SCOPE: Add the new Chapter 24 to the 2009 Provisions

PROPOSAL FOR CHANGE:

Add the new Chapter 24 to the Provisions. The Code language is provided in Attachment A and the commentary language is provided in Attachment B.

REASON FOR PROPOSAL:

There has been concern in recent years over code complexity. This concern is general to all Seismic Design Categories, but the design of buildings in Seismic Design Category (SDC) B (by engineers not completely familiar with seismic design) is interpreted to be one of the large causative forces behind these concerns. The proposed Chapter 24 is a simplified alternative set of seismic design requirements for these SDC B buildings. The purpose of this new proposed chapter is to provide a simplified design procedure for SDC B, with the goals of both decreasing code-complexity for SDC B buildings and also decreasing the likelihood that the design requirements will be improperly applied in design.
ATTACHMENT A

Chapter 24

ALTERNATIVE SEISMIC DESIGN REQUIREMENTS
FOR SEISMIC DESIGN CATEGORY B BUILDINGS

Legend:

- Technical changes (and scope-related changes), with respect to ASCE7-10, are shown in orange.
- Editorial changes (and scope-related changes), with respect to ASCE7-10, are shown in green.
- Simpler editorial changes, with respect to ASCE7-10, associated with the editorial process of creating a single chapter from the four source chapters are shown in blue.

24.1 GENERAL

24.1.1 Scope and Applicability

The seismic analysis and design requirements in this chapter are permitted to be used in lieu of the requirements in Chapter 12 and Chapter 13 for the seismic analysis and design of structures assigned to Seismic Design Category B and for the design of parapets and exit stairways attached to those structures. Nonbuilding structures as defined in Chapter 15 and below, seismically isolated structures as defined in Chapter 17, and structures with damping systems as defined in Chapter 18, are not permitted to be designed by the procedures adopted.

Where the weight of a nonstructural component is greater than or equal to 25 percent of the effective seismic weight, W, of the structure as defined in Section 24.1.3, the component shall be classified as a nonbuilding structure and is not permitted to be designed in accordance with Chapter 24.

24.2 STRUCTURAL DESIGN BASIS

24.2.1 Basic Requirements

The building 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 horizontal direction of a building structure. The adequacy of the structural systems shall be demonstrated through the construction of a mathematical model and evaluation of this model for the effects of design ground motions. The design seismic forces, and their distribution over the height of the building structure, shall be established in accordance with one of the applicable procedures indicated in Section 24.7 and the corresponding internal forces and deformations in the members of the structure shall be determined. An approved alternative procedure shall not be used to establish the seismic forces and their distribution unless the corresponding internal forces and deformations in the members are determined using a model consistent with the procedure adopted.

24.2.2 Member Design, Connection Design, and Deformation Limit

Individual members, including those not part of the seismic force-resisting system, shall be provided with adequate strength to resist the shears, axial forces, and moments determined in accordance with this standard, and connections shall develop the strength of the connected members or the forces indicated in Section 24.2.1. The deformation of the structure shall not exceed the prescribed limits where the structure is subjected to the design seismic forces.

24.2.3 Continuous Load Path and Interconnection

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. All parts of the structure between separation joints shall be interconnected to form a continuous path to the seismic force-resisting system, and the connections shall be capable of transmitting the seismic force \( F_{ps} \) 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 design strength capable of transmitting a seismic force of 5 percent of the weight of the smaller portion. This connection force does not apply to the overall design of the seismic force-resisting system. Connection design forces need not exceed the maximum forces that the structural system can deliver to the connection.

24.2.4 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 dia-
phragm, 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.

24.2.5 Foundation Design

The foundation shall be designed to resist the forces developed and accommodate the movements imparted to the structure by the design ground motions. The dynamic nature of the forces, the expected ground motion, the design basis for strength and energy dissipation capacity of the structure, and the dynamic properties of the soil shall be included in the determination of the foundation design criteria. The design and construction of foundations shall comply with Section 24.14.

24.2.6 Material Design and Detailing Requirements

Structural elements including foundation elements shall conform to the material design and detailing requirements set forth in Chapter 14.

24.3 STRUCTURAL SYSTEM SELECTION

24.3.1 Selection and Limitations

The basic lateral and vertical seismic force-resisting system shall conform to one of the types indicated in Table 24.3-1 or a combination of systems as permitted in Sections 24.3.2, 24.3.3, and 24.3.4. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. The structural system used shall be in accordance with the structural system limitations and the limits on structural height, \( h_n \), contained in Table 24.3-1. The appropriate response modification coefficient, \( R \), overstrength factor, \( \Omega_b \), and the deflection amplification factor, \( C_d \), indicated in Table 24.3-1 shall be used in determining the base shear, element design forces, and design story drift.

Each selected seismic force-resisting system shall be designed and detailed in accordance with the specific requirements for the system as set forth in the applicable reference document listed in Table 24.3-1 and the additional requirements set forth in Chapter 14.

Seismic force-resisting systems not contained in Table 24.3-1 are permitted provided analytical and test data are submitted to the authority having jurisdiction for approval that establish their dynamic characteristics and demonstrate their lateral force resistance and energy dissipation capacity to be equivalent to the structural systems listed in Table 24.3-1 for equivalent values of response modification coefficient, \( R \), overstrength factor, \( \Omega_b \), and deflection amplification factor, \( C_d \).

24.3.2 Combinations of Framing Systems in Different Directions

Different seismic force-resisting systems are permitted to be used to resist seismic forces along each of the two orthogonal axes of the structure. Where different systems are used, the respective \( R \), \( C_d \), and \( \Omega_b \) coefficients shall apply to each system, including the structural system limitations contained in Table 24.3-1.

24.3.3 Combinations of Framing Systems in the Same Direction

Where different seismic force-resisting systems are used in combination to resist seismic forces in the same direction, other than those combinations considered as dual systems, the most stringent applicable structural system limitations contained in Table 24.3-1 shall apply and the design shall comply with the requirements of this section.

24.3.3.1 \( R, C_d \), and \( \Omega_b \) Values for Vertical Combinations

Where a structure has a combination in the same direction, the following requirements shall apply:

1. Where the lower system has a lower Response Modification Coefficient, \( R \), the design coefficients (\( R \), \( \Omega_b \), and \( C_d \)) for the upper system are permitted to be used to calculate the forces and drifts of the upper system. For the design of the lower system, the design coefficients (\( R \), \( \Omega_b \), and \( C_d \)) for the lower system shall be used. Forces transferred from the upper system to the lower system shall be increased by multiplying by the ratio of the higher response modification coefficient to the lower response modification coefficient.

2. Where the upper system has a lower Response Modification Coefficient, the Design Coefficients (\( R \), \( \Omega_b \), and \( C_d \)) for the upper system shall be used for both systems.

**EXCEPTIONS:**

1. Rooftop structures not exceeding two stories in height and 10 percent of the total structure weight.
2. Other supported structural systems with a weight equal to or less than 10 percent of the weight of the structure.

24.3.3.2 Two Stage Analysis Procedure

A two-stage equivalent lateral force procedure is permitted to be used for structures having a flexible upper portion above a rigid lower portion, provided the design of the structure complies with all of the following:

a. The stiffness of the lower portion shall be at least 10 times the stiffness of the upper portion.
b. The period of the entire structure shall not be greater than 1.1 times the period of the upper portion considered as a separate
structure supported at the transition from the upper to the lower portion.

c. The upper portion shall be designed as a separate structure using the appropriate value of $R$.

d. The lower portion shall be designed as a separate structure using the appropriate value of $R$. The reactions from the upper por-
tion shall be those determined from the analysis of the upper portion amplified by the ratio of $R$ of the upper portion over $R$ of
the lower portion. This ratio shall not be less than 1.0.

e. The upper portion is analyzed with the equivalent lateral force or modal response spectrum procedure, and the lower portion is
analyzed with the equivalent lateral force procedure.

