Report on the 2015 NEHRP Chapter 24 Stand-Alone Seismic Design Requirements for SDC B Buildings

Prepared under the BSSC Simplified Seismic Design Procedures Development Program

February 4, 2015
1. **INTRODUCTION**

Code complexity has become an issue in recent years and the seismic provisions are no exception to this. Part of the issue is that engineers in low-seismic areas are required to read and interpret the complex seismic provisions for high-seismic areas (because the requirements in ASCE are presented all together for all Seismic Design Categories).

To attempt to help remedy this, a new Chapter 24, entitled Alternative Seismic Design Requirements for Seismic Design Category (SDC) B Buildings, was developed and was subsequently approved for the 2015 National Earthquake Hazard Reduction Program (NEHRP) Provisions. The purpose of this chapter is to provide separate seismic provisions for SDC B, such that engineers can design a building in SDC B without needing to “wade through” all of the provisions related to higher seismic design categories. The provisions and commentary text of this new Chapter 24 are provided in Appendix A of this report.

2. **PRACTITIONER TRIAL DESIGN**

2.1 **Overview**

Toward the goal of vetting and refining the provisions in this new Chapter 24, a trial design effort was initiated and four practitioners designed SDC B buildings with these new provisions. Table 1-1 outlines the four design firms and the buildings they evaluated. This report is the product of this trial design effort.
### Table 2-1 Overview of Trial Designs

<table>
<thead>
<tr>
<th>Firm Name</th>
<th>Firm Location</th>
<th>Engineering Lead</th>
<th>Trial Building Description</th>
<th>Appendix Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shaefer</td>
<td>Cincinnati, Ohio</td>
<td>Marshall Carman, PE, SE</td>
<td>Three-story, steel moment frame (R=3)</td>
<td>B.1</td>
</tr>
<tr>
<td>Summit Engineering</td>
<td>Portsmouth, NH</td>
<td>Peter Griem, PE, SE</td>
<td>Four-story, light-framed (wood) (R=6.5)</td>
<td>B.2</td>
</tr>
<tr>
<td>Martin/Martin</td>
<td>Lakewood, Colorado</td>
<td>Elizabeth Jones, PE, SE</td>
<td>Four-story, ordinary RC wall (R=4)</td>
<td>B.3</td>
</tr>
<tr>
<td>Stanley D. Lindsey &amp; Associates</td>
<td>Atlanta, GA</td>
<td>Will Jacobs, PE, SE</td>
<td>Six-story, steel braced frame (R=3)</td>
<td>B.4</td>
</tr>
</tbody>
</table>

### 2.2 Summary of Practitioner Review Comments

#### 2.2.1 General Feedback

Overall, the practitioners were excited about the prospect of simplified provisions for SDC B. One mentioned the fact that Ohio is currently considering dropping all seismic design requirements, and that such actions illustrate the need for advancing simplified seismic design procedures such as Chapter 24.

The practitioners found the Chapter 24 draft easy to use and they appreciated the consistency with ASCE 7 Chapter 12 (calling the new Chapter 24 draft “familiar”). They stated that Chapter 24 would be readily useable by both engineers that design only in SDC B and also by engineers that design in multiple SDCs (because it is easy to go back and forth between Chapters 24 and 12).

#### 2.2.2 Revisions to Table 12.2-1 (Table 24.3-1 in Chapter 24)

The practitioners recommended that Table 12.2-1 (Table 24.3-1 in Chapter 24) be further simplified by removing additional system from the table which are not traditionally used in SDC B. Specific recommendations are provided in the related sections of Appendix B.

#### 2.2.3 Technical Change to use \( E_v = 0 \) in Section 12.4.3 (24.5.3)

The practitioners found this to be a useful change. The trial designs all showed that the use of \( E_v = 0 \) had no meaningful impact on any of the trial designs; one practitioner also generalized this finding by stating that he had never seen \( E_v \) meaningfully impact any design in SDC B.
2.2.4 Technical Change to Only Require Accidental Torsion if a Type 1b Horizontal Irregularity Exists (Section 12.8.4.2/24.9.4.2)

The practitioners provided a great deal of useful and constructive feedback regarding the modifications to the accidental torsion requirements.

The practitioners stated that the simplification to the accidental torsion requirements is potentially the biggest of Chapter 24, but that the current approach is ineffectual because the user is required to include accidental torsion in the preliminary analysis in order to determine whether the building has a plan torsional irregularity and to, therefore, determine if accidental torsion can be excluded in the design process. Accordingly, the user does not get much simplification benefit from this current approach and the practitioners strongly suggested additional simplification for checking the plan torsional irregularity.

The quantitative impacts of this accidental torsion simplification were also testing in the trial designs. Even though two of the four trial buildings had a torsional irregularity ratio of nearly 1.4 (on the verge of an extreme plan torsional irregularity), the removal of the torsion check only changed the component design forces by 2-14% for the examples that were tested. Based on the four trial designs, and the associated discussions among the practitioners, the consensus was that the removal of accidental torsion did not have any significant impact on design in SDC B.

2.2.5 Technical and Editorial Simplifications in Chapter 13 (Section 24.15)

The practitioners noted that many SDC B designers either completely ignore the Chapter 13 non-structural design provisions or they apply the requirements too broadly. Therefore, they stated that the simplified and shortened Chapter 13 content is a useful change and should help practitioners appropriately apply the design requirements.

2.2.6 Summary of Feedback on Secondary Items in Chapter 24

During the practitioner trial designs, it was also requested that they review a list of secondary changes made when writing Chapter 24. This highly-detailed feedback is included in the associated sections of Appendix Band is not repeated in this section.
2.2.7 Additional Feedback on Possible Further Simplifications to Chapter 24

Practitioner feedback was also requested regarding if further simplification should be considered for Chapter 24, which were not considered in the initial version of the document. The following four additional simplifications were suggested:

- Remove the modal analysis procedure.
- Remove the two-stage design procedure.
- Work to simplify the stability checks (with the understanding that this is not a simple issue).
- Simplify how diaphragm flexibility is handled.

