1.7 LOAD TESTS

A load test of any construction shall be conducted when required by the authority having jurisdiction whenever there is reason to question its safety for the intended occupancy or use.

1.8 CONSENSUS STANDARDS AND OTHER REFERENCED DOCUMENTS

This section lists the consensus standards and other documents which are adopted by reference within this chapter.

<table>
<thead>
<tr>
<th>Nature of Occupancy</th>
<th>Occupancy Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to:</td>
<td>I</td>
</tr>
<tr>
<td>• Agricultural facilities</td>
<td></td>
</tr>
<tr>
<td>• Certain temporary facilities</td>
<td></td>
</tr>
<tr>
<td>• Minor storage facilities</td>
<td></td>
</tr>
<tr>
<td>All buildings and other structures except those listed in Occupancy Categories I, III, and IV</td>
<td>II</td>
</tr>
<tr>
<td>Buildings and other structures that represent a substantial hazard to human life in the event of failure, including, but not limited to:</td>
<td>III</td>
</tr>
<tr>
<td>• Buildings and other structures where more than 300 people congregate in one area</td>
<td></td>
</tr>
<tr>
<td>• Buildings and other structures with daycare facilities with a capacity greater than 150</td>
<td></td>
</tr>
<tr>
<td>• Buildings and other structures with elementary school or secondary school facilities with a capacity greater than 250</td>
<td></td>
</tr>
<tr>
<td>• Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities</td>
<td></td>
</tr>
<tr>
<td>• Health care facilities with a capacity of 50 or more resident patients, but not having surgery or emergency treatment facilities</td>
<td></td>
</tr>
<tr>
<td>• Jails and detention facilities</td>
<td></td>
</tr>
<tr>
<td>Buildings and other structures, not included in Occupancy Category IV, with potential to cause a substantial economic impact and/or mass disruption of daily-to-citizen civilian life in the event of failure, including, but not limited to:</td>
<td>IV</td>
</tr>
<tr>
<td>• Power generating stations</td>
<td></td>
</tr>
<tr>
<td>• Water treatment facilities</td>
<td></td>
</tr>
<tr>
<td>• Sewage treatment facilities</td>
<td></td>
</tr>
<tr>
<td>• Telecommunication centers</td>
<td></td>
</tr>
<tr>
<td>Buildings and other structures not included in Occupancy Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.</td>
<td></td>
</tr>
<tr>
<td>Buildings and other structures containing toxic or explosive substances shall be eligible for classification as Occupancy Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.7 that a release of the toxic or explosive substances does not pose a threat to the public.</td>
<td></td>
</tr>
<tr>
<td>Buildings and other structures designated as essential facilities, including, but not limited to:</td>
<td></td>
</tr>
<tr>
<td>• Hospitals and other health care facilities having surgery or emergency treatment facilities</td>
<td></td>
</tr>
<tr>
<td>• Fire, rescue, ambulance, and police stations and emergency vehicle garages</td>
<td></td>
</tr>
<tr>
<td>• Designated earthquake, hurricane, or other emergency shelters</td>
<td></td>
</tr>
<tr>
<td>• Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response</td>
<td></td>
</tr>
<tr>
<td>• Power generating stations and other public utility facilities (required in an emergency)</td>
<td></td>
</tr>
<tr>
<td>• Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire suppression material or equipment) required for operation of Occupancy Category IV structures during an emergency</td>
<td></td>
</tr>
<tr>
<td>• Aviation control towers, air traffic control centers, and emergency aircraft hangars</td>
<td></td>
</tr>
<tr>
<td>• Water storage facilities and pump structures required to maintain water pressure for fire suppression</td>
<td></td>
</tr>
<tr>
<td>• Buildings and other structures having critical national defense functions</td>
<td></td>
</tr>
<tr>
<td>Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing highly toxic substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction.</td>
<td></td>
</tr>
<tr>
<td>Buildings and other structures containing highly toxic substances shall be eligible for classification as Occupancy Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.7 that a release of the highly toxic substances does not pose a threat to the public. This reduced classification shall not be permitted if the buildings or other structures also function as essential facilities.</td>
<td></td>
</tr>
</tbody>
</table>

*Cogeneration power plants that do not supply power on the national grid shall be designated Occupancy Category II.
Chapter 2
COMBINATIONS OF LOADS

2.1 GENERAL
Buildings and other structures shall be designed using the provisions of either Section 2.3 or 2.4. Either Section 2.3 or 2.4 shall be used exclusively for proportioning elements of a particular construction material throughout the structure.

2.2 SYMBOLS AND NOTATION
\( \begin{align*}
D &= \text{dead load} \\
D_i &= \text{weight of ice} \\
E &= \text{earthquake load} \\
F &= \text{load due to fluids with well-defined pressures and maximum heights} \\
F_a &= \text{flood load} \\
H &= \text{load due to lateral earth pressure, ground water pressure, or pressure of bulk materials} \\
L &= \text{live load} \\
L_r &= \text{roof live load} \\
R &= \text{rain load} \\
S &= \text{snow load} \\
T &= \text{self-straining force} \\
W &= \text{wind load} \\
W_i &= \text{wind-on-ice determined in accordance with Chapter 10}
\end{align*} \)

Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in \( H \) but shall be included in the design resistance.

3. In combinations (2), (4), and (5), the companion load \( S \) shall be taken as either the flat roof snow load \( (p_f) \) or the sloped roof snow load \( (p_s) \).

Each relevant strength limit state shall be investigated. Effects of one or more loads not acting shall be investigated. The most unfavorable effects from both wind and earthquake loads shall be investigated, where appropriate, but they need not be considered to act simultaneously. Refer to Section 12.4 for specific definition of the earthquake load effect \( E \).\(^1\)

2.3.3 Load Combinations Including Flood Load. When a structure is located in a flood zone (Section 5.3.1), the following load combinations shall be considered:

1. In V-Zones or Coastal A-Zones, \( 1.6W \) in combination (4) and (6) shall be replaced by \( 1.6W + 2.0F_a \).
2. In noncoastal A-Zones, \( 1.6W \) in combinations (4) and (6) shall be replaced by \( 0.8W + 1.0F_a \).

2.3.4 Load Combinations Including Atmospheric Ice Loads. When a structure is subjected to atmospheric ice and wind-on-ice loads, the following load combinations shall be considered:

1. \( 0.5(L_r + S) \text{ or } R \) in combination (2) shall be replaced by \( 0.2L_r + 0.3S \).
2. \( 1.6W + 0.5(L_r, \text{ or } S + R) \) in combination (4) shall be replaced by \( D_i + W + 0.5S \).
3. \( 1.6W \) in combination (6) shall be replaced by \( D_i + W_i \).

2.4 COMBINING NOMINAL LOADS USING ALLOWABLE STRESS DESIGN

2.4.1 Basic Combinations. Loads listed herein shall be considered to act in the following combinations; whichever produces the most unfavorable effect in the building, foundation, or structural member being considered. Effects of one or more loads not acting shall be considered.

\( \begin{align*}
1. & \quad D + F \\
2. & \quad D + H + F + L + T \\
3. & \quad D + H + F + L_r \text{ or } S + R \\
4. & \quad D + H + F + 0.75(L + T) + 0.75(L_r + S + R) \\
5. & \quad D + H + F + (W + 0.7E) \\
6. & \quad D + H + F + 0.75(W + 0.7E) + 0.75T + 0.75L_r \text{ or } S + R \\
7. & \quad 0.6D + W + H \\
8. & \quad 0.6D + 0.7E + H
\end{align*} \)

\( ^1 \) The same \( E \) from Section 12.4 is used for both Sections 2.3.3 and 2.4.1. Refer to the Chapter 11 Commentary for the Seismic Provisions.
\( \delta_{crm} = \text{deflection of Level } x \text{ at the center of the mass at and above Level } x \text{ determined by an elastic analysis, Section 12.8-6} \)

\( \delta_{re} = \text{modal deflection of Level } x \text{ at the center of the mass at and above Level } x \text{ as determined by Section 19.3.2} \)

\( \delta_{re}, \delta_{re} = \text{deflection of Level } x \text{ at the center of the mass at and above Level } x \text{, Eqs. 19.2-13 and 19.3-3} \) (in or mm)

\( \theta = \text{stability coefficient for P-delta effects as determined in Section 17.8.7} \)

\( \rho = \text{a redundancy factor based on the extent of structural redundancy present in a building as defined in Section 12.3.4} \)

\( \rho_s = \text{spiral reinforcement ratio for precast, prestressed piles in Sections 14.2.7.1.6 and 14.2.7.2.6} \)

\( \lambda = \text{line effect factor} \)

\( \Omega_0 = \text{overstrength factor as defined in Tables 12.2-1, 5.4-1, and 15.3-1} \)

11.4 SEISMIC GROUND MOTION VALUES

11.4.1 Mapped Acceleration Parameters. The parameters \( S_S \) and \( S_1 \) shall be determined from the 0.2 and 1.0 s spectral response accelerations shown in Figs. 22.1 through 22.14, respectively. Where \( S_1 \) is less than or equal to 0.04 and \( S_e \) is less than or equal to 0.15, the structure is permitted to be assigned to Seismic Design Category A and is only required to comply with Section 11.7.

11.4.2 Site Class. Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E, or F in accordance with Chapter 20. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the authority having jurisdiction or geotechnical data determines Site Class E or F soils are present at the site.

11.4.3 Site Coefficients and Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameters. The MCE spectral response acceleration for short periods (\( S_{MS} \)) and at 1 s (\( S_{M1} \)), adjusted for Site Class effects, shall be determined by Eqs. 11.4-1 and 11.4-2, respectively.

\[
S_{MS} = F_a S_S \quad (11.4-1)
\]

\[
S_{M1} = F_a S_1 \quad (11.4-2)
\]

where

\( S_S = \text{the mapped MCE spectral response acceleration at short periods as determined in accordance with Section 11.4.1 and} \)

\( S_1 = \text{the mapped MCE spectral response acceleration at a period of } 1 \text{ s as determined in accordance with Section 11.4.1} \)

where site coefficients \( F_a \) and \( F_a \) are defined in Tables 11.4-1 and 11.4-2, respectively. Where the simplified design procedure of Section 12.14 is used, the value of \( F_a \) shall be determined in accordance with Section 12.14.8.1, and the values for \( F_a, S_{MS} \), and \( S_{M1} \) need not be determined.