**24.3.3 $R$, $C_d$ and $Q_o$ Values for Horizontal Combinations**

The value of the response modification coefficient, $R$, used for design in the direction under consideration shall not be greater
than the least value of $R$ for any of the systems utilized in that direction. The deflection amplification factor, $C_d$, and the over-
strength factor, $Q_o$, shall be consistent with $R$ required in that direction.

**EXCEPTION:** Resisting elements are permitted to be designed using the least value of $R$ for the different structural systems
found in each independent line of resistance if the following three conditions are met: (1) Risk Category I or II building, (2) two
stories or less above grade plane, and (3) use of light-frame construction or flexible diaphragms. The value of $R$ used for design of
diaphragms in such structures shall not be greater than the least value of $R$ for any of the systems utilized in that same direction.

**24.3.4 Combination Framing Detailing Requirements**

Structural members common to different framing systems used to resist seismic forces in any direction shall be designed using
the detailing requirements of this chapter required by the highest response modification coefficient, $R$, of the connected framing
systems.

**24.3.5 System Specific Requirements**

The structural framing system shall also comply with the following system specific requirements of this section.

**24.3.5.1 Dual System**

For a dual system, the moment frames shall be capable of resisting at least 25 percent of the design seismic forces. The total
seismic force resistance is to be provided by the combination of the moment frames and the shear walls or braced frames in pro-
portion to their rigidities.

**24.3.5.2 Cantilever Column Systems**

Cantilever column systems are permitted as indicated in Table 24.3-1 and as follows. The required axial strength of individual
cantilever column elements, considering only the load combinations that include seismic load effects, shall not exceed 15 percent
of the available axial strength, including slenderness effects.

Foundation and other elements used to provide overturning resistance at the base of cantilever column elements shall be de-
dsigned to resist the seismic load effects including overstrength factor of Section 24.5.3.

**24.3.5.3 Inverted Pendulum-Type Structures**

Regardless of the structural system selected, inverted pendulums as defined in Section 11.2, shall comply with this section.
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 Section 24.9 and varying uniformly to a moment at the top equal to one-half the calcul-
lated bending moment at the base.

**24.3.5.4 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.

**24.4 DIAPHRAGM FLEXIBILITY AND CONFIGURATION IRREGULARITIES**

**24.4.1 Diaphragm Flexibility**

The structural analysis shall consider the relative stiffnesses 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 24.4.1.1, 24.4.1.2, or
24.4.1.3, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semirigid modeling
assumption).

**24.4.1.1 Flexible Diaphragm Condition**

Diaphragms constructed of untapped steel decking or wood structural panels are permitted to be idealized as flexible if any of
the following conditions exist:
24.4.1.2 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.

24.4.1.3 Calculated Flexible Diaphragm Condition

Diaphragms not satisfying the conditions of Sections 24.4.1.1 or 24.4.1.2 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. 24.4-1. The loadings used for this calculation shall be those prescribed by Section 24.9.

24.4.2 Irregular and Regular Classification

Structures shall be classified as having a structural irregularity based upon the criteria in this section. Such classification shall be based on their structural configurations.

24.4.2.1 Horizontal Irregularity

Structures having one or more of the irregularity types listed in Table 24.4-1 shall be designated as having a horizontal structural irregularity. Such structures shall comply with the requirements in the sections referenced in that table.

24.4.2.2 Vertical Irregularity

Structures having one or more of the irregularity types listed in Table 24.4-2 shall be designated as having a vertical structural irregularity. Such structures shall comply with the requirements in the sections referenced in that table.

24.4.3 Limitations and Additional Requirements for Systems with Structural Irregularities

24.4.3.1 Extreme Weak Stories

Structures with a vertical irregularity Type 5b as defined in Table 24.4-2, shall not be over two stories or 30 ft (9 m) in structural height, \( h_c \).

EXCEPTION: The limit does not apply where the “weak” story is capable of resisting a total seismic force equal to \( \Omega_5 \) times the design force prescribed in Section 24.9.

24.4.3.2 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 24.4-1 or vertical irregularity Type 4 of Table 24.4-2 shall be designed to resist the seismic load effects including overstrength factor of Section 24.5.3. 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.

24.5 SEISMIC LOAD EFFECTS AND COMBINATIONS

24.5.1 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 24.5 unless otherwise exempted by this chapter. Seismic load effects are the axial, shear, and flexural member forces resulting from application of horizontal and vertical seismic forces as set forth in Section 24.5.2. Where specifically required, seismic load effects shall be modified to account for overstrength, as set forth in Section 24.5.3.

24.5.2 Seismic Load Effect

The seismic load effect, \( E \), shall be determined, based only on horizontal seismic forces, in accordance with Eq. 24.5-1 as follows:

\[
E = Q_e
\]  

(24.5-1)

where

\( E = \) seismic load effect
\[ Q_E \text{ = effects of horizontal seismic forces from } V \text{ or } F_p. \]

### 24.5.2.1 Seismic Load Combinations

Where the prescribed seismic load effect, \( E \), defined in Section 24.5.2 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. \( 1.2D + Q_E + L + 0.2S \)
6. \( 0.9D + Q_E + 1.6H \)

**NOTES:**
1. The load factor on \( L \) in combination 5 is permitted to equal 0.5 for all occupancies in which \( L_e \) in Table 4-1 is less than or equal to 100 psf (4.79 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 \( E \).
3. 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. \( 1.0D + H + F + 0.7Q_E \)
6. \( 1.0D + H + F + 0.525Q_E + 0.75L + 0.75(L_e \text{ or } S \text{ or } R) \)
7. \( 0.6D + 0.7Q_E + H \)

### 24.5.3 Seismic Load Effect Including Overstrength Factor

Where specifically required, conditions requiring overstrength factor applications shall be determined based only on horizontal seismic forces in accordance with the following:

\[ E_m = \Omega E \]  \hspace{1cm} (24.5-2)

where

- \( E_m \) = seismic load effect including overstrength factor
- \( Q_E \) = effects of horizontal seismic forces from \( V, F_p, \text{ or } F_r \) as specified in Sections 24.9.1, 24.11, or 24.15.3.1.
- \( \Omega \) = overstrength factor

### 24.5.3.1 Load Combinations with Overstrength Factor

Where the seismic load effect with overstrength factor, \( E_{m\Omega} \), defined in Section 24.5.3, 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 2.4.1:

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

5. \( 1.2D + \Omega Q_E + L + 0.2S \)
6. \( 0.9D + \Omega Q_E + 1.6H \)

**NOTES:**
1. The load factor on \( L \) in combination 5 is permitted to equal 0.5 for all occupancies in which \( L_e \) in Table 4-1 is less than or equal to 100 psf (4.79 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 \( E \).
3. 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. \( 1.0D + H + F + 0.7\Omega Q_E \)
6. \( 1.0D + H + F + 0.525\Omega Q_E + 0.75L + 0.75(L_e \text{ or } S \text{ or } R) \)
7. \( 0.6D + 0.7\Omega Q_E + H \)

### 24.5.3.2 Allowable Stress Increase for Load Combinations with Overstrength

Where allowable stress design methodologies are used with the seismic load effect defined in Section 24.5.3 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 or the material reference document except for increases due to adjustment factors in accordance with AF&PA NDS.
24.6 DIRECTION OF LOADING

The directions of application of seismic forces used in the design shall be those which will produce the most critical load effects. To satisfy this requirement, 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.