3. Conclusions and Recommendations

3.1 Overview

The trial designs provided in Appendix B demonstrate the usefulness of a stand-alone seismic design provision from SDC B buildings. Even so, the ASCE 7 committee has decided against creating separate stand-alone provisions, partly due to issues of ongoing maintenance of the building code provisions. Based on this, Section 3.2 summarizes a possible path-forward to improve the 2015 NEHRP Chapter 24 and Section 3.3 suggests a possible path-forward for proposing SDC-B-simplification related changes directly in Chapters 12 and 13 of ASCE 7. It is suggested that future efforts be focused on the later.

3.2 Recommendations for Improvements to Chapter 24 in the NEHRP Provisions

Based on the feedback from the practitioners doing the trial designs, the following would be the recommended next steps for improving Chapter 24 of the 2015 NEHRP document.

- Improve the simplified accidental torsion approach so that accidental torsion need not be included when checking for plan torsional irregularity (which currently dictates if torsion can be excluded in the design process).
- Further simplify Table 12.2-1.
- Further simplify by removing the modal analysis procedure and the two-stage design procedure.

3.3 Recommendations for Changes Directly to ASCE 7 Chapters 12 and 13

The Chapter 24 simplified SDC B design requirements has been approved for inclusion in the 2015 NEHRP Provisions but was not approved by the
ASCE7 Seismic Subcommittee for inclusion in ASCE 7-16. Accordingly, the following are the recommended next steps for proposing changes to ASCE 7 Chapters 12-13, with the goal of simplifying seismic design in SDC B.

- Determine how to bring the technical changes from Chapter 24 (of the 2015 NEHRP Provisions) into ASCE7 Chapter 12.
- Further improve the technical change related to accidental torsion (per Section 6.2.4 summary).
- Evaluate if there is any way to simplify or restructure ASCE7 Chapter 13, with the goal of achieving the “clear SDC B standard” that was provided in Section 24.15 of Chapter 24 (of the 2015 NEHRP Provisions).
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Chapter 24

ALTERNATIVE SEISMIC DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORY B BUILDINGS

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 egress 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 in this chapter.

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 12.7.2, 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 \( F_p \) induced by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure with elements having a 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 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.

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, $\Omega_0$, 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, $\Omega_0$, 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 $\Omega_0$ 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 $\Omega_0$ Values for Vertical Combinations

Where a structure has a vertical 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 ($R$, $\Omega_0$, 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 ($R$, $\Omega_0$, 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 ($R$, $\Omega_0$, 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 portion 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.

24.3.3.3 $R$, $C_d$, and $\Omega_0$ 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 overstrength factor, $\Omega_0$, 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 proportion 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 designed 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 calculated 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 untopped steel decking or wood structural panels are permitted to be idealized as flexible if any of the following conditions exist:

a. In structures where the vertical elements are steel braced frames, steel and concrete composite braced frames or concrete, masonry, steel, or steel and concrete composite shear walls.

b. In one- and two-family dwellings.

c. In structures of light-frame construction where all of the following conditions are met:
   1. Topping of concrete or similar materials is not placed over wood structural panel diaphragms except for nonstructural topping no greater than 1 1/2 in. (38 mm) thick.
   2. Each line of vertical elements of the seismic force-resisting system complies with the allowable story drift of Table 24.13-1.

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, \( h_n \).

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
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:
\[ E = Q_E \]  \hspace{1cm} (24.5-1)

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

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 \( L_o \) 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 \( E \). 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. \( 1.0D + H + F + 0.525Q_E + 0.75L + 0.75(L_o, \text{ or } S \text{ or } R) \)
8. \( 0.6D + 0.7Q_E + H \)

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:

\[ E_m = \Omega_o Q_E \]  \hspace{1cm} (24.5-2)

where
\[ E_m = \text{seismic load effect including overstrength factor} \]
\[ Q_E = \text{effects of horizontal seismic forces from } V, \text{ or } F_p, \text{ as specified in Sections } 24.9.1, 24.11, \text{ or } 24.15.3.1. \]
\[ \Omega_o = \text{overstrength factor} \]

24.5.3.1 Load Combinations with Overstrength Factor
Where the seismic load effect with overstrength factor, \( E_m \), 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. \(1.2D + \Omega_o Q_e + L + 0.2S\)
7. \(0.9D + \Omega_o Q_e + 1.6H\)

**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 (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 \(E\). 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. \(1.0D + H + F + 0.7\Omega_o Q_e\)
6. \(1.0D + H + F + 0.525\Omega_o Q_e + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)\)
8. \(0.6D + 0.7\Omega_o Q_e + H\)

**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-I 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:

$$V = C_s W$$  \hspace{1cm} (24.9-1)

where $C_s$ = the seismic response coefficient determined in accordance with this section $W$ = the effective seismic weight per Section 24.8.2

The seismic response coefficient, $C_s$, shall be determined in accordance with Eq. 24.9-2.

$$C_s = \frac{SDS}{(R/I_e)}$$  \hspace{1cm} (24.9-2)

where

$SDS$ = 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 Table 1.5-2 Section 11.5.1

The value of $C_s$ computed in accordance with Eq. 24.9-2 need not exceed the following:

$$C_s = \frac{S_{DI}}{T(R/I_e)}$$  \hspace{1cm} (24.9-3)

$C_s$ shall not be less than

$$C_s = 0.044S_{DI}I_e \geq 0.01$$  \hspace{1cm} (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 $1.6T_\mu$, where $T_\mu$ 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, $T_\mu$, 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:

\[
T_a = C h_n^x \tag{24.9-5}
\]

where \( h_n \) is the structural height as defined in Section 11.2 and the coefficients \( C \) 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:

\[
F_x = C_{vx}V \tag{24.9-6}
\]

and

\[
C_{vx} = \frac{w_i h_n^k}{\sum w_j h_j^k} \tag{24.9-7}
\]