11.4.4 Design Spectral Acceleration Parameters. Design earthquake spectral response acceleration parameter at short period, \( S_{DS} \), and at 1 s period, \( S_{D1} \), shall be determined from Eqs. 11.4-3 and 11.4-4, respectively. Where the alternate simplified design procedure of Section 12.14 is used, the value of \( S_{DS} \) shall be determined in accordance with Section 12.14.8.1, and the value for \( S_{D1} \) need not be determined.

\[
S_{DS} = \frac{2}{3} S_{MS} \quad (11.4-3)
\]

\[
S_{D1} = \frac{2}{3} S_{M1} \quad (11.4-4)
\]

11.4.5 Design Response Spectrum. Where a design response spectrum is required by this standard and site specific ground motion procedures are not used, the design response spectrum curve shall be developed as indicated in Fig. 11.4-1 and as follows:

1. For periods less than \( T_0 \), the design spectral response acceleration, \( S_a \), shall be taken as given by Eq. 11.4-5:

\[
S_a = S_{DS} \left( 0.4 + 0.6 \frac{T}{T_0} \right) \quad (11.4-5)
\]

2. For periods greater than or equal to \( T_0 \) and less than or equal to \( T_S \), the design spectral response acceleration, \( S_a \), shall be taken equal to \( S_{DS} \).

---

**TABLE 11.4-2 SITE COEFFICIENT, \( F_a \)**

<table>
<thead>
<tr>
<th>Site Class</th>
<th>( S_S \leq 0.25 )</th>
<th>( S_S = 0.25 )</th>
<th>( S_S = 0.5 )</th>
<th>( S_S = 1.0 )</th>
<th>( S_S = 1.25 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.2</td>
<td>1.2</td>
<td>1.1</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
<td>1.4</td>
<td>1.2</td>
<td>1.1</td>
<td>1.0</td>
</tr>
<tr>
<td>E</td>
<td>2.5</td>
<td>1.7</td>
<td>1.2</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td>F</td>
<td>See Section 11.4.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTE:** Use straight-line interpolation for intermediate values of \( S_S \).

---

**FIGURE 11.4-1 DESIGN RESPONSE SPECTRUM**

---

Minimum Design Loads for Buildings and Other Structures
3. For periods greater than $T_L$, and less than or equal to $T_L$, the design spectral response acceleration, $S_a$, shall be taken as given by Eq. 11.4-6:

$$S_a = \frac{S_{D1}}{T}$$  \hspace{1cm} (11.4-6)

4. For periods greater than $T_L$, $S_a$ shall be taken as given by Eq. 11.4-7:

$$S_a = \frac{S_{D1}T_L}{T^2}$$  \hspace{1cm} (11.4-7)

where $S_{DS}$ = the design spectral response acceleration parameter at short periods

$S_{D1}$ = the design spectral response acceleration parameter at 1-s period

$T$ = the fundamental period of the structure, s

$T_0 = 0.2 \frac{S_{D1}}{S_{DS}}$

$T_5 = \frac{S_{D1}}{S_{DS}}$ and

$T_L$ = long-period transition period (s) shown in Fig. 22-15 (Continental United States), Fig. 22-16 (Region 1), Fig. 22-17 (Alaska), Fig. 22-18 (Hawaii), Fig. 22-19 (Puerto Rico, Culebra, Vieques, St. Thomas, St. John, and St. Croix), and Fig. 22-20 (Guam and Tutuila).

11.4.6 MCE Response Spectrum. Where a MCE response spectrum is required, it shall be determined by multiplying the design response spectrum by 1.5.

11.4.7 Site-Specific Ground Motion Procedures. The site-specific ground motion procedures set forth in Chapter 21 are permitted to be used to determine ground motions for any structure. A site response analysis shall be performed in accordance with Section 21.1 for structures on Site Class F sites, unless the exception to Section 20.3.1 is applicable. For seismically isolated structures and for structures with damping systems on sites with $S_1$ greater than or equal to 0.6, a ground motion hazard analysis shall be performed in accordance with Section 21.2.

11.5 IMPORTANCE FACTOR AND OCCUPANCY CATEGORY

11.5.1 Importance Factor. An importance factor, $I$, shall be assigned to each structure in accordance with Table 11.5-1 based on the Occupancy Category from Table 1-1.

11.5.2 Protected Access for Occupancy Category IV. Where operational access to an Occupancy Category IV structure is required through an adjacent structure, the adjacent structure shall conform to the requirements for Occupancy Category IV structures. Where operational access is less than 10 ft from an interior lot line or another structure on the same lot, protection from potential falling debris from adjacent structures shall be provided by the owner of the Occupancy Category IV structure.

<table>
<thead>
<tr>
<th>OCCUPANCY CATEGORY</th>
<th>IMPORTANCE FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>I or II</td>
<td>1.0</td>
</tr>
<tr>
<td>III</td>
<td>1.25</td>
</tr>
<tr>
<td>IV</td>
<td>1.5</td>
</tr>
</tbody>
</table>

11.6 SEISMIC DESIGN CATEGORY

Structures shall be assigned a Seismic Design Category in accordance with Section 11.6.1.1.

Occupancy Category I, II, or III structures located where the mapped spectral response acceleration parameter at 1-s period, $S_1$, is greater than or equal to 0.75 shall be assigned to Seismic Design Category E. Occupancy Category IV structures located where the mapped spectral response acceleration parameter at 1-s period, $S_1$, is greater than or equal to 0.75 shall be assigned to Seismic Design Category F. All other structures shall be assigned to a Seismic Design Category based on their Occupancy Category and the design spectral response acceleration parameters, $S_{DS}$ and $S_{D1}$, determined in accordance with Section 11.4.4. Each building and structure shall be assigned to the more severe Seismic Design Category in accordance with Table 11.6.1 or 11.6.2, irrespective of the fundamental period of vibration of the structure, $T$.

Where $S_1$ is less than 0.75, the Seismic Design Category is permitted to be determined from Table 11.6-1 alone where all of the following apply:

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, $T_0$, determined in accordance with Section 12.8.2.1 is less than $0.8T$, where $T_0$ is determined in accordance with Section 11.4.5.

2. In each of two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than $T_0$.

3. Eq. 12.8-2 is used to determine the seismic response coefficient $C_s$.

4. The diaphragms are rigid as defined in Section 12.3.1 or for diaphragms that are flexible, the distance between vertical elements of the seismic force-resisting system does not exceed 40 ft.

Where the alternate simplified design procedure of Section 12.14 is used, the Seismic Design Category is permitted to be determined from Table 11.6-1 alone, using the value of $S_{DS}$ determined in Section 12.14.8.1.

11.7 DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORY A

11.7.1 Applicability of Seismic Requirements for Seismic Design Category A Structures. Structures assigned to Seismic Design Category A need only comply with the requirements of

<table>
<thead>
<tr>
<th>OCCUPANCY CATEGORY</th>
<th>IMPORTANCE FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>I or II</td>
<td>1.0</td>
</tr>
<tr>
<td>III</td>
<td>1.25</td>
</tr>
<tr>
<td>IV</td>
<td>1.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>OCCUPANCY CATEGORY</th>
<th>IMPORTANCE FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>I or II</td>
<td>1.0</td>
</tr>
<tr>
<td>III</td>
<td>1.25</td>
</tr>
<tr>
<td>IV</td>
<td>1.5</td>
</tr>
</tbody>
</table>
Section 11.7. The effects on the structure and its components due to the forces prescribed in this section shall be taken as E and combined with the effects of other loads in accordance with the load combinations of Section 2.3 or 2.4. For structures with damping systems, see Section 18.2.1.

11.7.2 Lateral Forces. Each structure shall be analyzed for the effects of static lateral forces applied independently in each of two orthogonal directions. In each direction, the static lateral forces at all levels shall be applied simultaneously. For purposes of analysis, the force at each level shall be determined using Eq. 11.7-1 as follows:

\[ F_x = 0.01 w_x \]  

where \( F_x \) = the design lateral force applied at story \( x \), and \( w_x \) = the portion of the total dead load of the structure, \( D \), located or assigned to level \( x \).

11.7.3 Load Path Connections. All parts of the structure between separation joints shall be interconnected to form a continuous path to the lateral force-resisting system, and the connections shall be capable of transmitting the lateral forces induced by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure with elements having design strength of not less than 5 percent of the portion’s weight. This connection force does not apply to the overall design of the lateral force-resisting system. Connection design forces need not exceed the maximum forces that the structural system can deliver to the connection.

11.7.4 Connection to Supports. A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder, or truss either directly to its supporting elements, or to slabs designed to act as diaphragms. Where the connection is through a diaphragm, then the member’s supporting element must also be connected to the diaphragm. The connection shall have a minimum design strength of 5 percent of the dead plus live load reaction.

11.7.5 Anchorage of Concrete or Masonry Walls. Concrete and masonry walls shall be anchored to the roof and all floors and members that provide lateral support for the wall or that 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 the horizontal forces specified in Section 11.7.3, but not less than a minimum strength level horizontal force of 280 lb/linear ft (4.09 kN/m) of wall substituted for \( E \) in the load combinations of Section 2.3 or 2.4.

11.8 GEOLOGIC HAZARDS AND GEOTECHNICAL INVESTIGATION

11.8.1 Site Limitation for Seismic Design Categories E and F. A structure assigned to Seismic Design Category E or F shall not be located where there is a known potential for an active fault to cause rupture of the ground surface at the structure.

11.8.2 Geotechnical Investigation Report for Seismic Design Categories C through E. A geotechnical investigation report shall be provided for a structure assigned to Seismic Design Category C, D, E, or F in accordance with this section. An investigation shall be conducted and a report shall be submitted that shall include an evaluation of the following potential geologic and seismic hazards:

a. Slope instability.
b. Liquefaction.
c. Differential settlement.
d. Surface displacement due to faulting or lateral spreading.