24.7 ANALYSIS PROCEDURE SELECTION

The structural analysis required by this chapter shall consist of either the Equivalent Lateral Force Analysis procedure (Section 24.9) or the Modal Response Spectrum Analysis procedure (Section 24.10).

24.8 MODELING CRITERIA

24.8.1 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 24.14.3.

24.8.2 Effective Seismic Weight

The effective seismic weight, \( W \), of a structure shall include the dead load, as defined in Section 3.1, above the base and other loads above the base as listed below:

1. In areas used for storage, a minimum of 25 percent of the floor live load shall be included.

   EXCEPTIONS:
   
   a. Where the inclusion of storage loads adds no more than 5% to the effective seismic weight at that level, it need not be included in the effective seismic weight.
   
   b. 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, \( P_f \), exceeds 30 psf (1.44 kN/m²), 20 percent of the uniform design snow load, regardless of actual roof slope.

5. Weight of landscaping and other materials at roof gardens and similar areas.

24.8.3 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 P-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.

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.

Structures that have horizontal structural irregularity Type 1a, 1b, 4, or 5 of Table 24.4-1 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 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 24.4.1, 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.

EXCEPTION: Analysis using a 3-D representation is not required for structures with flexible diaphragms that have Type 4 horizontal structural irregularities.

24.8.4 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 these rigid elements on the structural system at structural deformations corresponding to the design story drift \( \Delta \) as determined in Section 24.9.6. 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 24.4.2.
24.9 EQUIVALENT LATERAL FORCE PROCEDURE

24.9.1 Seismic Base Shear

The seismic base shear, \( V \), in a given direction shall be determined in accordance with the following equation:

\[
V = C_r W
\]  

(24.9-1)

where

\( C_r \) = the seismic response coefficient determined in accordance with Section 24.9.1

\( W \) = the effective seismic weight per Section 24.8.2

The seismic response coefficient, \( C_r \), shall be determined in accordance with Eq. 24.9-2.

\[
C_r = \frac{S_{DI}}{(R/L_e)}
\]  

(24.9-2)

where

\( S_{DI} \) = the design spectral response acceleration parameter in the short period range as determined from Section 11.4.4 or 11.4.7

\( R \) = the response modification factor in Table 24.3-1

\( I_e \) = the importance factor determined in accordance with Section 11.5.1

The value of \( C_r \) computed in accordance with Eq. 24.9-2 need not exceed the following:

\[
C_r = \frac{S_{DI}}{T(R/L_e)}
\]  

(24.9-3)

\( C_r \) shall not be less than

\[
C_r = 0.044S_{DI}I_e \geq 0.01
\]  

(24.9-4)

where \( I_e \) and \( R \) are as defined in Section 24.9.1 and

\( S_{DI} \) = the design spectral response acceleration parameter at a period of 1.0 s, as determined from Section 11.4.4 or 11.4.7

\( T \) = the fundamental period of the structure(s) determined in Section 24.9.2

\( S_1 \) = the mapped maximum considered earthquake spectral response acceleration parameter determined in accordance with Section 11.4.1 or 11.4.7

24.9.2 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 1.6\( T_a \), where \( T_a \) is determined in accordance with Section 24.9.2.1. 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 24.9.2.1, directly.

24.9.2.1 Approximate Fundamental Period

The approximate fundamental period (\( T_a \)), in s, shall be determined from the following equation:

\[
T_a = \frac{C_h x^4}{h_x}
\]  

(24.9-5)

where \( h_x \) is the structural height as defined in Section 11.2 and the coefficients \( C_h \) and \( x \) are determined from Table 24.9-1.

24.9.3 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:

\[
F_x = C_{vx} V
\]  

(24.9-6)

and

\[
C_{vx} = \frac{w_i h_x^k}{\sum w_i h_i^k}
\]  

(24.9-7)

where

\( C_{vx} \) = vertical distribution factor

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

\( w_i \) and \( w_x \) = the portion of the total 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:

- for structures having a period of 0.5 s or less, \( k = 1 \)
- for structures having a period of 2.5 s or more, \( k = 2 \)
- for structures having a period between 0.5 and 2.5 s, \( k \) shall be 2 or shall be determined by linear interpolation between 1 and 2

### 24.9.4 Horizontal Distribution of Forces

The seismic design story shear in any story \( (V_s) \) (kip or kN) shall be determined from the following equation:

\[
V_s = \sum_{i=x} F_i
\]  

(24.9-8)

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

The seismic design story shear \( (V_s) \) (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 diaphragm.

#### 24.9.4.1 Inherent Torsion

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

#### 24.9.4.2 Accidental Torsion

Where diaphragms are not flexible, the design shall include the inherent torsional moment \( (M_t) \) resulting from the location of the structure masses plus the accidental torsional moments \( (M_a) \) 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. The accidental torsional moment shall also be included in the determination of possible horizontal structural irregularities in Table 24.4-1.

**EXCEPTION:** The accidental torsional moments \( (M_a) \) need not be included in design of buildings that do not have a Type 1b horizontal structural irregularity.

### 24.9.5 Overturning

The structure shall be designed to resist overturning effects caused by the seismic forces determined in Section 24.12.3.

### 24.9.6 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. 24.9-1. Where centers of mass do not align vertically, it is permitted to compute the deflection at the bottom of the story based on the vertical projection of the center of mass at the top of the story. Where allowable stress design is used, \( \Delta \) shall be computed using the strength level seismic forces specified in Section 24.9 without reduction for allowable stress design.

The deflection at Level \( x \) \( (\delta_x) \) (in. or mm) used to compute the design story drift, \( \Delta \), shall be determined in accordance with the following equation:

\[
\delta_x = \frac{C_d \delta_{ce}}{I_e}
\]  

(24.9-9)

where

- \( C_d \) = the deflection amplification factor in Table 24.3-1
- \( \delta_{ce} \) = the deflection at the location required by this section determined by an elastic analysis
- \( I_e \) = the importance factor determined in accordance with Section 11.5.1

#### 24.9.6.1 Minimum Base Shear for Computing Drift

The elastic analysis of the seismic force-resisting system for computing drift shall be made using the prescribed seismic design forces of Section 24.9.

**EXCEPTION:** Eq. 24.9-4 need not be considered for computing drift.

#### 24.9.6.2 Period for Computing Drift

For determining compliance with the story drift limits of Section 24.13.1, it is permitted to determine the elastic drifts \( (\delta_{ce}) \), using seismic design forces based on the computed fundamental period of the structure without the upper limit \( (C_uT_a) \) specified in Section 24.9.2.
24.9.7 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 (θ) as determined by the following equation is equal to or less than 0.10:

$$\theta = \frac{P_x \Delta I_e}{V_x h_{ls} C_d}$$  \hspace{1cm} (24.9-10)

where

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

$\Delta =$ the design story drift as defined in Section 24.9.6 occurring simultaneously with $V_x$ (in. or mm)

$I_e =$ the importance factor determined in accordance with Section 11.5.1

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

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

$C_d =$ the deflection amplification factor in Table 24.3-1

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

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25$$  \hspace{1cm} (24.9-11)

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 (θ) 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 $1.0/(1 – \theta)$.

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. 24.9-11 shall still be satisfied, however, the value of $\theta$ computed from Eq. 24.9-10 using the results of the P-delta analysis is permitted to be divided by $(1 + \theta)$ before checking Eq. 24.9-11.

24.10 MODAL RESPONSE SPECTRUM ANALYSIS

24.10.1 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.

24.10.2 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 11.4.5 or 21.2 divided by the quantity $R/I_c$. The value for displacement and drift quantities shall be multiplied by the quantity $C/I_c$.