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 \( (V_x) \) (kip or kN) shall be determined from the following equation:

\[
V_x = \sum_{i=1}^{n} F_i \tag{24.9-8}
\]

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

The seismic design story shear \( (V_x) \) (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 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 \( (M_{t_a}) \) 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 \( (M_{t_a}) \) 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 \( x \) \( (\delta_e) \) (in. or mm) used to compute the design story drift, \( \Delta \), shall be determined in accordance with the following equation:

\[
\delta_x = \frac{C_d \delta_e}{I_e}
\]

where

- \( C_d \) = the deflection amplification factor in Table 24.3-1
- \( \delta_e \) = 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, \( (\delta_e) \), 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 \( (\theta) \) as determined by the following equation is equal to or less than 0.10:
\[ \theta = \frac{P_x \Delta I_e}{V_x h_{xx} C_d} \]  

(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 \( V_x \)
- \( I_e \) = 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)
- \( h_{xx} \) = the story height below Level \( x \) (in. or mm)
- \( C_d \) = the deflection amplification factor in Table 24.3-1

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

\[ \theta_{\text{max}} = \frac{0.5}{\beta C_d} \leq 0.25 \]  

(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 (\( \theta \)) is greater than 0.10 but less than or equal to \( \theta_{\text{max}} \), 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 \( \theta_{\text{max}} \), 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 \( R/I_e \). The value for displacement and drift quantities shall be multiplied by the quantity \( C_d/I_e \).

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 $1.6C_uT_a$ in a given direction, $1.6C_uT_a$ shall be used in lieu of $T$ in that direction. Where the combined response for the modal base shear ($V_t$) is less than 85 percent of the calculated base shear ($V$) using the equivalent lateral force procedure, the forces shall be multiplied by $0.85\frac{V_t}{V}$.

where

$V = $ 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:

$$F_{px} = \frac{\sum_{i=1}^{n} F_i w_{px}}{\sum_{i=1}^{n} w_i}$$

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

\[
F_{px} = 0.2SDSI_{ew_{px}} \tag{24.11-2}
\]

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

\[
F_{px} = 0.4SDSI_{ew_{px}} \tag{24.11-3}
\]

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 \( F_p = 0.4SDSI_e \) 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:

\[
F_p = 0.2k_sI_eW_p \tag{24.12-1}
\]

\[
k_s = 1.0 + L_f/100 \tag{24.12-2}
\]

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

where

\( F_p \) = the design force in the individual anchors
\( I_e \) = the importance factor determined in accordance with Section 11.5.1
\( k_s \) = 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 (\(\delta_M\)). \(\delta_M\) shall be determined at critical locations with consideration for translational and torsional displacements of the structure using the following equation:

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

(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:

\[
\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2}
\]  

(24.13-2)

where \(\delta_{M1}\) 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.

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A: Full Text of NEHRP 2015 Chapter 24
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:

\[
F_p = \frac{0.4 a_p S_{DS}}{R_p} \left( 1 + \frac{2 z}{h} \right)
\]

and $F_p$ shall not be taken as less than

\[
F_p = 0.3 S_{DS} I_p W_p \tag{24.15-2}
\]

where

- $F_p$ = horizontal seismic design force applied to the parapet or egress stairway
- $S_{DS}$ = spectral acceleration, short period, as determined from Section 11.4.4
- $a_p$ = component amplification factor. $a_p$ shall be taken as 2.5 for parapets that are unbraced or braced to the structural frame below the center of mass, 1.0 for parapets braced above the center of mass, and 1.0 for egress stairways
- $I_p$ = component importance factor. $I_p$ shall be taken as 1.0 for parapets and 1.5 for egress stairways.
- $W_p$ = weight of the parapet or egress stairway
- $R_p$ = component response modification factor. $R_p$ shall be taken as 2.5.
- $z$ = height in structure of point of attachment of parapet or egress stairway with respect...
to the base of the structure. For items at or below the base, \( z \) shall be taken as 0. The value of \( z/h \) need not exceed 1.0

\[
h = \text{average roof height of structure with respect to the base of the structure.}
\]

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, \( \Omega_0 \), 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, \( D_p1 \), shall be determined in accordance with Eq. 24.15-3 as:

\[
D_p1 = D_p I_e
\]  
(24.15-3)

where

\( I_e \) = the importance factor in Section 11.5.1

\( D_p \) = displacement determined in accordance with the equations set forth in Sections 24.15.4.1 and 24.15.4.2.

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 \( h_x \) and the other at a height \( h_y \), \( D_p \) shall be determined as

\[
D_p = \delta_x - \delta_y
\]  
(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

\[
D_p = \left( \frac{(h_x - h_y) \Delta_{aA}}{h_{xx}} \right) \left( \frac{\Delta_{aA}}{h_{xx}} \right)
\]  
(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

\[
D_p = |\delta_x| + |\delta_y|
\]  
(24.15-6)

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

\[
D_p = \frac{h_x \Delta_{aA}}{h_{xx}} + \frac{h_y \Delta_{aB}}{h_{xx}}
\]  
(24.15-7)

where

\( D_p \) = relative seismic displacement that the component must be designed to accommodate

\( \delta_{x,x} \) = deflection at building Level \( x \) of Structure A, determined in accordance with
Eq. (24.9-9)
\( \delta_{yA} \) = deflection at building Level \( y \) of Structure A, determined in accordance with Eq. (24.9-9).
\( \delta_{yB} \) = deflection at building Level \( y \) of Structure B, determined in accordance with Eq. (24.9-9).
\( h_x \) = height of Level \( x \) to which upper connection point is attached
\( h_y \) = height of Level \( y \) to which lower connection point is attached
\( \Delta_{aA} \) = allowable story drift for Structure A as defined in Table 24.13-1
\( \Delta_{aB} \) = allowable story drift for Structure B as defined in Table 24.13-1
\( h_{sx} \) = story height used in the definition of the allowable drift \( \Delta_a \) in Table 24.13-1.