The report shall contain recommendations for appropriate foundation designs or other measures to mitigate the effects of the previously mentioned hazards. Where deemed appropriate by the authority having jurisdiction, a site-specific geotechnical report is not required where prior evaluations of nearby sites with similar soil conditions provide sufficient direction relative to the proposed construction.

11.8.3 Additional Geotechnical Investigation Requirements for Seismic Design Categories D through F. The geotechnical investigation report for a structure assigned to Seismic Design Category D, E, or F shall include:

1. The determination of lateral pressures on basement and retaining walls due to earthquake motions.
2. The potential for liquefaction and soil strength loss evaluated for site peak ground accelerations, magnitudes, and source characteristics consistent with the design earthquake ground motions. Peak ground acceleration is permitted to be determined based on a site-specific study taking into account soil amplification effects or, in the absence of such a study, peak ground accelerations shall be assumed equal to \( S_\alpha/2.5 \).
3. Assessment of potential consequences of liquefaction and soil strength loss, including estimation of differential settlement, lateral movement, lateral loads on foundations, reduction in foundation soil-bearing capacity, increases in lateral pressures on retaining walls, and flotation of buried structures.
4. Discussion of mitigation measures such as, but not limited to, ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements and forces, or any combination of these measures and how they shall be considered in the design of the structure.
<table>
<thead>
<tr>
<th>Seismic Force-Resisting System</th>
<th>ASCE 7 Section where Detailing Requirements are Revised</th>
<th>Response Modification Coefficient, $\mu^*$</th>
<th>System Overstrength Factor, $\Omega_0^*$</th>
<th>Deflection Amplification Factor, $C_d^*$</th>
<th>Structural system / imitations and building height (ft) Limit$^*$</th>
</tr>
</thead>
<tbody>
<tr>
<td>22. Prestressed masonry shear walls</td>
<td>14.1</td>
<td>$1^{1/2}$</td>
<td>$2^{1/2}$</td>
<td>$3^{1/2}$</td>
<td>NL, NP, NP, NP, NP, NP</td>
</tr>
<tr>
<td>23. Light framed walls sheathed with wood structural panels rated for shear resistance or steel sheets</td>
<td>14.1, 14.1.4.2, and 14.5</td>
<td>7</td>
<td>$2^{1/2}$</td>
<td>$4^{1/2}$</td>
<td>NL, NL, 65, 65, 65</td>
</tr>
<tr>
<td>24. Light framed walls with shear panels of all other materials</td>
<td>14.1, 14.1.4.2, and 14.5</td>
<td>$7^{1/2}$</td>
<td>$2^{1/2}$</td>
<td>$2^{1/2}$</td>
<td>NL, NL, 35, NP, NP</td>
</tr>
<tr>
<td>25. Buckling-restrained braced frames, non-moment-resisting beam-column connections</td>
<td>14.1</td>
<td>7</td>
<td>2</td>
<td>$5^{1/2}$</td>
<td>NL, NL, 160, 160, 100</td>
</tr>
<tr>
<td>26. Buckling-restrained braced frames, moment-resisting beam-column connections</td>
<td>14.1</td>
<td>8</td>
<td>$2^{1/2}$</td>
<td>5</td>
<td>NL, NL, 160, 160, 100</td>
</tr>
<tr>
<td>27. Special steel plate shear wall</td>
<td>14.1</td>
<td>7</td>
<td>2</td>
<td>6</td>
<td>NL, NL, 160, 160, 100</td>
</tr>
<tr>
<td>C. MOMENT-RESISTING FRAME SYSTEMS</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Special steel moment frames</td>
<td>14.1 and 12.2.5.5</td>
<td>8</td>
<td>3</td>
<td>$5^{1/2}$</td>
<td>NL, NL, NL, NL, NL</td>
</tr>
<tr>
<td>2. Special steel truss moment frames</td>
<td>14.1</td>
<td>7</td>
<td>3</td>
<td>$5^{1/2}$</td>
<td>NL, NL, 160, 100, NP</td>
</tr>
<tr>
<td>3. Intermediate steel moment frames</td>
<td>12.2.5.6, 12.2.5.7, 12.2.5.8, 12.2.5.9, and 14.1</td>
<td>4.5</td>
<td>3</td>
<td>4</td>
<td>NL, NL, 35$^b$, NP$^a$, NP$^a$</td>
</tr>
<tr>
<td>4. Ordinary steel moment frames</td>
<td>12.2.5.6, 12.2.5.7, 12.2.5.8, and 14.1</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>NL, NL, NP$^a$, NP$^a$, NP$^a$</td>
</tr>
<tr>
<td>5. Special reinforced concrete moment frames</td>
<td>12.2.5.5 and 14.2</td>
<td>8</td>
<td>3</td>
<td>$5^{1/2}$</td>
<td>NL, NL, NL, NL, NL</td>
</tr>
<tr>
<td>6. Intermediate reinforced concrete moment frames</td>
<td>14.2</td>
<td>5</td>
<td>3</td>
<td>$4^{1/2}$</td>
<td>NL, NL, NP, NP, NP</td>
</tr>
<tr>
<td>7. Ordinary reinforced concrete moment frames</td>
<td>14.2</td>
<td>3</td>
<td>3</td>
<td>$2^{1/2}$</td>
<td>NL, NL, NP, NP, NP</td>
</tr>
<tr>
<td>8. Special composite steel and concrete moment frames</td>
<td>12.2.5.5 and 14.3</td>
<td>8</td>
<td>3</td>
<td>$5^{1/2}$</td>
<td>NL, NL, NL, NL, NL</td>
</tr>
<tr>
<td>9. Intermediate composite moment frames</td>
<td>14.3</td>
<td>5</td>
<td>3</td>
<td>$4^{1/2}$</td>
<td>NL, NL, NP, NP, NP</td>
</tr>
<tr>
<td>10. Composite partially restrained moment frames</td>
<td>14.3</td>
<td>6</td>
<td>3</td>
<td>$5^{1/2}$</td>
<td>160, 160, 100, NP, NP</td>
</tr>
<tr>
<td>11. Ordinary unrestrained moment frames</td>
<td>14.3</td>
<td>3</td>
<td>3</td>
<td>$2^{1/2}$</td>
<td>NL, NP, NP, NP, NP</td>
</tr>
<tr>
<td>D. DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 22% OF PRESCRIBED SEISMIC FORCES</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Steel eccentrically braced frames</td>
<td>14.1</td>
<td>8</td>
<td>$2^{1/2}$</td>
<td>4</td>
<td>NL, NL, NL, NL, NL</td>
</tr>
<tr>
<td>2. Special steel concentrically braced frames</td>
<td>14.1</td>
<td>7</td>
<td>$2^{1/2}$</td>
<td>$5^{1/2}$</td>
<td>NL, NL, NL, NL, NL</td>
</tr>
<tr>
<td>3. Special reinforced concrete shear walls</td>
<td>14.2</td>
<td>7</td>
<td>$2^{1/2}$</td>
<td>$5^{1/2}$</td>
<td>NL, NL, NL, NL, NL</td>
</tr>
<tr>
<td>4. Ordinary reinforced concrete shear walls</td>
<td>14.2</td>
<td>6</td>
<td>$2^{1/2}$</td>
<td>3</td>
<td>NL, NL, NP, NP, NP</td>
</tr>
<tr>
<td>5. Composite steel and concrete eccentrically braced frames</td>
<td>14.3</td>
<td>8</td>
<td>$2^{1/2}$</td>
<td>4</td>
<td>NL, NL, NL, NL, NL</td>
</tr>
<tr>
<td>6. Composite steel and concrete concentrically braced frames</td>
<td>14.3</td>
<td>6</td>
<td>$2^{1/2}$</td>
<td>5</td>
<td>NL, NL, NL, NL, NL</td>
</tr>
<tr>
<td>7. Composite steel plate shear walls</td>
<td>14.3</td>
<td>$7^{1/2}$</td>
<td>$2^{1/2}$</td>
<td>6</td>
<td>NL, NL, NL, NL, NL</td>
</tr>
<tr>
<td>8. Special composite reinforced concrete shear walls with steel elements</td>
<td>14.3</td>
<td>7</td>
<td>$2^{1/2}$</td>
<td>6</td>
<td>NL, NL, NL, NL, NL</td>
</tr>
<tr>
<td>9. Ordinary composite reinforced concrete shear walls with steel elements</td>
<td>14.3</td>
<td>6</td>
<td>$2^{1/2}$</td>
<td>5</td>
<td>NL, NL, NP, NP, NP</td>
</tr>
<tr>
<td>10. Special reinforced masonry shear walls</td>
<td>14.4</td>
<td>$5^{1/2}$</td>
<td>3</td>
<td>5</td>
<td>NL, NL, NL, NL, NL</td>
</tr>
<tr>
<td>11. Intermediate reinforced masonry shear walls</td>
<td>14.4</td>
<td>4</td>
<td>3</td>
<td>$3^{1/2}$</td>
<td>NL, NL, NP, NP, NP</td>
</tr>
<tr>
<td>12. Buckling-restrained braced frame</td>
<td>14.1</td>
<td>8</td>
<td>$2^{1/2}$</td>
<td>5</td>
<td>NL, NL, NL, NL, NL</td>
</tr>
<tr>
<td>13. Special steel plate shear walls</td>
<td>14.1</td>
<td>8</td>
<td>$2^{1/2}$</td>
<td>$6^{1/2}$</td>
<td>NL, NL, NL, NL, NL</td>
</tr>
</tbody>
</table>
to the moment frames exceeds 35 psf (1.68 kN/m²). In addition, the dead load of the exterior walls tributary to the moment frame shall not exceed 20 psf (0.96 kN/m²).

12.2.5.8 Single-Story Steel Ordinary and Intermediate Moment Frames in Structures Assigned to Seismic Design Category F. Single-story steel ordinary moment frames and intermediate moment frames in structures assigned to Seismic Design Category F are permitted up to a height of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m²). In addition, the dead loads of the exterior walls tributary to the moment frame shall not exceed 20 psf (0.96 kN/m²).