24.10.3 Combined Response Parameters

The value for each parameter of interest calculated for the various modes shall be combined using the square root of the sum of the squares (SRSS) method, the complete quadratic combination (CQC) method, the complete quadratic combination method as modified by ASCE 4 (CQC-4), or an approved equivalent approach. The CQC or the CQC-4 method shall be used for each of the modal values where closely spaced modes have significant cross-correlation of translational and torsional response.

24.10.4 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 24.9.

24.10.4.1 Scaling of Forces

Where the calculated fundamental period exceeds 1.6$cT_a$ in a given direction, 1.6$cT_a$ shall be used in lieu of $T$ in that direction. Where the combined response for the modal base shear ($V_e$) is less than 85 percent of the calculated base shear ($V$) using the equivalent lateral force procedure, the forces shall be multiplied by $0.85 V$:

$$V = \text{the equivalent lateral force procedure base shear, calculated in accordance with this section and Section 24.9}$$
$V_t =$ the base shear from the required modal combination

24.10.5 Horizontal Shear Distribution

The distribution of horizontal shear shall be in accordance with Section 24.9.4.

24.10.6 P-Delta Effects

The P-delta effects shall be determined in accordance with Section 24.9.7. The base shear used to determine the story shears and the story drifts shall be determined in accordance with Section 24.9.6.

24.11 DIAPHRAGMS, CHORDS, AND COLLECTORS

24.11.1 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.

24.11.1.1 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. 24.11-1 as follows:

$$ F_{px} = \frac{\sum F_i}{\sum w_i} $$

(24.11-1)

where

- $F_{px}$ = 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. 24.11-1 shall not be less than

$$ F_{px} = 0.2 S_{DS} I_e w_{px} $$

(24.11-2)

The force determined from Eq. 24.11-1 need not exceed

$$ F_{px} = 0.4 S_{DS} I_e w_{px} $$

(24.11-3)

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. 24.11-1.

24.11.2 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.

24.12 STRUCTURAL WALLS AND THEIR ANCHORAGE

24.12.1 Design for Out-of-Plane Forces

Structural walls and their anchorage shall be designed for a force normal to the surface equal to $F_p = 0.4 S_{DS} I_e$ times the weight of the structural wall with a minimum force of 10 percent of the weight of the structural wall. Interconnection of structural wall elements and connections to supporting framing systems shall have sufficient ductility, rotational capacity, or sufficient strength to resist shrinkage, thermal changes, and differential foundation settlement when combined with seismic forces.

24.12.2 Anchorage of Structural Walls

The anchorage of structural walls to supporting construction shall provide a direct connection capable of resisting the following force:

$$ F_p = 0.2 k_s I_e W_p $$

(24.12-1)
\[ k_a = 1.0 + L_a / 100 \]  

(24.12-2)

\( k_a \) need not be taken larger than 2.0.

where

- \( F_p \) = the design force in the individual anchors
- \( I_a \) = the importance factor determined in accordance with Section 11.5.1
- \( k_a \) = amplification factor for diaphragm flexibility.
- \( L_f \) = the span, in feet, of a flexible diaphragm that provides the lateral support for the wall; the span is measured between vertical elements that provide lateral support to the diaphragm in the direction considered; use zero for rigid diaphragms
- \( W_p \) = the weight of the wall tributary to the anchor

Where the anchorage is not located at the roof and all diaphragms are not flexible, the value from Eq. 24.12-1 is permitted to be multiplied by the factor \((1 + 2z/h)/3\), where \( z \) is the height of the anchor above the base of the structure and \( h \) is the height of the roof above the base.

Structural walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 ft (1,219 mm).

**24.13 DRIFT AND DEFORMATION**

**24.13.1 Story Drift Limit**

The design story drift (\( \Delta \)) as determined in Sections 24.9.6 or 24.10.2, shall not exceed the allowable story drift (\( \Delta_a \)) as obtained from Table 24.13-1 for any story.

**24.13.2 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.

**24.13.3 Structural 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 as set forth in this section.

Separations shall allow for the maximum inelastic response displacement (\( \delta_{in} \)). \( \delta_{in} \) shall be determined at critical locations with consideration for translational and torsional displacements of the structure using the following equation:

\[
\delta_M = \frac{C_d \delta_{max}}{I_a} \]  

[Editorial Note: Need to fix equation format.]

(24.13-1)

Where \( \delta_{max} = \) maximum elastic displacement at the critical location.

Adjacent structures on the same property shall be separated by at least \( \delta_{MT} \), determined as follows:

\[
\delta_{MT} = \sqrt{(\delta_M)^2 + (\delta_{M2})^2} \]  

(24.13-2)

where \( \delta_M \) and \( \delta_{M2} \) are the maximum inelastic response displacements of the adjacent structures at their adjacent edges.

Where a structure adjoins a property line not common to a public way, the structure shall be set back from the property line by at least the displacement \( \delta_M \) of that structure.

**EXCEPTION:** Smaller separations or property line setbacks are permitted where justified by rational analysis based on inelastic response to design ground motions.

**24.13.4 Members Spanning between Structures**

Gravity connections or supports for members spanning between structures or seismically separate portions of structures shall be designed for the maximum anticipated relative displacements. These displacements shall be calculated:

1. Using the deflection calculated at the locations of support, per Eq. 24.9-9 multiplied by \( 1.5R/C_d \) and
2. Considering additional deflection due to diaphragm rotation, and
3. Considering diaphragm deformations, and
4. Assuming the two structures are moving in opposite directions and using the absolute sum of the displacements.

**24.14 FOUNDATION DESIGN**

**24.14.1 Design Basis**

The design basis for foundations shall be as set forth in Section 24.2.5.
24.14.2 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 14.1.7 Design and detailing of concrete piles shall comply with Section 14.2.3.

24.14.3 Foundation Load-Deformation Characteristics
Where foundation flexibility is included for the linear analysis procedures in this chapter, the load-deformation characteristics of the foundation–soil system (foundation stiffness) shall be modeled in accordance with the requirements of this section. The linear load-deformation 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.

24.14.4 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 24.9.

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 24.10.

24.15 SEISMIC DESIGN REQUIREMENTS FOR EGRESS STAIRWAYS AND PARAPETS

24.15.1 Scope
This section establishes minimum design criteria for parapets and egress stairways and their supports and attachments in Seismic Design Category B. All other nonstructural components and their supports and attachments are exempt from the requirements of Section 24.15.

24.15.2 General Design Requirements

24.15.2.1 Submittal Requirements
Evidence demonstrating compliance with the requirements of this section shall be submitted for approval to the authority having jurisdiction after review and acceptance by a registered design professional. Parapets and egress stairways may also be seismically qualified by analysis, testing, or experience data in accordance with Section 13.2.1.

24.15.2.2 Construction Documents
The design of parapets and egress stairways, and their supports and attachments, shall be shown in construction documents prepared by a registered design professional for use by the owner, authorities having jurisdiction, contractors, and inspectors.

24.15.3 Seismic Design Force
Parapets and egress stairways, and their supports and attachments, shall be designed for the seismic forces defined in this section. Where nonseismic loads on nonstructural components exceed \( F_p \), such loads shall govern the strength design, but the limitations prescribed in this chapter shall apply.