Note that \( \Delta/\Delta_{sx} \) = 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 stairways, 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 designed to 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 concrete or steel shall not be used for sustained tension loads. Power actuated fasteners in masonry are not permitted unless approved for seismic loading.

**EXCEPTION 1:** Power actuated fasteners in concrete used for support of acoustical tile or lay-in panel suspended ceiling applications and distributed systems where the service load on any individual fastener does not exceed 90 lb (400 N). Power actuated fasteners in steel where the service load on any individual fastener does not exceed 250 lb (1,112 N).

**EXCEPTION 2:** Power actuated fasteners in steel where the service load on any individual fastener does not exceed 250 lb. (1,112 N).
Table 24.3-1  Design Coefficients and Factors for Seismic Force-Resisting Systems

<table>
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<th>Overstrength Factor, $\Omega^c$</th>
<th>Deflection Amplification Factor, $C_d^b$</th>
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<td><strong>A. BEARING WALL SYSTEMS</strong></td>
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<td>$2\frac{1}{2}$</td>
<td>4</td>
</tr>
<tr>
<td>5. Intermediate precast shear walls$^d$</td>
<td>14.2</td>
<td>4</td>
<td>$2\frac{1}{2}$</td>
<td>4</td>
</tr>
<tr>
<td>6. Ordinary precast shear walls$^d$</td>
<td>14.2</td>
<td>3</td>
<td>$2\frac{1}{2}$</td>
<td>3</td>
</tr>
<tr>
<td>8. Intermediate reinforced masonry shear walls</td>
<td>14.4</td>
<td>$3\frac{1}{2}$</td>
<td>$2\frac{1}{2}$</td>
<td>$2\frac{1}{4}$</td>
</tr>
<tr>
<td>9. Ordinary reinforced masonry shear walls</td>
<td>14.4</td>
<td>2</td>
<td>$2\frac{1}{2}$</td>
<td>$1\frac{1}{2}$</td>
</tr>
<tr>
<td>13. Ordinary reinforced AAC masonry shear walls</td>
<td>14.4</td>
<td>2</td>
<td>$2\frac{1}{2}$</td>
<td>2</td>
</tr>
<tr>
<td>14. Ordinary plain AAC masonry shear walls</td>
<td>14.4</td>
<td>$1\frac{1}{2}$</td>
<td>$2\frac{1}{2}$</td>
<td>$1\frac{1}{2}$</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>$6\frac{1}{2}$</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>$6\frac{1}{2}$</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>$2\frac{1}{2}$</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>$3\frac{1}{2}$</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>$3\frac{1}{4}$</td>
<td>2</td>
<td>$3\frac{1}{4}$</td>
</tr>
<tr>
<td>5. Ordinary reinforced concrete shear walls$^d$</td>
<td>14.2</td>
<td>5</td>
<td>$2\frac{1}{2}$</td>
<td>$4\frac{1}{2}$</td>
</tr>
<tr>
<td>8. Intermediate precast shear walls$^d$</td>
<td>14.2</td>
<td>5</td>
<td>$2\frac{1}{2}$</td>
<td>$4\frac{1}{2}$</td>
</tr>
<tr>
<td>9. Ordinary precast shear walls$^d$</td>
<td>14.2</td>
<td>4</td>
<td>$2\frac{1}{2}$</td>
<td>4</td>
</tr>
<tr>
<td>Seismic Force-Resisting System</td>
<td>ASCE 7 Section Where Detailing Requirements Are Specified</td>
<td>Response Modification Coefficient, $R^a$</td>
<td>Overstrength Factor, $\Omega_0$</td>
<td>Deflection Amplification Factor, $C_d$</td>
</tr>
<tr>
<td>------------------------------------------------------------------</td>
<td>-----------------------------------------------------------</td>
<td>------------------------------------------</td>
<td>---------------------------------</td>
<td>---------------------------------</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>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>2½</td>
<td>4½</td>
</tr>
<tr>
<td>24. Light-frame walls with shear panels of all other materials</td>
<td>14.1 and 14.5</td>
<td>2½</td>
<td>2½</td>
<td>2½</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>4½</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>4. Steel ordinary moment frames</td>
<td>14.1</td>
<td>3½</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>4½</td>
</tr>
<tr>
<td>7. Ordinary reinforced concrete moment frames</td>
<td>14.2</td>
<td>3</td>
<td>3</td>
<td>2½</td>
</tr>
<tr>
<td>9. Steel and concrete composite intermediate moment frames</td>
<td>14.3</td>
<td>5</td>
<td>3</td>
<td>4½</td>
</tr>
<tr>
<td>10. Steel and concrete composite partially restrained moment frames [System is limited to a structural height, $h_m$, of 160 ft (48.8 m)]</td>
<td>14.3</td>
<td>6</td>
<td>3</td>
<td>5½</td>
</tr>
<tr>
<td>11. Steel and concrete composite ordinary moment frames</td>
<td>14.3</td>
<td>3</td>
<td>3</td>
<td>2½</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>2½</td>
</tr>
<tr>
<td>4. Intermediate reinforced masonry shear walls</td>
<td>14.4</td>
<td>3½</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>6. Steel and concrete composite ordinary</td>
<td>14.3</td>
<td>3½</td>
<td>2½</td>
<td>3</td>
</tr>
<tr>
<td>Seismic Force-Resisting System</td>
<td>ASCE 7 Section Where Detailing Requirements Are Specified</td>
<td>Response Modification Coefficient, $R$</td>
<td>Overstrength Factor, $\Omega_0$</td>
<td>Deflection Amplification Factor, $C_d$</td>
</tr>
<tr>
<td>-------------------------------</td>
<td>--------------------------------------------------------</td>
<td>---------------------------------</td>
<td>------------------------------</td>
<td>----------------------------------</td>
</tr>
<tr>
<td>braced frames</td>
<td>7. Steel and concrete composite ordinary shear walls</td>
<td>14.3</td>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>8. Ordinary reinforced concrete shear walls$^d$</td>
<td>14.2</td>
<td>5½</td>
<td>2½</td>
</tr>
<tr>
<td>F. SHEAR WALL-FRAME INTERACTIVE SYSTEM WITH ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS$^d$</td>
<td>24.3.5.4 and 14.2</td>
<td>4½</td>
<td>2½</td>
<td>4</td>
</tr>
<tr>
<td>G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR [System is limited to a structural height, $h_n$, of 35 ft (10.7 m)];</td>
<td>24.3.5.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2. Steel ordinary cantilever column systems</td>
<td>14.1</td>
<td>1¼</td>
<td>1¼</td>
</tr>
<tr>
<td></td>
<td>4. Intermediate reinforced concrete moment frames</td>
<td>14.2</td>
<td>1½</td>
<td>1¼</td>
</tr>
<tr>
<td></td>
<td>5. Ordinary reinforced concrete moment frames</td>
<td>14.2</td>
<td>1</td>
<td>1¼</td>
</tr>
<tr>
<td></td>
<td>6. Timber frames</td>
<td>14.5</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>