12.2.5.9 Other Steel Intermediate Moment Frame Limitations in Structures Assigned to Seismic Design Category F. In addition to the limitations for steel intermediate moment frames in structures assigned to Seismic Design Category E as set forth in Section 12.2.5.7, steel intermediate moment frames in structures assigned to Seismic Design Category F are permitted in light-frame construction.

12.2.5.10 Shear Wall-Frame Interactive Systems. The shear strength of the shear walls of the shear wall-frame interactive system shall be at least 75 percent of the design story shear at each story. The frames of the shear wall-frame interactive system shall be capable of resisting at least 25 percent of the design story shear in every story.

12.3 DIAPHRAGM FLEXIBILITY, CONFIGURATION IRREGULARITIES, AND REDUNDANCY

12.3.1 Diaphragm Flexibility. The structural analysis shall consider the relative stiffness of diaphragms and the vertical elements of the seismic force-resisting system. Unless a diaphragm can be idealized as either flexible or rigid in accordance with Sections 12.3.1.1, 12.3.1.2, or 12.3.1.3, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semi-rigid modeling assumption).

12.3.1.1 Flexible Diaphragm Condition. Diaphragms constructed of untopped steel decking or wood structural panels are permitted to be idealized as flexible in structures in which the vertical elements are steel or composite steel and concrete braced frames, or concrete, masonry, steel, or composite shear walls. Diaphragms of wood structural panels or untopped steel decks in one- and two-family residential buildings of light-frame construction shall also be permitted to be idealized as flexible.

12.3.1.2 Rigid Diaphragm Condition. Diaphragms of concrete slabs or concrete filled metal deck with span-to-depth ratios of 3 or less in structures that have no horizontal irregularities are permitted to be idealized as rigid.

12.3.1.3 Calculated Flexible Diaphragm Condition. Diaphragms not satisfying the conditions of Sections 12.3.1.1 or 12.3.1.2 are permitted to be idealized as flexible where the computed maximum in-plane deflection of the diaphragm under lateral load is more than two times the average story drift of adjoining vertical elements of the seismic force-resisting system of the associated story under equivalent tributary lateral load as shown in Fig. 12.3-1. The loadings used for this calculation shall be those prescribed by Section 12.8.

12.3.2 Irregular and Regular Classification. Structures shall be classified as regular or irregular based upon the criteria in this section. Such classification shall be based on horizontal and vertical configurations.

12.3.2.1 Horizontal Irregularity. Structures having one or more of the irregularity types listed in Table 12.3-1 shall be designated as having horizontal structural irregularity. Such structures assigned to the seismic design categories listed in Table 12.3-1 shall comply with the requirements in the sections referenced in that table.

12.3.2.2 Vertical Irregularity. Structures having one or more of the irregularity types listed in Table 12.3-2 shall be designated as having vertical irregularity. Such structures assigned to the seismic design categories listed in Table 12.3-2 shall comply with the requirements in the sections referenced in that table.

EXCEPTIONS:
1. Vertical structural irregularities of Types 1a, 1b, or 2 in Table 12.3-2 do not apply where no story drift ratio under design lateral seismic force is greater than 1/100 percent of the story drift ratio of the next story above. Torsional effects need not be considered in the calculation of story drifts. 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 12.3-2 are not required to be considered for one-story buildings in any seismic design category or for two-story buildings assigned to Seismic Design Categories B, C, or D.

12.3.3 Limitations and Additional Requirements for Systems with Structural Irregularities.

12.3.3.1 Prohibited Horizontal and Vertical Irregularities for Seismic Design Categories D through F. Structures assigned to Seismic Design Category E or F having horizontal irregularity Type 1b or vertical irregularities Type 1b, 5a, or 5b of Table 12.3-2 shall not be permitted. Structures assigned to Seismic Design Category D having vertical irregularity Type 5b of Table 12.3-2 shall not be permitted.
### TABLE 12.3-1 HORIZONTAL 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 is defined to exist where 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. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.</td>
<td>12.3.3.4, 12.8.4.3, 12.7.3, 12.12.1 Table 12.6-1 Section 16.2.2</td>
<td>D, E, and F, C, D, E, and F, B, C, D, E, and F, C, D, E, and F</td>
</tr>
<tr>
<td>1b. Extreme Torsional Irregularity is defined to exist where 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. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.</td>
<td>12.5.3.1, 12.5.3.4, 12.7.3, 12.8.4.3, 12.12.1 Table 12.6-1 Section 16.2.2</td>
<td>E, F, D, B, C, D, and D, C and D, D</td>
</tr>
<tr>
<td>2. Reentrant Corner Irregularity is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.</td>
<td>12.7.3.1, Table 12.6-1</td>
<td>D, E, and F, D, E, and F</td>
</tr>
<tr>
<td>3. Diaphragm Discontinuity Irregularity is defined to exist where there are diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.</td>
<td>12.3.3.4, 12.3.3.3, 12.7.3, 12.8.6-1, 16.2.2 Table 12.6-1</td>
<td>D, E, and F, B, C, D, E, F, and F, B, C, E, F, and F, B, C, E, F, and F, B, C, D, E, and F</td>
</tr>
<tr>
<td>4. Out-of-Plane Offset Irregularity is defined to exist where there are discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.</td>
<td>12.3.3.4, 12.3.3.3, 12.7.3, 12.8.6-1, 16.2.2 Table 12.6-1</td>
<td>D, E, and F, B, C, D, E, F, and F, B, C, E, F, and F, B, C, E, F, and F, B, C, D, E, and F</td>
</tr>
<tr>
<td>5. Nonparallel Systems- Irregularity is defined to exist where the vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force-resisting system.</td>
<td>12.3.3.4, 12.3.3.3, 12.7.3, 12.8.6-1, 16.2.2 Table 12.6-1</td>
<td>D, E, and F, B, C, D, E, F, and F, B, C, E, F, and F, B, C, E, F, and F, B, C, D, E, and F</td>
</tr>
</tbody>
</table>

### 12.3.3.2 Extreme Weak Stories
Structures with a vertical irregularity Type 5b as defined in Table 12.3-2, shall not be over two stories or 30 ft (9 m) in height.

**EXCEPTION:** The limit does not apply where the "weak" story is capable of resisting a total seismic force equal to \( \Omega_0 \) times the design force prescribed in Section 12.8.

### 12.3.3.3 Elements Supporting Discontinuous Walls or Frames
Columns, beams, trusses, or slabs supporting discontinuous walls or frames of structures having horizontal irregularity Type 4 of Table 12.3-1 or vertical irregularity Type 4 of Table 12.3-2 shall have the design strength to resist the maximum axial force that can develop in accordance with the load combinations with overstrength factor of Section 12.4.3.2. The connections of such discontinuous elements to the supporting members shall be adequate to transmit the forces for which the discontinuous elements were required to be designed.

### 12.3.4 Increase in Forces Due to Irregularities for Seismic Design Categories D through F
For structures assigned to Seismic Design Category D, E, or F and having a horizontal structural irregularity of Type 1a, 1b, 7, 3, or 4 in Table 12.3-1 or a vertical structural irregularity of Type 4 in Table 12.3-2, the design forces determined from Section 12.8.1 shall be increased 25% for connections of diaphragms to vertical elements and to collectors and for connections of collectors to the vertical elements. Collectors and their connections shall be designed for these increased forces unless they are designed for the load combinations with overstrength factor of Section 12.4.3.2, in accordance with Section 12.10.2.1.

### 12.3.4 Redundancy
A redundancy factor, \( \rho \), shall be assigned to the seismic force-resisting system in each of two orthogonal directions for all structures in accordance with this section.

### TABLE 12.3-2 VERTICAL 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. Stiffness-Soft Story Irregularity is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.</td>
<td>Table 12.6-1</td>
<td>D, E, and F</td>
</tr>
<tr>
<td>1b. Stiffness-Extreme Soft Story Irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.</td>
<td>Table 12.6-1</td>
<td>F, and F, D, E, and F</td>
</tr>
<tr>
<td>2. Weight (Mass) Irregularity is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.</td>
<td>Table 12.6-1</td>
<td>D, E, and F</td>
</tr>
<tr>
<td>3. Vertical Geometric Irregularity is defined to exist where the horizontal dimension of the seismic force-resisting system in any story is more than 130% of that in an adjacent story.</td>
<td>Table 12.6-1</td>
<td>D, E, and F</td>
</tr>
<tr>
<td>4. In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity is defined to exist where an in-plane offset of the lateral force-resisting elements is greater than the length of those elements or there exists a reduction in stiffness of the resisting elements in the story below.</td>
<td>Table 12.6-1</td>
<td>B, C, D, E, F, and F, D, E, and F, B, C, E, F, and F, B, C, E, F, and F, B, C, D, E, and F</td>
</tr>
<tr>
<td>5a. Discontinuity in Lateral Strength-Weak Story Irregularity is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.</td>
<td>Table 12.6-1</td>
<td>E and F, D, E, and F</td>
</tr>
<tr>
<td>5b. Discontinuity in Lateral Strength-Extreme Weak Story Irregularity is defined to exist where the story lateral strength is less than 60% of that in the story above. The story lateral strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.</td>
<td>Table 12.6-1</td>
<td>D, E, and F, B and C, D, E, and F</td>
</tr>
</tbody>
</table>

Minimum Design Loads for Buildings and Other Structures 125
12.3.4.1 Conditions Where Value of ρ is 1.0. The value of ρ is permitted to equal 1.0 for the following:

1. Structures assigned to Seismic Design Category B or C.
2. Drift calculation and P-delta effects.
3. Design of nonstructural components.
4. Design of nonbuilding structures that are not similar to buildings.
5. Design of collector elements, splices, and their connections for which the load combinations with overstrength factor of Section 12.4.3.2 are used.
6. Design of members or connections where the load combinations with overstrength of Section 12.4.3.2 are required for design.
7. Diaphragm loads determined using Eq. 12.10-1.
8. Structures with damping systems designed in accordance with Section 18.