The horizontal seismic design force \( F_p \) shall be applied at the component’s center of gravity and distributed relative to the component’s mass distribution and shall be determined in accordance with Eq. 24.15-1:

\[
F_p = \frac{0.4a_p S_{Dh} W_p}{R_p} \left( 1 + 2 \frac{z}{h} \right) \quad (24.15-1)
\]

and \( F_p \) shall not be taken as less than
\[ F_p = 0.3S_{DSI_p}W_p \]  

(24.15-2)

where

\[ F_p = \text{horizontal seismic design force applied to the parapet or egress stairway} \]
\[ S_{DS} = \text{spectral acceleration, short period, as determined from Section 11.4.4} \]
\[ I_p = \text{component importance factor} \]
\[ W_p = \text{weight of the parapet or egress stairway} \]
\[ R_p = \text{component response modification factor} \]
\[ z = \text{height in structure of point of attachment of parapet or egress stairway with respect to the base of the structure}. \]
\[ h = \text{average roof height of structure with respect to the base of the structure}. \]

The force \( F_p \) shall be applied independently in at least two orthogonal horizontal directions in combination with service loads associated with the component, as appropriate. For vertically cantilevered systems, however, the force \( F_p \) shall be assumed to act in any horizontal direction. The overstrength factor, \( \Omega_{o} \), does not apply.

### 24.15.4 Design of Egress Stairways for Seismic Relative Displacements

Egress stairways, and their supports and attachments, shall be designed to accommodate the seismic relative displacement requirements of this section. **Egress stairways shall be designed considering vertical deflection due to joint rotation of cantilever structural members.**

The effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate. Seismic relative displacements, \( D_p \), shall be determined in accordance with Eq. 24.15-3 as:

\[ D_p = D_p I_e \]  

(24.15-3)

where

\[ I_e = \text{the importance factor in Section 11.5.1} \]
\[ D_p = \text{displacement determined in accordance with the equations set forth in Section 24.15.3.2}. \]

**24.15.4.1 Displacements within Structures**

For two connection points on the same Structure A or the same structural system, one at a height \( h_a \), and the other at a height \( h_b \), \( D_p \) shall be determined as

\[ D_p = \delta_{yA} - \delta_{yA} \]  

(24.15-4)

Alternatively, \( D_p \) is permitted to be determined using modal procedures described in Section 24.10, using the difference in story deflections calculated for each mode and then combined using appropriate modal combination procedures. \( D_p \) is not required to be taken as greater than

\[ D_p = \frac{(h_{yA} - h_{yA}) \Delta_{xA}}{h_{xA}} \]  

(24.15-5)

**24.15.4.2 Displacements between Structures**

For two connection points on separate Structures A and B or separate structural systems, one at a height \( h_x \) and the other at a height \( h_y \), \( D_p \) shall be determined as

\[ D_p = |\delta_{yA}| + |\delta_{yB}| \]  

(24.15-6)

\( D_p \) is not required to be taken as greater than

\[ D_p = \frac{h_x \Delta_{yA} + h_y \Delta_{yB}}{h_{xA}} \]  

(24.15-7)

where

\[ D_p = \text{relative seismic displacement that the component must be designed to accommodate} \]
\[ \Delta_{xA} = \text{deflection at building Level x of Structure A, determined in accordance with Eq. (24.9-9)} \]
\[ \Delta_{yA} = \text{deflection at building Level y of Structure A, determined in accordance with Eq. (24.9-9)} \]
\[ \Delta_{yB} = \text{deflection at building Level y of Structure B, determined in accordance with Eq. (24.9-9)} \]
\[ h_x = \text{height of Level x to which upper connection point is attached} \]
\[ h_y = \text{height of Level \( y \) to which lower connection point is attached} \]
\[ \Delta_{aA} = \text{allowable story drift for Structure A as defined in Table 24.13-1} \]
\[ \Delta_{aB} = \text{allowable story drift for Structure B as defined in Table 24.13-1} \]
\[ h_{sx} = \text{story height used in the definition of the allowable drift} \Delta_a \text{ in Table 24.13-1}. \text{ Note that } \Delta_a/h_{sx} = \text{the drift index}. \]

The effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate.

24.15.5 Out-of-Plane Bending

Transverse or out-of-plane bending or deformation of a parapet or egress stairway subjected to forces as determined in Section 24.15.3, or displacements as determined in Section 24.15.4, shall not exceed the deflection capability of the parapet or egress stairway.

24.15.6 Anchorage

Parapet and egress stairway components, and their supports, shall be attached (or anchored) to the structure in accordance with the requirements of this section and the attachment shall satisfy the requirements for the parent material as set forth elsewhere in this standard.

Parapets and egress stairways, and their supports, shall be bolted, welded, or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity. A continuous load path of sufficient strength and stiffness shall be provided between the parapet or egress stairway and the supporting structure. Local elements of the structure including connections shall be designed and constructed for the forces in the attachment where they control the design of the elements or their connections. The design documents shall include sufficient information relating to the attachments to verify compliance with the requirements of this section.

24.15.6.1 Design Force in the Attachment

The force in the attachment shall be determined based on the prescribed forces and displacements for the parapet or egress stairway as determined in Sections 24.15.3 and 24.15.4.

24.15.6.2 Anchors in Concrete or Masonry

Anchors in concrete shall be designed in accordance with Appendix D of ACI 318.

Anchors in masonry shall be designed in accordance with TMS 402/ACI 503/ASCE 5. Anchors shall be governed by the tensile or shear strength of a ductile steel element.

**EXCEPTION:** Anchors in masonry shall be permitted to be designed so that the support that the anchor is connecting to the structure undergoes ductile yielding at a load level corresponding to anchor forces not greater than their design strength, or the minimum design strength of the anchors shall be at least 2.5 times the factored forces transmitted by the parapet or egress stairway.

Post-installed anchors in concrete shall be prequalified for seismic applications in accordance with ACI 355.2, ACI 355.4 or other approved qualification procedures. Post-installed anchors in masonry shall be prequalified for seismic applications in accordance with approved qualification procedures.

24.15.6.3 Installation Conditions

Determination of forces in attachments shall take into account the expected conditions of installation including eccentricities and prying effects.

24.15.6.4 Multiple Attachments

Determination of force distribution of multiple attachments at one location shall take into account the stiffness and ductility of the component, component supports, attachments, and structure and the ability to redistribute loads to other attachments in the group. Designs of anchorage in concrete in accordance with Appendix D of ACI 318 shall be considered to satisfy this requirement.

24.15.6.5 Power Actuated Fasteners

Power actuated fasteners in masonry are not permitted unless approved for seismic loading.
### Figures and Tables