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

$^d$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.

$^m$In Section 2.2 of ACI 318. A shear wall is defined as a structural wall.
See FIGURE 12.3-1 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>4.</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>5.</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><strong>In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity:</strong> 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><strong>Discontinuity in Lateral Strength–Extreme Weak Story Irregularity:</strong> 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>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 where subjected to seismic forces:</td>
<td></td>
<td></td>
</tr>
<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.
See FIGURE 12.8-2 Story Drift Determination

See FIGURE 12.10-1 Collectors

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

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

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

$^b$There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. 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.
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 $S_{DYN}$ 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 \( E_v = 0.2SDS \) 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, \( \rho \), 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, $E_{su}$, 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 $C_uT_a$, but in these alternative procedures, for simplicity, $C_u$ is taken as a constant value of 1.6. This 1.6 value is used because the Chapter 12 $C_u$ 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.
C24.9.4.2 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.

C24.13 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 Seismic Design Force

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.

C24.15.4 Design of Egress Stairways for Seismic Relative Displacements

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

Appendix B

Trial Design Reports

B.1 Trial Design by Schaefer Engineering

B.1.1 Overview of Trial Design Team

This trial design was completed by Marshall Carman, PE, SE, with Shaefer Engineering in Cincinnati, Ohio.

B.1.2 Overview of Example Building(s) used as Basis of the Trial Design

The proposed NEHRP 2015 Chapter 24 provisions were applied to a three-story, 45ft tall steel moment frame structure. The building is a risk category III structure, with a seismic site class of C. The structure was originally designed/analyzed according IBC 2009/ASCE 7-05 requirements, and then re-analyzed using IBC 2012/ASCE7-10 to provide a comparison to the proposed NEHRP Provisions.

The floor framing is typical composite deck supported on steel floor framing, which was analyzed as a rigid diaphragm. The 3” deep roof deck is supported on steel framing, which was analyzed as a semi-rigid diaphragm.

Reference tables and figures are provided at the end of this chapter, to more fully document the trial building and the design checks.

B.1.3 General Review Comments for the New Chapter 24 Document

The document currently includes too many provisions and systems to serve the target audience. It is my understanding that the traditional provisions for SDC B will still exist, and that this chapter will serve as a pared down standalone chapter. As such, an engineer more adept at seismic design would still have additional provisions available should the design warrant their use, but the vast majority of projects in SDC B would not require them.

I recommend the following:

1) Removal of section 24.3.3.2 – Two Stage Analysis Procedure
2) Review/Modification of section 24.4 – Diaphragm flexibility and configuration irregularities. Currently diaphragms can only be idealized as rigid for buildings with no horizontal irregularities. Can this requirement be reviewed/simplified?

   a. In this trial design, there are non-orthogonal frames which would result in a horizontal irregularity that would require the diaphragm stiffness to be considered. However, given the size, aspect ratio of the floor plate, and the relative stiffness of the moment frames, this seems unnecessary.

   b. The provisions do not require consideration of accidental torsion for a non-severe torsional irregularity. However, it is still classified as a horizontal irregularity, which also would trigger modeling the stiffness of the diaphragm.

3) Removal of section 24.10 - Modal Response Spectrum Analysis and related sections (24.7, portions of 24.8.3, 24.14.4, etc.). I think it is reasonable to have engineers use the standard SDC B provisions if performing a Modal Response Spectrum Analysis.


5) Elimination if possible of egress stairway requirements for seismic relative displacements, OR replace with more prescriptive requirements.

B.1.4 Specific Review Comments for Primary Items in Chapter 24

B.1.4.1 Revisions to Table 12.2-1(Table 24.3-1 in Chapter 24)

There are probably several systems that could still be removed from the table. It would be unusual to use AISC 341 provisions in SDC B. I would consider removing the following systems:

- B.3 – Steel ordinary concentrically braced frames
- C.3 – Steel intermediate moment frames
- C.4 – Steel ordinary moment frames
- C.9 – Steel and concrete composite intermediate moment frames
- C.10 – Steel and concrete composite partially restrained moment frames
- C.11 – Steel and concrete composite ordinary moment frames
• E in its entirety

**B.1.4.2 Editorial Changes to Irregularity Section 12.3.2 (Section 24.4.2 in Chapter 24) and Tables 12.3-1 and 12.3-2 (Tables 24.4-1 and 24.4-2)**

I have no objection to this change.

**B.1.4.3 Technical Change to use Ev = 0 in Sections 12.4.2-12.4.3 (24.5.2-24.5.3)**

I have no objection to this change. It’s been my experience that \((1.2 + 0.2SDS) + 0.5LL\) is less than \(1.4DL\) or \(1.2DL + 1.6LL\). On this project it had no impact.