12.3.4.2 Redundancy Factor, ρ, for Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E, or F, ρ shall equal 1.3 unless one of the following two conditions is met, whereby ρ is permitted to be taken as 1.0:

a. Each story resisting more than 35 percent of the base shear in the direction of interest shall comply with Table 12.3-3.

b. Structures that are regular in plan at all levels provided that the seismic force-resisting systems consist of at least two bays of seismic force-resisting perimeter framing on each side of the structure in each orthogonal direction at each story resisting more than 35 percent of the base shear. The number of bays for a shear wall shall be calculated as the length of shear wall divided by the story height or, in two cases, the length of shear wall divided by the story height for light-framed construction.

### TABLE 12.3-3 REQUIREMENTS FOR EACH STORY RESISTING MORE THAN 35% OF THE BASE SHEAR

<table>
<thead>
<tr>
<th>Lateral Force-Resisting Element</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Braced Frames</td>
<td>Removal of an individual brace, or connection thereto, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).</td>
</tr>
<tr>
<td>Moment Frames</td>
<td>Loss of moment resistance at the beam-to-column connections at both ends of a single beam would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).</td>
</tr>
<tr>
<td>Shear Walls or Wall Pier with a height-to-length ratio of greater than 1.0</td>
<td>Removal of a shear wall or wall pier with a height-to-length ratio greater than 1.0 within any story, or collector connections thereto, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).</td>
</tr>
<tr>
<td>Cantilever Columns</td>
<td>Loss of moment resistance at the base connections of any single cantilever column would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).</td>
</tr>
<tr>
<td>Other</td>
<td>No requirements</td>
</tr>
</tbody>
</table>

12.4 SEISMIC LOAD EFFECTS AND COMBINATIONS

12.4.1 Applicability. All members of the structure, including those not part of the seismic force resisting system, shall be designed using the seismic load effects of Section 12.4 unless otherwise exempted by this standard. Seismic load effects are the axial, shear, and flexural members forces resulting from application of horizontal and vertical seismic forces as set forth in Section 12.4.2. Where specifically required, seismic load effects shall be modified to account for system overstrength, as set forth in Section 12.4.3.

12.4.2 Seismic Load Effect. The seismic load effect, E, shall be determined in accordance with the following:

1. For use in load combination 5 in Section 2.3.2 or load combination 3 and 6 in Section 2.4.1, E shall be determined in accordance with Eq. 12.4-1 as follows:

   \[ E = E_h + E_v \]  \hspace{1cm} (12.4-1)

2. For use in load combination 7 in Section 2.3.2 or load combination 8 in Section 2.4.1, E shall be determined in accordance with Eq. 12.4-2 as follows:

   \[ E = E_h - E_v \]  \hspace{1cm} (12.4-2)

   where

   - \( E \) = seismic load effect
   - \( E_h \) = effect of horizontal seismic forces as defined in Section 12.4.2.1
   - \( E_v \) = effect of vertical seismic forces as defined in Section 17.4.4.

12.4.2.1 Horizontal Seismic Load Effect. The horizontal seismic load effect, \( E_h \), shall be determined in accordance with Eq. 12.4-3 as follows:

   \[ E_h = \rho Q_E \]  \hspace{1cm} (12.4-3)

   where

   - \( Q_E \) = effects of horizontal seismic forces from V or \( F_p \), where required in Sections 12.3.3 and 12.5.4, such effects shall result from application of horizontal forces simultaneously in two directions at right angles to each other;
   - \( \rho \) = redundancy factor, as defined in Section 12.3.4

12.4.2.2 Vertical Seismic Load Effect. The vertical seismic load effect, \( E_v \), shall be determined in accordance with Eq. 12.4-4 as follows:

   \[ E_v = 0.2 S_{DS} D \]  \hspace{1cm} (12.4-4)

   where

   - \( S_{DS} \) = design spectral response acceleration parameter at short periods obtained from Section 11.4.4
   - \( D \) = effect of dead load

EXCEPTIONS: The vertical seismic load effect, \( E_v \), is permitted to be taken as zero for either of the following conditions:

1. In Eqs. 12.4-1, 12.4-2, 12.4-3, and 12.4-4 where \( S_{DS} \) is equal to or less than 0.125.
2. In Eq. 12.4-2 where determining demands on the soil-structure interface of foundations.

12.4.2.3 Seismic Load Combinations. Where the prescribed seismic load effect, E, defined in Section 12.4.2 is combined with the effects of other loads as set forth in Chapter 2, the following seismic load combinations for structures not subject to flood or atmospheric ice loads shall be used in lieu of the seismic load combinations in either Section 2.3.2 or 2.4.1:
Basic Combinations for Strength Design (see Sections 2.3.2 and 2.2 for notation).

5. \(1.2 + 0.2S_{DS})D + \rho Q_E \mid L + 0.2S\)
6. \((0.9 - 0.2S_{DS})D + \rho Q_E \mid 1.6H\)

NOTES:
1. The load factor on \(L\) in combination 5 is permitted to equal 0.5 for all occupancies in which \(T_r\) in Table 4-1 is less than or equal to 100 psf (47.9 kN/m²), with the exception of garages or areas occupied as places of public assembly.
2. The load factor on \(H\) shall be set equal to zero in combination 7 if the structural action due to \(H\) counteracts that due to \(F\). Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in \(H\) but shall be included in the design resistance.

Basic Combinations for Allowable Stress Design (see Sections 2.4.1 and 2.2 for notation).

5. \((1.0 + 0.14S_{DS})D + H + F = 0.1\rho Q_E\)
6. \((1.0 + 0.105S_{DS})D + H + F = 0.525\rho Q_E + 0.75L + 0.75(L_c + S\ or\ R)\)
8. \((0.6 - 0.14S_{DS})D + 0.7\rho Q_E + H\)

12.4.3 Seismic Load Effect Including Overstrength Factor. Where specifically required, conditions requiring overstrength factor applications shall be determined in accordance with the following:

1. For use in load combination 5 in Section 2.3.2 or load combinations 5 and 6 in Section 2.4.1, \(E_m\) shall be taken equal to \(E_m\) as determined in accordance with Eq. 12.4-5 as follows:

\[E_m = E_{mh} + E_v\]  \hspace{1cm} (12.4-5)

2. For use in load combination 7 in Section 2.3.2 or load combination 8 in Section 2.4.1, \(E_m\) shall be taken equal to \(E_{mh}\) as determined in accordance with Eq. 12.4-6 as follows:

\[E_m = E_{mh} - F_E\]  \hspace{1cm} (12.4-6)

where

\(E_m\) = seismic load effect including overstrength factor
\(E_{mh}\) = effect of horizontal seismic forces including structural overstrength as defined in Section 12.4.3.1
\(E_v\) = vertical seismic load effect as defined in Section 12.4.2.2

12.4.3.1 Horizontal Seismic Load Effect with Overstrength Factor. The horizontal seismic load effect with overstrength factor, \(E_{mh}\), shall be determined in accordance with Eq. 12.4-7 as follows:

\[E_{mh} = \Omega_o Q_E\]  \hspace{1cm} (12.4-7)

where

\(Q_E\) = effects of horizontal seismic forces from \(V\) or \(F_r\) as specified in Sections 12.8.1 and 13.3.1, respectively. Where required in Sections 12.5.3 and 12.5.4, such effects shall result from application of horizontal forces simultaneously in two directions at right angles to each other.
\(\Omega_o\) = overstrength factor

EXCEPTION: The value of \(E_{mh}\) 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.

12.4.3.2 Load Combinations with Overstrength Factor. Where the seismic load effect with overstrength, \(L_m\), defined in Section 12.4.3 is combined with the effects of other loads as set forth in Chapter 2, the following seismic load combination for structures not subject to flood or atmospheric ice loads shall be used in lieu of the seismic load combinations in either Section 2.3.2 or 2.2.1.

Basic Combinations for Strength Design with Overstrength Factor (see Sections 2.3.2 and 2.2 for notation).

5. \((1.2 + 0.2S_{DS})D + \Omega_o Q_E + L + 0.2S\)
6. \((0.9 - 0.2S_{DS})D + \Omega_o Q_E + 1.6H\)

NOTES:
1. The load factor on \(L\) in combination 5 is permitted to equal 0.5 for all occupancies in which \(L_o\) in Table 4-1 is less than or equal to 100 psf (47.9 kN/m²), with the exception of garages or areas occupied as places of public assembly.
2. The load factor on \(H\) shall be set equal to zero in combination 7 if the structural action due to \(H\) counteracts that due to \(E\). Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in \(H\) but shall be included in the design resistance.

Basic Combinations for Allowable Stress Design with Overstrength Factor (see Sections 2.4.1 and 2.2 for notation).

5. \((1.0 + 0.14S_{DS})D + H + F + 0.75\rho Q_E\)
6. \((1.0 + 0.105S_{DS})D + H + F + 0.525\rho Q_E + 0.75L + 0.75(L_c + S\ or\ R)\)
8. \((0.6 - 0.14S_{DS})D + 0.7\rho Q_E + H\)

12.4.3.3 Allowable Stress Increase for Load Combinations with Overstrength. Where allowable stress design methodologies are used with the seismic load effect defined in Section 12.4.3 applied in load combinations 5, 6, or 8 of Section 2.4.1, allowable stresses are permitted to be determined using an allowable stress increase of 1.2. This increase shall not be combined with increases in allowable stresses or load combination reductions otherwise permitted by this standard in the material reference document except that combination with the duration of load increases permitted in AFPA NDS is permitted.

12.4.4 Minimum Upward Force for Horizontal Cantilevers for Seismic Design Categories D through F. In structures assigned to Seismic Design Category D, E, or F, 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 Section 12.4.

12.5 DIRECTION OF LOADING

12.5.1 Direction of Loading Criteria. The directions of application of seismic forces used in the design shall be those which will produce the most critical load effects. It is permitted to satisfy this requirement using the procedures of Section 12.5.2 for Seismic Design Category D, Section 12.5.3 for Seismic Design Category C, and Section 12.5.4 for Seismic Design Categories D, E, and F. The seismic design forces are permitted to be applied independently in each of two orthogonal directions and orthogonal interaction effects are permitted to be neglected.

