2.1.1 Values of \( G/G_o \) and \( v/v_{so} \)

<table>
<thead>
<tr>
<th>Peak Ground Acceleration, (g)</th>
<th>( \leq 0.10 )</th>
<th>( \leq 0.15 )</th>
<th>0.20</th>
<th>( \geq 0.30 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value of ( G/G_o )</td>
<td>0.81</td>
<td>0.64</td>
<td>0.49</td>
<td>0.42</td>
</tr>
<tr>
<td>Value of ( v/v_{so} )</td>
<td>0.90</td>
<td>0.80</td>
<td>0.70</td>
<td>0.65</td>
</tr>
</tbody>
</table>

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:

\[
\bar{T} = T \sqrt{1 + \frac{25\alpha r_a h}{v_s^2 T^2} \left( 1 + \frac{1.12 r_a h^2}{\alpha_0 r_m^3} \right)} \quad \text{(5.8.2.1.1-3)}
\]

where:

\[ \alpha = \frac{\bar{W}}{\gamma A_o \bar{h}} \quad \text{(5.8.2.1.1-4)} \]

\( r_a \) and \( r_m \) = characteristic foundation lengths defined by:

\[ r_a = \sqrt{\frac{A_o}{\pi}} \quad \text{(5.8.2.1.1-5)} \]

and

\[ r_m = \sqrt[4]{\frac{4 A_o}{\pi}} \quad \text{(5.8.2.1.1-6)} \]

where:

\[ A_o = \text{the area of the foundation,} \]
\[ I_o = \text{the static moment of the foundation about a horizontal centroidal axis normal to the direction in which the structure is analyzed, and} \]

### 5.8.2.1.2 Effective Damping:

The effective damping factor for the structure-foundation system, \( \hat{\beta} \), shall be computed as follows:

\[
\hat{\beta} = \beta_o + \frac{0.05}{\left( \frac{T}{T'} \right)^3}
\]

(5.8.2.1.2-1)

where \( \beta_o \) = the foundation damping factor as specified in Figure 5.8.2.1.2.

The values of \( \beta_o \) 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 \( \overline{h}/L_o \leq 0.5 \),

\[
r = r_a = \sqrt{\frac{A_o}{\pi}}
\]

(5.8.2.1.2-2)

For \( \overline{h}/L_o \geq 1 \),

\[
r = r_m = \sqrt[4]{\frac{4I_o}{\pi}}
\]

(5.8.2.1.2-3)

where:

\[
L_o = \text{the overall length of the side of the foundation in the direction being analyzed},
\]

\[
A_o = \text{the area of the load-carrying foundation, and}
\]

\[
I_o = \text{the static moment of inertia of the load-carrying foundation.}
\]
For intermediate values of \( \frac{h}{L_0} \), 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 \( \beta_o \) in Eq. 5.8.2.1.2-1 shall be replaced by:

\[
\beta_o' = \left( \frac{4D_s}{V_s \tilde{T}} \right)^2 \beta_o
\]  

(5.8.2.1.2-4)

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 structures without interaction using the reduced lateral forces.

The modified deflections, $\delta_x$, shall be determined as follows:

$$\delta_x = \frac{\tilde{V}}{V} \left( \frac{M_o h_x}{K_0} + \delta_\times \right)$$

(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
- $\delta_\times$ = 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, $V_f$, is permitted to be reduced to:

$$\tilde{V}_1 = V_1 - \Delta V_1$$

(5.8.3.1-1)

The reduction, $\Delta V_1$, 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, $C$, computed from Eq. 5.5.4-3 using the fundamental period of the fixed-base structure, $T_f$, and $\tilde{C}_f$ computed from Eq. 5.5.4-3 using the fundamental period of the elastically supported structure, $\tilde{T}_f$.

The period $\tilde{T}_f$ shall be determined from Eq. 5.8.2.1.1-1, or from Eq. 5.8.2.1.1-3 when applicable, taking $T = \tilde{T}_f$, evaluating $K$ from Eq. 5.8.2.1.1-2 with $\overline{W} = \overline{W}_1$, and computing $\tilde{h}$ as follows:
The above designated values of \( \overline{W}, \overline{H}, T, \) and \( \hat{T} \) also shall be used to evaluate the factor \( \alpha \) from Eq. 5.8.2.1.1-4 and the factor \( \beta_o \) 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, \( \hat{V}_1 \), shall in no case be taken less than 0.7\( V_1 \).

### 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, \( \hat{V}_1 \), instead of \( V_1 \). The modified modal deflections, \( \delta_{xm} \), shall be determined as follows:

\[
\hat{\delta}_{xm} = \frac{\hat{V}_1}{V_1} \left[ \frac{M_{o1} h_x}{K_\theta} + \delta_{xl} \right] 
\]

for \( m = 2, 3, \ldots \) (5.8.3.2-1)

and

\[
\hat{\delta}_{xm} = \delta_x \\
\text{for } m = 2, 3, \ldots \quad (5.8.3.2-2)
\]

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
- \( \delta_{xm} \) = 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, \( \Delta_m \), shall be computed as the difference of the deflections, \( \delta_{xm} \), 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 Methodologies 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, \( Q_i \), and member deformations, \( \gamma_i \), 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, $\Delta_k$, 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:

$$C_i = \frac{(1 - T_s/T_1)}{R_d} + (T_s/T_1)$$

(5A.1.3-1)

where $T_s$ and $T_1$ are as defined above and $R_d$ is given by the following equation:

$$R_d = \frac{1.5R}{\Omega_o}$$

(5A.1.3-2)

where $R$ and $\Omega_o$ 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 $\Omega_{ei}$ obtained from the analysis at the expected inelastic deformation, as calculated from Sec. 5A.1.3, shall be taken in place of the quantity $\Omega_0Q_E$.

5A.1.4.2 Member Deformation: The adequacy of individual members and their connections to withstand the design deformation values, $\gamma_i$, 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 together 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.