#### Table 24.3-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

<table>
<thead>
<tr>
<th>Seismic Force-Resisting System</th>
<th>ASCE 7 Section Where Detailing Requirements Are Specified</th>
<th>Response Modification Coefficient, $R^a$</th>
<th>Overstrength Factor, $\Omega^b$</th>
<th>Deflection Amplification Factor, $C_d^c$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>A. BEARING WALL SYSTEMS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. Ordinary reinforced concrete shear walls</td>
<td>14.2</td>
<td>4</td>
<td>2½</td>
<td>4</td>
</tr>
<tr>
<td>5. Intermediate precast shear walls</td>
<td>14.2</td>
<td>4</td>
<td>2½</td>
<td>4</td>
</tr>
<tr>
<td>6. Ordinary precast shear walls</td>
<td>14.2</td>
<td>3</td>
<td>2½</td>
<td>3</td>
</tr>
<tr>
<td>8. Intermediate reinforced masonry shear walls</td>
<td>14.4</td>
<td>3½</td>
<td>2½</td>
<td>2½</td>
</tr>
<tr>
<td>9. Ordinary reinforced masonry shear walls</td>
<td>14.4</td>
<td>2</td>
<td>2½</td>
<td>1½</td>
</tr>
<tr>
<td>13. Ordinary reinforced AAC masonry shear walls</td>
<td>14.4</td>
<td>2</td>
<td>2½</td>
<td>2</td>
</tr>
<tr>
<td>14. Ordinary plain AAC masonry shear walls</td>
<td>14.4</td>
<td>1½</td>
<td>2½</td>
<td>1½</td>
</tr>
<tr>
<td>15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance or steel sheets</td>
<td>14.1 and 14.5</td>
<td>6½</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>16. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets</td>
<td>14.1</td>
<td>6½</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>17. Light-frame walls with shear panels of all other materials</td>
<td>14.1 and 14.5</td>
<td>2</td>
<td>2½</td>
<td>2</td>
</tr>
<tr>
<td>18. Light-frame (cold-formed steel) wall systems using flat strap bracing</td>
<td>14.1</td>
<td>4</td>
<td>2</td>
<td>3½</td>
</tr>
<tr>
<td><strong>B. BUILDING FRAME SYSTEMS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3. Steel ordinary concentrically braced frames</td>
<td>14.1</td>
<td>3½</td>
<td>2</td>
<td>3½</td>
</tr>
<tr>
<td>5. Ordinary reinforced concrete shear walls</td>
<td>14.2</td>
<td>5</td>
<td>2½</td>
<td>4½</td>
</tr>
<tr>
<td>8. Intermediate precast shear walls</td>
<td>14.2</td>
<td>5</td>
<td>2½</td>
<td>4½</td>
</tr>
<tr>
<td>9. Ordinary precast shear walls</td>
<td>14.2</td>
<td>4</td>
<td>2½</td>
<td>4</td>
</tr>
<tr>
<td>17. Intermediate reinforced masonry shear walls</td>
<td>14.4</td>
<td>4</td>
<td>2½</td>
<td>4</td>
</tr>
<tr>
<td>18. Ordinary reinforced masonry shear walls</td>
<td>14.4</td>
<td>2</td>
<td>2½</td>
<td>2</td>
</tr>
<tr>
<td>22. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance</td>
<td>14.5</td>
<td>7</td>
<td>2½</td>
<td>4½</td>
</tr>
<tr>
<td>Seismic Force-Resisting System</td>
<td>ASCE 7 Section</td>
<td>Where Detailing Requirements Are Specified</td>
<td>Response Modification Coefficient, $R^a$</td>
<td>Overstrength Factor, $\Omega_0$</td>
</tr>
<tr>
<td>-------------------------------</td>
<td>----------------</td>
<td>---------------------------------------------</td>
<td>------------------------------------------</td>
<td>---------------------------------</td>
</tr>
<tr>
<td>23. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets</td>
<td>14.1</td>
<td>7</td>
<td>$2\frac{1}{2}$</td>
<td>$4\frac{1}{2}$</td>
</tr>
<tr>
<td>24. Light-frame walls with shear panels of all other materials</td>
<td>14.1 and 14.5</td>
<td>$2\frac{1}{2}$</td>
<td>$2\frac{1}{2}$</td>
<td>$2\frac{1}{2}$</td>
</tr>
<tr>
<td><strong>C. MOMENT-RESISTING FRAME SYSTEMS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3. Steel intermediate moment frames</td>
<td>14.1</td>
<td>$4\frac{1}{2}$</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>4. Steel ordinary moment frames</td>
<td>14.1</td>
<td>$3\frac{1}{2}$</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>6. Intermediate reinforced concrete moment frames</td>
<td>14.2</td>
<td>5</td>
<td>3</td>
<td>$4\frac{1}{2}$</td>
</tr>
<tr>
<td>7. Ordinary reinforced concrete moment frames</td>
<td>14.2</td>
<td>3</td>
<td>3</td>
<td>$2\frac{1}{2}$</td>
</tr>
<tr>
<td>9. Steel and concrete composite intermediate moment frames</td>
<td>14.3</td>
<td>5</td>
<td>3</td>
<td>$4\frac{1}{2}$</td>
</tr>
<tr>
<td>10. Steel and concrete composite partially restrained moment frames [System is limited to a structural height, $h_{gr}$, of 160 ft (48.8 m)]</td>
<td>14.3</td>
<td>6</td>
<td>3</td>
<td>$5\frac{1}{2}$</td>
</tr>
<tr>
<td>11. Steel and concrete composite ordinary moment frames</td>
<td>14.3</td>
<td>3</td>
<td>3</td>
<td>$2\frac{1}{2}$</td>
</tr>
<tr>
<td><strong>E. DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES</strong></td>
<td>24.3.5.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3. Ordinary reinforced masonry shear walls</td>
<td>14.4</td>
<td>3</td>
<td>3</td>
<td>$2\frac{1}{2}$</td>
</tr>
<tr>
<td>4. Intermediate reinforced masonry shear walls</td>
<td>14.4</td>
<td>$3\frac{1}{2}$</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>6. Steel and concrete composite ordinary braced frames</td>
<td>14.3</td>
<td>$3\frac{1}{2}$</td>
<td>$2\frac{1}{2}$</td>
<td>3</td>
</tr>
<tr>
<td>7. Steel and concrete composite ordinary shear walls</td>
<td>14.3</td>
<td>5</td>
<td>3</td>
<td>$4\frac{1}{2}$</td>
</tr>
<tr>
<td>8. Ordinary reinforced concrete shear walls$^d$</td>
<td>14.2</td>
<td>$5\frac{1}{2}$</td>
<td>$2\frac{1}{2}$</td>
<td>$4\frac{1}{2}$</td>
</tr>
<tr>
<td><strong>F. SHEAR WALL-FRAME INTERACTIVE SYSTEM WITH ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS</strong></td>
<td>24.3.5.4 and 14.2</td>
<td>$4\frac{1}{2}$</td>
<td>$2\frac{1}{2}$</td>
<td>4</td>
</tr>
<tr>
<td><strong>G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR</strong> [System is</td>
<td>24.3.5.2</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

Proposal SWG2-001 (2013)
Where Detailing Requirements Are Specified

Seismic Force-Resisting System

<table>
<thead>
<tr>
<th>Seismic Force-Resisting System</th>
<th>ASCE 7 Section Where Detailing Requirements Are Specified</th>
<th>Response Modification Coefficient, $R^a$</th>
<th>Overstrength Factor, $\Omega_0$</th>
<th>Deflection Amplification Factor, $C_d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>limited to a structural height, $h_n$, of 35 ft (10.7 m):</td>
<td>14.1</td>
<td>1¼</td>
<td>1¼</td>
<td>1¼</td>
</tr>
<tr>
<td>2. Steel ordinary cantilever column systems</td>
<td>14.2</td>
<td>1½</td>
<td>1¼</td>
<td>1½</td>
</tr>
<tr>
<td>4. Intermediate reinforced concrete moment frames</td>
<td>14.2</td>
<td>1½</td>
<td>1¼</td>
<td>1½</td>
</tr>
<tr>
<td>5. Ordinary reinforced concrete moment frames</td>
<td>14.2</td>
<td>1</td>
<td>1¼</td>
<td>1</td>
</tr>
<tr>
<td>6. Timber frames</td>
<td>14.5</td>
<td>1½</td>
<td>1½</td>
<td>1½</td>
</tr>
<tr>
<td>H. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE, EXCLUDING CANTILEVER COLUMN SYSTEMS</td>
<td>14.1</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>

$^a$Response modification coefficient, $R$, for use throughout the standard. Note $R$ reduces forces to a strength level, not an allowable stress level.

$^b$Deflection amplification factor, $C_d$, for use in Sections 24.9.6, 24.9.7, and 24.10.2.

$^c$Where the tabulated value of the overstrength factor, $\Omega_0$, is greater than or equal to 2½, $\Omega_0$ is permitted to be reduced by subtracting the value of 1/2 for structures with flexible diaphragms.