12.5.2 Seismic Design Category B. For structures assigned to Seismic Design Category B, the design seismic forces are permitted to be applied independently in each of two orthogonal directions and orthogonal interaction effects are permitted to be neglected.

12.5.3 Seismic Design Category C. Loading applied to structures assigned to Seismic Design Category C shall, as a minimum,
requirements of Section 12.5.2 for Seismic Design Category B and the requirements of this section. Structures that have horizontal irregularity Type 5 in Table 12.3-1 shall use one of the following procedures:

a. **Orthogonal Combination Procedure.** The structure shall be analyzed using the equivalent lateral force analysis procedure of Section 12.8, the modal response spectrum analysis procedure of Section 12.9, or the linear response history procedure of Section 16.1, as permitted under Section 12.6, with the loading applied independently in any two orthogonal directions and the most critical load effect due to direction of application of seismic forces on the structure is permitted to be assumed to be satisfied if components and their foundations are designed for the following combination of prescribed loads: 100 percent of the forces in one direction plus 50 percent of the forces for the perpendicular direction, the combination requiring the maximum component strength shall be used.

b. **Simultaneous Application of Orthogonal Ground Motion.** The structure shall be analyzed using the linear response history procedure of Section 16.1 or the nonlinear response history procedure of Section 16.2, as permitted by Section 12.6, with orthogonal pairs of ground motion acceleration histories applied simultaneously.

12.5.4 Seismic Design Categories D through F. Structures assigned to Seismic Design Category D, E, or F shall, as a minimum, conform to the requirements of Section 12.5.3. In addition, any column or wall that forms part of two or more intersecting seismic-force-resisting systems and is subjected to axial loads due to seismic forces acting along principal plan axis exceeding or exceeding 20 percent of the axial design strength of the column or wall shall be designed for the most critical load effect due to application of seismic forces in any direction. Either of the procedures of Section 12.5.3 a or b are permitted to be used to satisfy this requirement. Except as required by Section 12.7.3, 2-D analyses are permitted for structures with flexible diaphragms.

### TABLE 12.6-1 PERMITTED ANALYTICAL PROCEDURES

<table>
<thead>
<tr>
<th>Seismic Design Category</th>
<th>Structural Characteristics</th>
<th>Equivalent Lateral Force Analysis Section 12.8</th>
<th>Modal Response Spectrum Analysis Section 12.9</th>
<th>Linear Response History Analysis Section 12.9</th>
<th>Seismic Return Period Chapter 16</th>
</tr>
</thead>
<tbody>
<tr>
<td>B, C</td>
<td>Occupancy Category I or II buildings of light-framed construction not exceeding 3 stories in height</td>
<td>P</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td></td>
<td>Other Occupancy Category I or II buildings not exceeding 2 stories in height</td>
<td>P</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td></td>
<td>All other structures</td>
<td>P</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>D, E, F</td>
<td>Occupancy Category I or II buildings of light-framed construction not exceeding 3 stories in height</td>
<td>P</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td></td>
<td>Other Occupancy Category I or II buildings not exceeding 2 stories in height</td>
<td>P</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td></td>
<td>Regular structures with $T &lt; 3.5T_i$ and all structures of light frame construction</td>
<td>P</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td></td>
<td>Irregular structures with $T &lt; 3.5T_i$ and having only horizontal irregularities Type 2, 3, 4, 5 of Table 12.2-1 or vertical irregularities Type 4, 5a, or 5b of Table 12.3-1</td>
<td>P</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td></td>
<td>All other structures</td>
<td>NP</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
</tbody>
</table>

**NOTE:** P: Permitted; NP: Not Permitted

minimum weight of 10 psf (0.48 kN/m²) of floor area, whichever is greater.

3. **Total operating weight of permanent equipment.**

4. Where the flat roof snow load, $P_y$, exceeds 30 psf (1.44 kN/m²), 20 percent of the uniform design snow load, regardless of actual roof slope.

### 12.6 ANALYSIS PROCEDURE SELECTION

The structural analysis required by Chapter 17 shall consist of one of the types permitted in Table 12.6-1, 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 is permitted to be used. The analysis procedure selected shall be completed in accordance with the requirements of the corresponding section referenced in Table 12.6-1.

### 12.7 MODELING CRITERIA

#### 12.7.1 Foundation Modeling.
For purposes of determining seismic loads, it is permitted to consider the structure to be fixed at the base. Alternatively, where foundation flexibility is considered, it shall be in accordance with Section 12.13.3 or Chapter 19.

#### 12.7.2 Effective Seismic Weight.
The effective seismic weight, $W_e$, of a structure shall include the total dead load and other loads listed below:

1. In areas used for storage, a minimum of 25 percent of the floor live load (floor live load in public garages and open parking structures need not be included).

2. Where provision for partitions is required by Section 4.2.2 in the floor load design, the actual partition weight or a

minimum weight of 10 psf (0.48 kN/m²) of floor area, whichever is greater.

3. Total operating weight of permanent equipment.

4. Where the flat roof snow load, $P_y$, exceeds 30 psf (1.44 kN/m²), 20 percent of the uniform design snow load, regardless of actual roof slope.

### 12.7.3 Structural Modeling.
A mathematical model of the structure shall be constructed for the purpose of determining member forces and structure displacements resulting from applied loads and any imposed displacements or F-Delta effects. The model shall include the stiffness and strength of elements that are significant to the distribution of forces and deformations in the structure and represent the spatial distribution of mass and stiffness throughout the structure.

Structures that have horizontal structural irregularity Type 1a, 1b, 4, or 5 of Table 12.3-1 shall be analyzed using a 3-D representation. Where a 3-D model is used, 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 have not been classified as rigid or flexible in accordance with Section 12.3.1, the model shall include representation of the diaphragm's stiffness characteristics 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:

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

b. For steel moment frame systems, the contribution of panel zone deformations to overall story drift shall be included.
12.7.4 Interaction Effects. Moment-resisting frames that are enclosed or adjoined by elements that are more rigid and 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 provide for the effect of those rigid elements on the structural system at structural deformations corresponding to the design story drift (Δ) as determined in Section 12.8.6. In addition, the effects of these elements shall be considered where determining whether a structure has one or more of the irregularities defined in Section 12.3.2.

12.8 EQUIVALENT LATERAL FORCE PROCEDURE

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

\[ V = C_s W \]  
(12.8-1)

where

\[ C_s = \text{the seismic response coefficient determined in accordance with Section 12.8.1.1} \]

\[ W = \text{the effective weight per Section 12.7.2} \]

12.8.1.1 Calculation of Seismic Response Coefficient. The seismic response coefficient, \( C_s \), shall be determined in accordance with Eq. 12.8.2.

\[ C_s = \frac{S_{DS}}{T} \]  
(12.8-2)

where

\[ S_{DS} = \text{the design spectral response acceleration parameter in the short period range as determined from Section 11.4.4} \]

\[ R = \text{the response modification factor in Table 12.2-1} \]

\[ I = \text{the occupancy importance factor determined in accordance with Section 11.5.1} \]

The value of \( C_s \) computed in accordance with Eq. 12.8-2 need not exceed the following:

\[ C_s = \frac{S_{DI}}{T^2} \left( \frac{R}{I} \right) \]  
(12.8-3)

\[ C_s = \frac{S_{DI}T_L}{T^2} \frac{R}{I} \text{ for } T > T_L \]  
(12.8-4)

\( C_s \) shall not be less than

\[ C_s = 0.01 \]  
(12.8-5)

In addition, for structures located where \( S_1 \) is equal to or greater than 0.6g, \( C_s \) shall not be less than

\[ C_s = \frac{0.5S_1}{I} \]  
(12.8-6)

<table>
<thead>
<tr>
<th>Design Spectral Response Acceleration Parameter at 1 s, S_{DS}</th>
<th>Coefficient C_s</th>
</tr>
</thead>
<tbody>
<tr>
<td>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>0.12</td>
<td>1.6</td>
</tr>
<tr>
<td>≤ 0.1</td>
<td>1.7</td>
</tr>
</tbody>
</table>

where \( I \) and \( R \) are as defined in Section 12.8.1.1 and \( S_{DS} \) is the design spectral response acceleration parameter at a period of 1.0 s, as determined from Section 11.4.4

\( T = \) the fundamental period of the structure (s) determined in Section 12.8.2

\( T_L = \) long-period transition period (s) determined in Section 11.4.5

\( S_1 = \) the mapped maximum considered earthquake spectral response acceleration parameter determined in accordance with Section 11.4.1

12.8.1.2 Soil Structure Interaction Reduction. A soil structure interaction reduction is permitted where determined using Chapter 19 or other generally accepted procedures approved by the authority having jurisdiction.

12.8.2 Period Determination. The fundamental period of the structure, \( T \), 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, \( T \), shall not exceed the product of the coefficient for upper limit on calculated period (\( C_p \)) from Table 12.8-1 and the approximate fundamental period, \( T_a \), determined from Eq. 12.8-7. As an alternative to performing an analysis to determine the fundamental period, \( T \), it is permitted to use the approximate building period, \( T_a \), calculated in accordance with Section 12.8.2.1, directly.

12.8.2.1 Approximate Fundamental Period. The approximate fundamental period (\( T_a \)), in s, shall be determined from the following equation:

\[ T_a = C_i h^x \]  
(12.8-7)

where \( h_a \) is the height in ft above the base to the highest level of the structure and the coefficients \( C_i \) and \( x \) are determined from Table 12.8-2.

<table>
<thead>
<tr>
<th>Structure Type</th>
<th>( C_i )</th>
<th>( x )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting when subjected to seismic forces.</td>
<td>0.0128 (0.0009754)</td>
<td>0.8</td>
</tr>
<tr>
<td>Concrete moment-resisting frames</td>
<td>0.016 (0.00466)</td>
<td>0.9</td>
</tr>
<tr>
<td>In eccentrically braced steel frames</td>
<td>0.03 (0.00731)</td>
<td>0.75</td>
</tr>
<tr>
<td>All other structural systems</td>
<td>0.02 (0.00488)</td>
<td>0.75</td>
</tr>
</tbody>
</table>

* Metric equivalents are shown in parentheses.
Alternatively, it is permitted to determine the approximate fundamental period \( T_a \), in s, from the following equation for structures not exceeding 12 stories in height in which the seismic force-resisting system consists entirely of concrete or steel moment resisting frames and the story height is at least 10 ft (3 m):

\[
T_a = 0.1 N \tag{12.8-8}
\]

where \( N \) = number of stories.