**B.1.4.4 Technical Change to Only Require Accidental Torsion if a Type 1b Horizontal Irregularity Exists (Section 12.8.4.2/24.9.4.2)**

This is probably the most significant technical change. In this sample project the structure was right at the limit for a type 1b horizontal irregularity. The worst maximum drift to average drift ratio as 1.41, and for the purpose of this sample it was assumed to be 1.39. It resulted in frame shears that were typically between 92% and 98% of the frame shears in the ASCE 7-10 analysis considering 5% accidental torsion. This is despite the fact that the base shear in the NEHRP 24 analysis was approximately 6.5% higher due to setting \(Cu\) to 1.6.

In general, this simplified could impact moment frame sizes. In this case, the sizes were governed by an \(H/400\) drift limit for a 10-year wind interval. However, if seismic drifts governed, the additional 6.5% base shear increase due to \(Cu\) would not be a factor if the engineer uses the calculated period for drift and did not use \(Cu\) to cap the period. The largest seismic inelastic drift with ASCE 7-10 considering accidental torsional was approximately 2.7”. The largest inelastic drift using the propose NEHRP 24 provisions with no accidental torsion was approximately 1.9”, or approximately 70% of the drift calculated using ASCE 7-10.

Table 24.4.1 still requires evaluating a torsional irregularity assuming accidental torsion exists. Therefore the engineer still needs to create the accidental torsion load cases in order to determine if accidental torsion is required. While there is arguably some benefit to being able reduce the number of load cases after the evaluation, it is limited. It would save significant effort if this could be evaluated without creating the accidental torsion in the first place. If the goal is to identify “cruciform” buildings, could the trigger for accidental torsion be related to a prescriptive comparison between the center of mass and perimeter frame and the center of mass and perimeter of the building?
There is potential for further improvements in this section and in ASCE 7.

- Torsional irregularity is determined using “maximum story drifts”, but Ax is required it is based upon “maximum displacements”. I am not sure if there is a technical basis for this, but it would be clearer to the engineer if the same parameters were used both cases.

- I don’t understand the concept of using average displacements vs maximum displacements to determine if there is a torsional irregularity in the first place. It punishes rigid buildings with small average displacements more than flexible buildings. Is there a better way to do this that looks at the rotational behavior of the structure, and identifies where rotational modes may dominate?

B.1.4.5 Technical and Editorial Simplifications in Chapter 13 (Section 24.15)

I would support removing chapter 13 requirements for architectural components with the exception of parapets. Chimneys have been toppled in previous seismic events in Ohio.

Regarding the reference text below (in the documentation), I don’t understand the items listed in the second bullet point above. Fire sprinklers are currently located in the Mechanical and Electrical Components section of ASCE 7-10 (13.6) as a piping system (13.6.8.2). As such, I have previously treated them as being exempt in SDC B already with all other Mechanical and Electrical Components already.

Reference Text: “Chapter 13 could be completely removed if not for two non-exempt components types: (1) parapets supported by bearing walls or shear walls and (2) Architectural components with Ip > 1.0 (ref. 13.1.4 item 3) (e.g. fire sprinklers, egress stairways, components with hazardous materials).”

Requirements for egress stairways are a recent addition from ASCE 7-05 to ASCE 7-10, and I don’t know if they need to be considered in SDC B. These provisions haven’t been adopted yet in this region, but represent a significant technical change.

In Ohio, Indiana, and portions of Kentucky, the seismic design category would often be A if considering $S_{DS}$ only. In these areas, is damage to nonstructural components less likely given that $S_{DS}$ values are so low? This is contrary to the statement above, noting previous damage to URM parapets and chimneys, but is there a reasonable limit to $S_{DS}$ values where chapter 13 is not required?
B.1.5 Specific Review Comments for Secondary Items in Chapter 24

B.1.5.1 Removal of Plastic Analysis Exception in Section 12.4.3.1 (24.5.3)

I have no objection to removing this in the simplified analysis. There are only a few elements that require consideration of over strength in SDC B.

B.1.5.2 Removal of Chapter 19 Foundation Modeling Option in Section 12.7.1 (24.8.1) and Associated Removal of Sections 12.8.1.2 and 12.9.7

I believe it would be unusual to try to increase period/reduce base shear by considering foundation flexibility in SDC B. I have no objection to removing this section.

B.1.5.3 Streamlined $C_s$ Calculation in Section 12.8.1.1 (24.9.1)

In concept I have no objection to removing $C_s$ based upon long periods. I assume it is less likely in areas of low-moderate seismicity for tall long period buildings to have seismic base shears which are greater than the base shear due to wind.

B.1.5.4 Streamlined Approximate Fundamental Period Determination in Section 12.8.2.1 (24.9.2.1)

Commercial software packages, or internal spreadsheets, automatically calculate $C_u$. Therefore, having a separate, but “simpler” assumption of $C_u=1.6$ would require a minor modification by the user or by software developers. This requires one additional step by the engineer to review and manually modify the building period.

In the case of the moment frame trial design example, increased base shear due to a lower $C_u$ partially offset some of the lower frame forces due to not considering accidental torsion. Perhaps this is an unintended, but worthwhile, impact.

B.1.5.5 Removal of the Simplified Design Procedure of Section 12.4

I have never used the simplified design procedure, and am not familiar with other engineers using this procedure. I believe the proposed approach should replace the simplified approach.

B.1.6 Summary

In general, I believe the chapter accomplishes its goal reducing the apparent complexities of seismic design by removing the sections that are not applicable to SDC B. I do not believe the technical changes regarding accidental torsion or providing a constant $C_u$ of 1.6 provide significant enough value to justify having and maintaining separate provisions from the
standard chapter 12 requirements, and in some instances may add some confusion.

### Table B.1-2  Frame Story Shear Comparison between NEHRP 24 simplified Analysis ($Cu=1.6$, No Accidental Torsion) and ASCE 7-10 Analysis ($Cu=1.7$ with 5% Accidental Torsion) for Frames Primarily Oriented in X-Direction.