$^d$In Section 2.2 of ACI 318. A shear wall is defined as a structural wall.

**FIGURE 24.4-1 Flexible Diaphragm**

**Table 24.4-1 Horizontal Structural Irregularities**

<table>
<thead>
<tr>
<th>Type</th>
<th>Description</th>
<th>Reference Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a.</td>
<td><strong>Torsional Irregularity:</strong> Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, 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>24.8.3</td>
</tr>
<tr>
<td>1b.</td>
<td><strong>Extreme Torsional Irregularity:</strong> Extreme torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, 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>24.8.3 24.9.4.2</td>
</tr>
<tr>
<td>2.</td>
<td><strong>Out-of-Plane Offset Irregularity:</strong> Out-of-plane offset irregularity is defined to exist where there is a discontinuity in a lateral force-resistance path, such as an out-of-plane offset of at least one of the vertical elements.</td>
<td>24.4.3.2 24.8.3</td>
</tr>
<tr>
<td>3.</td>
<td><strong>Nonparallel System Irregularity:</strong> Nonparallel system irregularity is defined to exist where vertical lateral force-resisting elements are not parallel to the major orthogonal axes of the seismic force-resisting system.</td>
<td>24.8.3</td>
</tr>
</tbody>
</table>
Table 24.4-2 Vertical Structural Irregularities

<table>
<thead>
<tr>
<th>Type</th>
<th>Description</th>
<th>Reference Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.</td>
<td>In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity: In-plane discontinuity in vertical lateral force-resisting elements irregularity is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on a supporting beam, column, truss, or slab.</td>
<td>24.4.3.2</td>
</tr>
<tr>
<td>5b.</td>
<td>Discontinuity in Lateral Strength–Extreme Weak Story Irregularity: Discontinuity in lateral strength–extreme weak story irregularity is defined to exist where the story lateral strength is less than 65% 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>24.4.3.1</td>
</tr>
</tbody>
</table>

Table 24.9-1 Values of Approximate Period Parameters $C_t$ and $x$

<table>
<thead>
<tr>
<th>Structure Type</th>
<th>$C_t$</th>
<th>$x$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel moment-resisting frames</td>
<td>0.028 (0.0724)$^a$</td>
<td>0.8</td>
</tr>
<tr>
<td>Concrete moment-resisting frames</td>
<td>0.016 (0.0466)$^a$</td>
<td>0.9</td>
</tr>
<tr>
<td>All other structural systems</td>
<td>0.02 (0.0488)$^a$</td>
<td>0.75</td>
</tr>
</tbody>
</table>

$^a$Metric equivalents are shown in parentheses.

FIGURE 24.9-1 Story Drift Determination

FIGURE 24.11-1 Collectors

Table 24.13-1 Allowable Story Drift, $\Delta_a$

<table>
<thead>
<tr>
<th>Structure</th>
<th>Risk Category</th>
<th>I or II</th>
<th>III</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structures, other than masonry shear wall structures, 4 stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts.</td>
<td>$0.025h_{sx}$</td>
<td>$0.020h_{sx}$</td>
<td></td>
</tr>
<tr>
<td>Masonry cantilever shear wall structures$^c$</td>
<td>$0.010h_{sx}$</td>
<td>$0.010h_{sx}$</td>
<td></td>
</tr>
<tr>
<td>Other masonry shear wall structures</td>
<td>$0.007h_{sx}$</td>
<td>$0.007h_{sx}$</td>
<td></td>
</tr>
<tr>
<td>All other structures</td>
<td>$0.020h_{sx}$</td>
<td>$0.015h_{sx}$</td>
<td></td>
</tr>
</tbody>
</table>

$h_{sx}$ is the story height below Level $x$.

$^a$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 24.13.3 is not waived.

$^c$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.
ATTACHMENT B

COMMENTARY to Chapter 24
ALTERNATIVE SEISMIC DESIGN REQUIREMENTS
FOR SEISMIC DESIGN CATEGORY B BUILDINGS

C24.1 General

In recent years, engineers and building officials have become concerned that the seismic design requirements for Seismic Design Category (SDC) B are complex and are difficult to implement because the SDC B requirements could not be easily separated from the many other seismic design requirements that are not applicable to SDC B. Additionally, a systematic examination of SDC B design requirements was warranted, because some of the existing Chapter 12 and Chapter 13 requirements may be unnecessary for the design of buildings at sites with moderate seismicity since the requirements have only a minimal influence on design.

In accordance with Section 11.1.3, the alternative seismic design procedure presented in this chapter may be used for the structural systems and nonstructural components of buildings assigned to SDC B. This chapter is equivalent to the procedures described in Chapters 12 and 13 of this Standard, but differs in two ways. First, the text and requirements presented in this chapter are substantially simpler and shorter, because the chapter has been editorially simplified to only include the requirements that apply in SDC B. Second, some of the seismic design requirements have been eliminated or simplified based on technical study. These technical simplifications apply to seismic design requirements which are applicable in SDC B, in accordance with Chapters 12 and 13, but do not have significant influence on the resulting design or seismic performance. As described in more detail below, the implications of removing or simplifying seismic design requirements was carefully evaluated through design studies and nonlinear structural analyses. The commentary that follows describes the important differences between Chapter 24 and the seismic design requirements of Chapter 12 and 13.

Nonbuilding structures (Chapter 15), seismically isolated structures (Chapter 17), and structures with damping systems (Chapter 18) are not permitted to be designed using the alternative procedures of Chapter 24.

C24.2 Structural Design Basis

The requirements of this section closely follow those of Section 12.1. The only modifications in this section are editorial simplifications, in which requirements have been eliminated when not applicable to SDC B buildings. In particular, a small change has been made in the design strength calculation for connections. In SDC B, all connections must be designed for 5% of the weight of the smaller portion of the structure. There is no need to calculate 0.133 times $S_{DS}$, as required in Chapter 12, because the 5% limiting value will always govern designs in SDC B.

C24.3 Structural System Selection

The requirements of this section closely follow those of Section 12.2. The only modifications in this section are editorial simplifications, in which requirements have been eliminated when not applicable to SDC B buildings. For example, numerous requirements found in Section 12.2, e.g. the requirements for Steel Intermediate Moment Frames in SDC D, have been eliminated because they are not applicable to SDC B buildings.

Additionally, the Table of Design Coefficients and Factors for Seismic Force-Resisting Systems (Table 24.3-1) has been substantially editorially simplified. Structural systems not commonly used in SDC B have been removed, including all “special” systems, which are used primarily in the higher SDCs. When rows were deleted from the Table of Design Coefficients and Factors for Seismic Force-Resisting Systems (Table 24.3-1), the numbering of the rows was intentionally kept unchanged and identical to the numbering used in Table 12.2-1. In addition, the columns relating to Structural System Limitations have been removed because all systems in the table are allowable in SDC B. The few remaining systems that have height limits imposed in SDC B have the height limits listed directly in the table, rather than in a separate column.

C24.4 Diaphragm Flexibility and Configuration Irregularities

The requirements of this section closely follow those of Section 12.3. The only modifications in this section are editorial simplifications, in which requirements have been eliminated when not applicable to SDC B buildings. The tables defining Horizontal Structural Irregularities and Vertical Structural Irregularities (Tables 24.4-1 and 24.2-2) have been simplified to only include the irregularities that affect the design procedures in SDC B. Other irregularities, while they may be present, do not affect the design requirements and have been eliminated from the table. The numbering of the irregularities in Table 24.4-1 and Table
24.4.2 was intentionally kept identical to those of Tables 12.3-1 and 12.3-2. The irregularities of Tables 12.3-1 and 12.3-2 omitted from the Chapter 24 tables are horizontal irregularities Type 2 and 3, and vertical irregularities Type 1a, 1b, 2, 3, and 5a. These irregularities were omitted because they do not apply to SDC B.