The approximate fundamental period, \( T_a \), in s for masonry or concrete shear wall structures is permitted to be determined from Eq. 12.8-9 as follows:

\[
T_a = \frac{0.0019}{\sqrt{C_w}h_n} \tag{12.8-9}
\]

where \( h_n \) is as defined in the preceding text and \( C_w \) is calculated from Eq. 12.8-10 as follows:

\[
C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left( \frac{h_i}{h_i^*} \right)^2 \frac{A_i}{\left[ 1 + 0.83 \left( \frac{h_i}{D_i} \right)^2 \right]} \tag{12.8-10}
\]

where

\( A_B = \) area of base of structure, \( ft^2 \)
\( A_i = \) web area of shear wall “i” in \( ft^2 \)
\( D_i = \) length of shear wall “i” in ft
\( h_i = \) height of shear wall “i” in ft
\( x = \) number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

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

\[
F_x = C_{ux}V \tag{12.8-11}
\]

and

\[
C_{ux} = \frac{w_xh_x^k}{\sum_{i=1}^{n} w_ih_i^k} \tag{12.8-12}
\]

where

\( C_{ux} = \) vertical distribution factor,
\( V = \) total design lateral force or shear at the base of the structure (kip or kN)
\( w_i \) and \( w_x \) = the portion of the total effective seismic weight of the structure \( W \) located or assigned to Level \( i \) or \( x \)
\( h_i \) and \( h_x \) = the height (ft or m) from the base to Level \( i \) or \( x \)
\( k = \) an exponent related to the structure period as follows:

<table>
<thead>
<tr>
<th>Structure Period</th>
<th>( k )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5 s or less</td>
<td>1</td>
</tr>
<tr>
<td>2.5 s or more</td>
<td>2</td>
</tr>
</tbody>
</table>

12.8.4 Horizontal Distribution of Forces. The seismic design story shear in any story \( V_x \) (kip or kN) shall be determined from the following equation:

\[
V_x = \sum_{i=x}^{n} F_i \tag{12.8-13}
\]

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

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

12.8.4.1 Inherent Torsion. For diaghrags that are not flexible, the distribution of lateral forces at each level shall consider the effect of the inherent torsional moment, \( M_i \), resulting from eccentricity between the locations of the center of mass and the center of rigidity. For flexible diaghrags, the distribution of forces to the vertical elements shall account for the position and distribution of the masses supported.

12.8.4.2 Accidental Torsion. Where diaghrags are not flexible, the design shall include the inherent torsional moment \( (M_i) \) (kip or kN) resulting from the location of the structure masses plus the accidental torsional moments \( (M_{acc}) \) (kip or kN) caused by assumed displacement of the center of mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied forces.

Where earthquake forces are applied concurrently in two orthogonal directions, the required 5 percent displacement of the center of mass need not be applied in the same direction; it shall be applied in the direction that produces the greater effect.

12.8.4.3 Amplification of Accidental Torsional Moment. Structures assigned to Seismic Design Category C, D, E, or F, where Type 1a or 1b torsional irregularity exists as defined in Table 12.3-1 shall have the effects accounted for by multiplying \( M_{acc} \) at each level by a torsional amplification factor \( (A_x) \) as illustrated in Fig. 12.8-1 and determined from the following equation:

\[
A_x = \left( \frac{\delta_{max}}{1.2\delta_{avg}} \right)^2 \tag{12.8-14}
\]

where

\( \delta_{max} = \) the maximum displacement at Level \( x \) (in. or mm) computed assuming \( A_x = 1 \)
\( \delta_{avg} = \) the average of the displacements at the extreme points of the structure at Level \( x \) computed assuming \( A_x = 1 \) (in. or mm)

**EXCEPTION:** The accidental torsional moment need not be amplified for structures of light-frame construction.

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.

12.8.5 Overturning. The structure shall be designed to resist overturning effects caused by the seismic forces determined in Section 12.8.3.

12.8.6 Story Drift Determination. The design story drift \( (\Delta) \) shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration. See Fig. 12.8-2. Where allowable stress design is used, \( \Delta \) shall be computed using the strength level seismic forces specified in Section 12.8 without reduction for allowable stress design.
The deflections of Level $x$ at the center of the mass ($\delta_x$) (in. or mm) shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I}$$  \hspace{1cm} (12.8-15)

where:

- $C_d$ — the deflection amplification factor in Table 12.2-1
- $\delta_{xe}$ — the deflections determined by an elastic analysis
- $I$ — the importance factor determined in accordance with Section 11.5.1

12.8.6.1 Minimum Base Shear for Computing Drift. The elastic analysis of the seismic force resisting system shall be made using the prescribed seismic design forces of Section 12.8.

12.8.6.2 Period for Computing Drift. For determining compliance with the story drift limits of Section 12.12.1, it is permitted to determine the elastic drifts, ($\delta_{xe}$), using seismic design forces based on the computed fundamental period of the structure without the upper limit ($C_d T_e$) specified in Section 12.8.2.

12.8.7 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.
where the stability coefficient ($\theta$) as determined by the following equation is equal to or less than 0.10:

$$\theta = \frac{P_s \Delta}{V_s h_{sz} C_d}$$  \hspace{1cm} (12.8-16)

where

$P_s =$ the total vertical design load at and above Level $x$ (kip or kN); where computing $P_s$, no individual load factor need exceed 1.0

$\Delta =$ the design story drift as defined in Section 12.8.6 occurring simultaneously with $V_s$ (in. or mm)

$V_s =$ the seismic shear force acting between Levels $x$ and $x - 1$ (kip or kN)

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

$C_d =$ the deflection amplification factor in Table 12.2-1

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

$$\theta_{max} = 0.5 \frac{\beta}{C_d} \leq 0.25$$  \hspace{1cm} (12.8-17)

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.

Where the stability coefficient ($\theta$) is greater than 0.10 but less than or equal to $\theta_{max}$, the incremental factor related to P-delta effects on displacements and member forces shall be determined by rational analysis. Alternatively, it is permitted to multiply displacements and member forces by $1/(1 - \theta)$.

Where $\theta$ is greater than $\theta_{max}$, the structure is potentially unstable and shall be redesigned.

Where the P-delta effect is included in an automated analysis, Eq. 12.8-17 shall still be satisfied, however, the value of $\theta$ computed from Eq. 12.8-16 using the results of the P-delta analysis is permitted to be divided by $(1 + \theta)$ before checking Eq. 12.8-17.

12.9 MODAL RESPONSE SPECTRUM ANALYSIS

12.9.1 Number of Modes. An analysis shall be conducted to determine the natural modes of vibration for the structure. 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 the orthogonal horizontal directions of response considered by the model.

12.9.2 Modal Response Parameters. The value for each force-related design parameter of interest, including story drifts, support forces, and individual member forces for each mode of response shall be computed using the properties of each mode and the response spectra defined in either Section 12.4.5 or 21.2 divided by the quantity $R_f$. The value for displacement and drift quantities shall be multiplied by the quantity $R_d$.

12.9.3 Combined Response Parameters. The value for each parameter of interest calculated for the various modes shall be combined using either the square root of the sum of the squares method (SRSS) or the complete quadratic combination method (CQC), in accordance with ASCE 4. The CQC method shall be used for each of the modal values or where closely spaced modes that have significant cross-correlation of translational and torsional response.

12.9.4 Scaling Design Values of Combined Response. A base shear ($V$) shall be calculated in each of the two orthogonal horizontal directions using the calculated fundamental period of the structure $T$ in each direction and the procedures of Section 12.8, except where the calculated fundamental period exceeds ($C_d(T_a)$), then ($C_d(T_a)$) shall be used in lieu of $T$ in that direction. Where the combined response for the modal base shear ($V_i$) is less than 85 percent of the calculated base shear ($V$) using the equivalent lateral force procedure, the forces, but not the drifts, shall be multiplied by $0.85\sqrt{V_i}$:

$$V = \text{the equivalent lateral force procedure base shear, calculated in accordance with this section and Section 12.8}$$

$V_i =$ the base shear from the required modal combination

12.9.5 Horizontal Shear Distribution. The distribution of horizontal shear shall be in accordance with the requirements of Section 12.8.4 except that amplification of torsion per Section 12.8.4.3 is not required where accidental torsional effects are included in the dynamic analysis model.

12.9.6 P-Delta Effects. The P-delta effects shall be determined in accordance with Section 12.8.7. The base shear used to determine the story shears and the story drifts shall be determined in accordance with Section 12.8.6.

12.9.7 Soil Structure Interaction Reduction. A soil structure interaction reduction is permitted where determined using Chapter 19 or other generally accepted procedures approved by the authority having jurisdiction.

12.10 DIAPHRAGMS, CHORDS, AND COLLECTORS

12.10.1 Diaphragm Design. Diaphragms shall be designed for both the shear and bending stresses resulting from design forces. At diaphragm discontinuities, such as openings and reentrant corners, the design shall assure that the dissipation or transfer of edge (chord) forces combined with other forces in the diaphragm is within shear and tension capacity of the diaphragm.