<table>
<thead>
<tr>
<th>Frame</th>
<th>Primary Frame Story Shears (EQ-x) kips</th>
<th>Difference %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>NEHRP 24 (E1)</td>
<td>ASCE 7-10 (MAX)</td>
</tr>
<tr>
<td></td>
<td>X</td>
<td>Y</td>
</tr>
<tr>
<td>Frame 0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Story3</td>
<td>18.7</td>
<td>-0.1</td>
</tr>
<tr>
<td>Story2</td>
<td>44.6</td>
<td>0.1</td>
</tr>
<tr>
<td>Story1</td>
<td>51.5</td>
<td>-0.1</td>
</tr>
<tr>
<td>Frame 4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Story3</td>
<td>19.1</td>
<td>-0.1</td>
</tr>
<tr>
<td>Story2</td>
<td>44.2</td>
<td>0.2</td>
</tr>
<tr>
<td>Story1</td>
<td>51.9</td>
<td>-0.1</td>
</tr>
</tbody>
</table>
### Table B.1-3  Frame Story Shear Comparison between NEHRP 24 simplified Analysis (Cu=1.6, No Accidental Torsion) and ASCE 7-10 Analysis (Cu=1.7 with 5% Accidental Torsion) for Frames Primarily Oriented in Y-Direction.

<table>
<thead>
<tr>
<th>Frame</th>
<th>Primary Frame Story Shears (EQ-Y) Kips</th>
<th>Difference %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>NEHRP 24 (E2)</td>
<td>ASCE 7-10 (MAX)</td>
</tr>
<tr>
<td>Frame 1</td>
<td>X</td>
<td>Y</td>
</tr>
<tr>
<td>Story3</td>
<td>-0.1</td>
<td>16.1</td>
</tr>
<tr>
<td>Story2</td>
<td>0.2</td>
<td>38.5</td>
</tr>
<tr>
<td>Story1</td>
<td>-0.4</td>
<td>38.6</td>
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<tr>
<td>Frame 2</td>
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<td>Y</td>
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<tr>
<td>Story3</td>
<td>-8.4</td>
<td>13.2</td>
</tr>
<tr>
<td>Story2</td>
<td>-19.7</td>
<td>34.3</td>
</tr>
<tr>
<td>Story1</td>
<td>-15.2</td>
<td>34.9</td>
</tr>
<tr>
<td>Frame 3</td>
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<td>Y</td>
</tr>
<tr>
<td>Story3</td>
<td>8.5</td>
<td>13.5</td>
</tr>
<tr>
<td>Story2</td>
<td>19.8</td>
<td>34.1</td>
</tr>
<tr>
<td>Story1</td>
<td>15.3</td>
<td>35.0</td>
</tr>
</tbody>
</table>

### Table B.1-4  Corner Displacement Comparison between NEHRP 24 simplified Analysis (No Accidental Torsion) and ASCE 7-10 Analysis (5% Accidental Torsion)

<table>
<thead>
<tr>
<th>Corner</th>
<th>Maximum Inelastic Drift at Roof (EQ-X) in</th>
<th>Difference %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>NEHRP 24 (E1)</td>
<td>ASCE 7-10 (MAX)</td>
</tr>
<tr>
<td>NE Corner</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Story3</td>
<td>1.93</td>
<td>2.71</td>
</tr>
<tr>
<td>Story2</td>
<td>1.52</td>
<td>2.13</td>
</tr>
<tr>
<td>Story1</td>
<td>0.79</td>
<td>1.10</td>
</tr>
<tr>
<td>SE Corner</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Story3</td>
<td>1.94</td>
<td>2.73</td>
</tr>
<tr>
<td>Story2</td>
<td>1.54</td>
<td>2.15</td>
</tr>
<tr>
<td>Story1</td>
<td>0.81</td>
<td>1.13</td>
</tr>
</tbody>
</table>
Figure B.1-1  Moment Frame Layout
B.2.1 Trial Design by Summit Engineering

B.2.1 Overview of Trial Design Team

This trial design was completed by Peter Griem, a senior structural engineer with Summit Engineering in Portsmouth, NH. Mr. Griem serves on the NCSEA Seismic Code Advisory Committee and the FEMA-sponsored BSSC Code Resource Support Committee. Summit Engineering typically provides designs for small- to moderate-sized structural steel, wood, and cold-formed steel structures throughout the northeast and the US Virgin Islands.

B.2.2 Overview of Example Building(s) used as Basis of the Trial Design

The example building is located in northern New Hampshire on Site Class C soil. It is a four-story, wood-framed condominium structure with a walkout basement. The primary ridge height is 69'-8" from finished grade on the low side. The seismic force-resisting system consists of bearing walls with light-frame (wood) walls sheathed with wood structural panels rated for shear resistance. Floors are topped with ¾” non-structural gypcrete placed over an acoustic mat. A model of the structure is shown in Figure B.2-1.
In the actual design, most of the maximum design loads were governed by wind load combinations. For the purpose of this example, a relatively straight-forward, non-perforated shear wall in the middle of the floor plan was selected to explore the impacts of the differences between Chapters 12 and 24 (highlighted in red in Figure B.2-1).

### B.2.3 General Review Comments for the New Chapter 24 Document

I generally found Chapter 24 to be familiar, easy to follow, and broadly applicable. Like most practitioners in the Northeastern United States, I usually design for Seismic Design Categories (SDC) B and C; and occasionally for SDC D. Chapter 24 does not introduce new equations, and it closely follows the layout, format, and language of Chapter 12, making it easier to go back and forth between the two. I appreciated not having to check a series of threshold limitations. Since the chapter is not limited to certain Risk Categories, threshold height limits, or structural systems – it is truly applicable to just about any design I can envision working on in Seismic Design Category B. And for the infrequent projects that others may find themselves working on that aren’t applicable to Chapter 24, you can always use Chapter 12 which is largely the same. Editorial deletions of provisions that do not apply in low seismic regions make the chapter easier to follow.

From a future development standpoint, I have mixed feelings about whether Chapter 24 should be administratively linked to mirror sections in Chapter 12. Linked language is easier to maintain in the standard, and makes it easier for the practitioner to design between categories, but it may hinder future efforts to distinguish justifiably less stringent provisions in SDC B. NEHRP/ASCE should strive to make the language as mirrored as possible, but not consider it an administrative requirement. It may be helpful to identify when one section is intentionally different than its parent section, perhaps by use of margin notes similar to those used to designate changes from a previous version.