C24.5 Seismic Load Effects and Combinations

The equations for seismic load effects and load combinations in the alternative design procedure are consistent with those for the general procedure of Chapter 12, with the one notable exception being that the requirement for including the vertical seismic load effect has been removed. Accordingly, $E_v$ is taken as zero in the Section 24.5 requirements and the $E_v = 0.2S_{DS}D$ term in the design load combinations has been removed.

The elimination of the vertical load effect requirement in SDC B was supported by design studies. These studies indicated that, due to the small $S_{DS}$ values in SDC B and, the small associated increase in design dead loads due to vertical seismic effects, there is no meaningful difference in member sizes and detailing if the vertical seismic load is considered in SDC B. Note that in the general Chapter 12 requirement, $E_v$ may already be taken as zero when $S_{DS} < 0.125g$, so this change simply expands the range of $S_{DS}$ values for which $E_v$ may be zero up to $S_{DS} < 0.33g$.

Additionally, the redundancy factor, $p$, has been removed from the load combinations because this factor is always equal to unity for SDC B buildings.

The final simplification in Section 24.5 is that the seismic load effect including the overstrength factor, $E_{ov}$, must be computed using Equation 24.5-2 and the exception has been removed. If the designer wants to use the more complex method of computing the maximum force that can be developed in the element, then Chapter 24 cannot be used and the general procedures of Chapters 12 and 13 must be used.

C24.6 Direction of Loading

The requirements of this section closely follow those of Section 12.5. Most of the text in Section 12.5 is related to SDC C and above, so the procedures in Section 24.6 have been shortened substantially.

C24.7 Analysis Procedure Selection

The structural analysis procedure must be either the Equivalent Lateral Force Analysis or the Modal Analysis procedure. If a designer desires to use the more advanced response-history analysis procedure (with the approval of the authority having jurisdiction), then Chapter 24 cannot be used, and the building must be designed in accordance with the provisions in Chapters 12, 13, and 16.

C24.8 Modeling Criteria

The requirements of this section closely follow those of Section 12.7 and only small editorial changes have been made.

C24.9 Equivalent Lateral Force Procedure

In this section, the seismic design requirements have been simplified using both editorial and technical simplifications. The discussion below describes the technical differences between the general procedures of Chapter 12 and the alternative procedures of this chapter.

C24.9.1 Seismic Base Shear

Determination of the seismic base shear is similar to the general procedure of Chapter 12. The primary technical simplification is the elimination of the long-period region of the spectrum, i.e. for $T > T_L$. In the Chapter 24 design procedure, longer period structures are to be designed following the same $1/T$ spectral shape used in the velocity sensitive region of the spectrum. The elimination of the long period region of the spectra is conservative, but it is not expected that it will affect many, if any, designs in SDC B.

Reductions associated with soil structure interaction are not permitted when using the alternative Chapter 24 design procedures.

C24.9.2 Period Determination

The approximate period, $T_a$, is computed according to Equation 24.9-5, and the other period determination equations from Chapter 12 have been eliminated for simplicity. As in Chapter 12, the fundamental period of the structure may not exceed $C_vT_a$, but in these alternative procedures, for simplicity, $C_v$ is taken as a constant value of 1.6. This 1.6 value is used because the Chapter 12 $C_v$ values range only from 1.6 to 1.7 for all sites in SDC B. Use of the constant lower-bound 1.6 value is both simpler and
slightly conservative, but will not result in any substantial change in the building design.

C24.9.4 Accidental Torsion

To simplify the process of computing member forces from seismic effects, the accidental torsional moment need not be included in design of SDC B buildings, unless the building has a Type 1b horizontal irregularity (Extreme Torsional Irregularity).

The decision to remove the accidental torsion requirement for most regular buildings is supported by rigorous analytical studies using nonlinear dynamic analysis of SDC B buildings designed both with and without use of the accidental torsion requirements. These analytical studies showed that the collapse resistance of buildings was not significantly altered if the accidental torsion requirements were eliminated in the design, for buildings with a torsional irregularity ratio of up to 1.4 (which is the torsional irregularity ratio corresponding to Type 1b horizontal irregularity). For structures with extreme torsional irregularities, the additional strength resulting from the use of the accidental torsion design requirements becomes critical for maintaining sufficient building collapse resistance. The details of this study, including the detailed design information for the 240 buildings analyzed, are available in Liel et al. (2012).

C24.10 Modal Response Spectrum Analysis

The requirements of this section closely follow those of Section 12.9 and only small changes have been made. The section on Scaling of Drifts was removed for editorial reasons because it does not apply to SDC B. Also, for simplicity, reductions associated with soil structure interaction are not permitted when using these Chapter 24 alternative procedures, and the associated guidelines were removed from the simplified procedure.

C24.11 Diaphragms, Chords and Collectors

The requirements of this section closely follow those of Section 12.10, and only minor editorial simplifications were made to remove requirements not applicable to SDC B buildings.

C24.12 Structural Walls and Their Anchorage

The requirements of this section closely follow those of Section 12.11. The primary technical change pertains to the calculation of the amplification factor for diaphragm flexibility, \(k_a\). Instead of interpolating between values of 1.0 and 2.0, \(k_a\) is simply taken as 1.0 for rigid diaphragms and 2.0 for flexible diaphragms. This use of 2.0 for flexible diaphragms, which is independent of the span length, is a conservative simplification.

C24.13 Drift and Deformation

The requirements of this section closely follow those of Section 12.12. The only modifications in this section are editorial simplifications, in which requirements have been eliminated when not applicable to SDC B buildings.

One specific editorial simplification is that the table for Allowable Story Drifts (Table 24.13-1) has been simplified to only provide the displacement limits for Risk Categories I, II and III, since it is not possible for Risk Category IV to occur in SDC B.

C24.14 Foundation Design

The requirements of this section closely follow those of Section 12.13. The only modifications in this section are editorial simplifications, in which requirements have been eliminated when not applicable to SDC B buildings.

C24.15 Seismic Design Requirements for Egress Stairways and Parapets

Section 24.15 includes all of the seismic design criteria for nonstructural components in Seismic Design Category B. In the general procedures of Chapter 13, all mechanical and electrical components and most architectural components in SDC B are exempt. Accordingly, Section 24.15 seismic design requirements are oriented exclusively toward egress stairways and parapets.

Additional editorial and technical simplifications have been made to the seismic design requirements for nonstructural components. The discussion below describes the technical differences between the general procedures of Chapter 13 and the alternative procedures of Section 24.15.

C24.15.2 General Design Requirements

The alternative procedure does not permit manufacturer’s certification that a component is qualified by testing or experience data; this simplification was made because it is expected that the use of this approach would be rare in SDC B. If it is desirable to
use one of these removed approaches in design of nonstructural components, Chapter 24 should not be used and the general provisions of Chapters 12 and 13 should be followed.

Additionally, the requirements related to flexibility and consequential damage were removed in the alternative procedures because they are not required for the design of egress stairways or parapets.

C24.15.3 Design of Parapets and Egress Stairways for Seismic Demands

The alternative seismic design requirements do not permit accelerations to be determined by the modal analysis procedures, as this approach is not commonly used in SDC B.

Only egress stairways are required to be designed for seismic relative displacements because design for seismic relative displacements does not affect the design of parapets.

References

Debock and Liel et al. 2013 paper on accidental torsion (in review).