12.10.1.1 Diaphragm Design Forces. Floor and roof diaphragms shall be designed to resist design seismic forces from the structural analysis, but shall not be less than that determined in accordance with Eq. 12.10-1 as follows:

$$F_{pz} = \sum_{i=x}^{n} \frac{F_i}{w_{pz}}$$  \hspace{1cm} (12.10-1)

where

$F_{pz} =$ the diaphragm design force

$F_i =$ the design force applied to Level $i$

$w_i =$ the weight tributary to Level $i$

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

The force determined from Eq. 12.10-1 need not exceed $0.4S_{DS} Iw_{pz}$, but shall not be less than $0.2S_{DS} Iw_{pz}$. Where the diaphragm is required to transfer design seismic force 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. 12.10-1. The redundancy factor, $\rho$, applies to the design of diaphragms in structures assigned to Seismic Design Category D, E, or F. For inertial forces calculated in accordance with Eq. 12.10-1, the redundancy factor shall equal 1.0. For transfer forces, the redundancy factor, $\rho$, shall be the same as that used for the structure. For structures having horizontal or vertical structural irregularities of the types indicated in Section 12.3.3.4, the requirements of that section shall also apply.
### TABLE 12.12-1 ALLOWABLE STORY DRIFT, $\Delta_a^{e,b}$

<table>
<thead>
<tr>
<th>Structure</th>
<th>Occupancy Category</th>
<th>I or II</th>
<th>III</th>
<th>IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.</td>
<td></td>
<td>0.025$h_{xs}$</td>
<td>0.020$h_{xs}$</td>
<td>0.015$h_{xs}$</td>
</tr>
<tr>
<td>Masonry cantilever shear wall structures $^d$</td>
<td></td>
<td>0.010$h_{xs}$</td>
<td>0.010$h_{xs}$</td>
<td>0.010$h_{xs}$</td>
</tr>
<tr>
<td>Other masonry shear wall structures</td>
<td></td>
<td>0.007$h_{xs}$</td>
<td>0.007$h_{xs}$</td>
<td>0.007$h_{xs}$</td>
</tr>
<tr>
<td>All other structures</td>
<td></td>
<td>0.020$h_{xs}$</td>
<td>0.015$h_{xs}$</td>
<td>0.010$h_{xs}$</td>
</tr>
</tbody>
</table>

$^a$h_{xs} is the story height below Level x.

$^b$For seismic force-resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.

$^c$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. The structure separation requirement of Section 12.12.3 is not waived.

$^d$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.

12.11.2.7 Walls with Pilasters. Where pilasters are present in the wall, the anchorage force at the pilasters shall be calculated considering the additional load transferred from the wall panels to the pilasters. However, the minimum anchorage force at a floor or roof shall not be reduced.

12.12 DRIFT AND DEFORMATION

12.12.1 Story Drift Limit. The design story drift ($\Delta$) as determined in Sections 12.8.6, 12.9.2, or 16.1, shall not exceed the allowable story drift ($\Delta_a$) as obtained from Table 12.12-1 for any story. For structures with significant torsional deflections, the maximum drift shall include torsional effects. For structures assigned to Seismic Design Category C, D, E, or F having horizontal irregularity types 1a or 1b of Table 12.3-1, the design story drift, $\Delta$, 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.

12.12.1.1 Moment Frames in Structures Assigned to Seismic Design Categories D through E. For seismic force-resisting systems comprised solely of moment frames in structures assigned to Seismic Design Categories D, E, or F, the design story drift ($\Delta$) shall not exceed $\Delta_{a}/\rho$ for any story, $\rho$ shall be determined in accordance with Section 12.3.4.2.

12.12.2 Diaphragm Deflection. 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 that deflection that will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

12.12.3 Building Separation. 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 ($\delta$) as determined in Section 12.8.6

12.12.4 Deformation Compatibility for Seismic Design Categories D through E. For structures assigned to Seismic Design Category D, E, or F, every structural component not included in the seismic force-resisting system in the direction under consideration shall be designed to be adequate for the gravity load effects and the seismic forces resulting from displacement to the design story drift ($\Delta$) as determined in accordance with Section 12.8.6 (see also Section 12.12.1).

EXCEPTION: Reinforced concrete frame members not designed as part of the seismic force-resisting system shall comply with Section 21.9 of ACI 318.

Where 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.

12.13 FOUNDATION DESIGN

12.13.1 Design Basis. The design basis for foundations shall be as set forth in Section 12.1.5.

12.13.2 Materials of Construction. Materials used for the design and construction of foundations shall comply with the requirements of Chapter 14. Design and detailing of steel piles shall comply with Section 14.1.8. Design and detailing of concrete piles shall comply with Section 14.2.3.

12.13.3 Foundation Load-Deflection Characteristics.

Where foundation flexibility is included for the linear analysis procedures in Chapters 12 and 16, the load-deflection characteristics of the foundation-soil system (foundation stiffness) shall be modeled in accordance with the requirements of this section. The linear load-deflection behavior of foundations shall be represented by an equivalent linear stiffness using soil properties that are compatible with the soil strain levels associated with the design earthquake motion. The strain-compatible shear modulus, $G$, and the associated strain compatible shear wave velocity, $V_s$, needed for the evaluation of equivalent linear stiffness shall be determined using the criteria in Section 19.2.1.1 or based on a site-specific study. A 50 percent increase and decrease in stiffness shall be incorporated in dynamic analyses unless smaller variations can be justified based on field measurements of dynamic soil properties or direct measurements of dynamic foundation stiffness. The largest values of response shall be used in design.

12.13.4 Reduction of Foundation Overturning. Overturning effects at the soil-foundation interface are permitted to be reduced by 25 percent for foundations of structures that satisfy both of the following conditions:

a. The structure is designed in accordance with the Equivalent Lateral Force Analysis as set forth in Section 12.8.

b. The structure is not an inverted pendulum or cantilevered column type structure.

Overturning effects at the soil-foundation interface are permitted to be reduced by 10 percent for foundations of structures designed in accordance with the modal analysis requirements of Section 12.9.
Chapter 20
SITE CLASSIFICATION PROCEDURE FOR SEISMIC DESIGN

20.1 SITE CLASSIFICATION

The site soil shall be classified in accordance with Table 20.3-1 and Section 20.3 based on the upper 100 ft (30 m) of the site profile. Where site specific data are not available to a depth of 100 ft, appropriate soil properties are permitted to be estimated by the registered design professional preparing the soil investigation report based on known geologic conditions. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the authority having jurisdiction or geotechnical data determines Site Class E or F soils are present at the site. Site Classes A and B shall not be assigned to a site if there is more than 10 ft of soil between the rock surface and the bottom of the spread footing or mat foundation.

20.2 SITE RESPONSE ANALYSIS FOR SITE CLASS F SOIL

A site-response analysis in accordance with Section 21.1 shall be provided for Site Class F soils, unless the exception to Section 20.3.1 is applicable.

20.3 SITE CLASS DEFINITIONS

Site class types shall be assigned in accordance with the definitions provided in Table 20.3-1 and this section.

20.3.1 Site Class F. Where any of the following conditions is satisfied, the site shall be classified as Site Class F and a site response analysis in accordance with Section 21.1 shall be performed.

1. Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils.

EXCEPTION: For those structures having fundamental periods of vibration equal to or less than 0.5 s, site response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class is permitted to be determined in accordance with Section 20.3 and the corresponding values of $F_0$ and $F_0$ determined from Tables 11.4.1 and 11.4.2.

2. Peats and/or highly organic clays ($H > 10$ ft (3 m)) of peat and/or highly organic clay where $H =$ thickness of soil.

3. Very high plasticity clays ($H > 25$ ft (7.6 m)) with $P_I > 75$.

4. Very thick soft/medium stiff clays ($H > 120$ ft (37 m)) with $\gamma_u < 1000$ psf (50 kPa).

20.3.2 Soft Clay Site Class E. Where a site does not qualify under the criteria for Site Class F, and there is a total thickness of soft clay greater than 10 ft (3 m) where a soft clay layer is defined by $\gamma_u < 500$ psf (25 kPa), $w \geq 40$ percent, and $P_I > 20$, it shall be classified as Site Class E.

20.3.3 Site Classes C, D, and E. The existence of Site Class C, D, and E soils shall be classified by using one of the following three methods with $N_c$, $N$, and $\gamma_u$ computed in all cases as specified in Section 20.4:

1. $N_c$ for the top 100 ft (30 m) ($\alpha$ method).

2. $N$ for the top 100 ft (30 m) ($\alpha$ method).

3. $N_c$ for cohesionless soil layers ($P_I < 20$) in the top 100 ft (30 m) and $\gamma_u$ for cohesive soil layers ($P_I > 20$) in the top 100 ft (30 m) ($\alpha$ method). Where the $N_c$ and $\gamma_u$ criteria differ, the site shall be assigned to the category with the softer soil.

20.3.4 Shear Wave Velocity for Site Class B. The shear wave velocity for rock, Site Class II, shall be either measured on site or estimated by a geotechnical engineer, engineering geologist, or seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Site Class C.

20.3.5 Shear Wave Velocity for Site Class A. The hard rock, Site Class A, category shall be supported by shear wave velocity measurement either on site or in profiles of the same rock type in the same formation with an equal or greater degree of weathering.

### Table 20.3-1 SITE CLASSIFICATION

<table>
<thead>
<tr>
<th>Site Class</th>
<th>$V_s$ (ft/s)</th>
<th>$N$ or $N_c$</th>
<th>$\gamma_u$ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Hard rock</td>
<td>$&gt;5,000$</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>B. Rock</td>
<td>2,500 to 5,000</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>C. Very dense soil and soft rock</td>
<td>1,200 to 2,500</td>
<td>$&gt;50$</td>
<td>$&gt;2,000$</td>
</tr>
<tr>
<td>D. Stiff soil</td>
<td>600 to 1,200</td>
<td>15 to 50</td>
<td>1,000 to 2,000</td>
</tr>
<tr>
<td>E. Soft clay soil</td>
<td>$&lt;600$</td>
<td>$&lt;15$</td>
<td>$&lt;1,000$</td>
</tr>
</tbody>
</table>

Any profile with more than 10 ft of soil having the following characteristics:
- Plasticity index $P_I < 20$.
- Moisture content $w > 40$ percent.
- Undrained shear strength $\gamma_u < 500$ psf

For SI: 1 ft/s = 0.3048 m/s 1 lb/ft² = 0.0479 kN/m²

Minimum Design Loads for Buildings and Other Structures 205
FIGURE 22-1 continued
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTINENTAL UNITED STATES OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (6% OF CRITICAL DAMPING), SITE CLASS B