There appear to be several opportunities to study further simplifications in future development cycles. Ideas include:

- Section 24.9.7: P-Delta Effects – Perhaps the stability coefficient check can be eliminated below a reasonable percentage of the allowable story drifts, or an exception could be made for relatively stiff systems below a height threshold.
• Section 24.10: Modal Response Spectrum Analysis – For the sake of brevity in Chapter 24, there may be some benefit in limiting the analysis method to Equivalent Lateral Force. I have seen engineers in the Northeast use Modal Analysis, but not often. Chapter 12 still allows for its use.

• Section 24.4.1: Diaphragm Flexibility – Changes to alternatives allowed for the idealization of flexible diaphragms in the 2012 IBC/2010 ASCE 7 code cycle arguably eliminated an opportunity for engineers to classify some untopped steel deck diaphragms as flexible. It would be beneficial to determine whether some semi-rigid analyses are necessary in low seismic regions, and/or whether it would be justifiable to limit required seismic torsional checks in SDC B.

The elimination of two-stage analysis provision was suggested. I believe that the two-stage analysis is a useful tool, especially in regions such as the eastern United States where relatively stiff construction such as partial basements, or commercial/retail platform construction using concrete or masonry shear walls on the lowest floor are common.

B.2.4 Specific Review Comments for Primary Items in Chapter 24

B.2.4.1 Revisions to Table 12.2-1 (Table 24.3-1 in Chapter 24)

Revisions to Table 24.3 had no effect on this trial design. The seismic force resisting system for the example (Bearing Wall System – Light Frame (Wood) Walls Sheathed with Wood Structural Panels Rated for Shear Resistance) is listed in the table, and there are no changes relative to Chapter 12 Provisions.

In general, it seems any system that is only allowed in SDC B should be listed in Table 24.3. Also, systems that have “ordinary” or “plain” levels of detailing should be listed in the table. Finally, systems that are “not specifically detailed for seismic resistance” should also be included in Table 24.3.

I have never personally seen or heard of a project that utilizes composite-type systems. These systems may be more common in more temperate climates, but if not it seems they could be eliminated from Table 24.3 for the sake of brevity. Also, any systems that have “intermediate” or “special” levels of detailing should be eliminated from the table as they are not common in SDC B.
Specifically, systems that should have remained in Table 24.3 include:

- A - Bearing Wall Systems: Detailed and Ordinary Plain Concrete Shear Walls, Detailed and Ordinary Masonry Shear Walls, Prestressed Masonry Shear Walls
- B - Building Frame systems: Detailed and Ordinary Plain Concrete Shear Walls, Detailed and Ordinary Masonry Shear Walls, Prestressed Masonry Shear Walls

Systems that should be eliminated from Table 24.3 include:

- B - Building Frame Systems: Intermediate Precast Shear Walls, Intermediate Masonry Shear Walls
- C - Moment Resisting Frame Systems: Steel Intermediate Moment Frames, Intermediate Reinforced Concrete Moment Frames, Steel and Concrete Composite Intermediate Moment Frames, Steel and Concrete Composite Partially Restrained Moment Frames, Steel and Concrete Composite Ordinary Moment Frames
- E - Dual Systems with Intermediate Moment Frames Capable of Resisting At Least 25% of Prescribed Seismic Forces: Intermediate Reinforced Masonry Shear Walls, Steel and Concrete Composite Ordinary Braced Frames, Steel and Concrete Composite Ordinary Shear Walls
- G - Cantilevered Column Systems Detailed to Conform to the Requirements for Intermediate Reinforced Concrete Moment Frames

**B.2.4.2 Editorial Changes to Irregularity Section 12.3.2 (Section 24.4.2 in Chapter 24) and Tables 12.3-1 and 12.3-2 (Tables 24.4-1 and 24.4-2)**

Because the floor assembly for the example structure consists of a plywood deck topped with less than 1-1/2” of cementitious material, it can be idealized as a flexible and Type 1a and 1b horizontal irregularities do not apply.

In general, torsional irregularities are problematic for very rigid buildings because they are dependent on drift ratios and there are no minimum thresholds. In SDC B where equivalent loads and/or deflection amplification factors are relatively low, the problem may extend into more flexible structural system types. It appears there is an opportunity to perform a more comprehensive study on the need for at least Type 1a Torsional Irregularities in SDC B.
B.2.4.3  Technical Change to use $E_v = 0$ in Section 12.4.3 (24.5.3)

In the shear wall studied for the example problem the elimination of the seismic vertical effects had very little effect on the results. The value for $0.2SD_s$ for this example works out to be 0.057 – less than 6% of the dead load effect. The maximum compressive shear wall chord force was calculated to be 10.4 kips with the vertical seismic effect considered, and 10.3 kips without. There was no net uplift on the shear wall in either case (minimum compressive load 4.1 kips considering vertical effect, 5.4 kips without). It is likely that there would be cases where small net uplifts are eliminated by not considering the vertical effects, however, I question whether the results of the seismic analysis are even accurate enough to worry about small differences.

The change is useful. It simplifies load combinations, thus reducing effort with negligible difference in analysis results.

B.2.4.4  Technical Change to Only Require Accidental Torsion if a Type 1b Horizontal Irregularity Exists (Section 12.8.4.2/24.9.4.2)

Since the diaphragms in the example problem are idealized as flexible, this change has no effect on the building type.

Section 24.9.4.2 (12.8.4.2) adds an exception that allows you to ignore the accidental torsional moments for non-Type 1b Horizontal Irregular buildings, but the charging language requires you to include them in your analysis to determine if an irregularity exists in the first place. From simplification standpoint, the technical change does not accomplish much. There may be some economic benefit to eliminating accidental torsion from the final iteration of an analysis, but I suspect it would be minor. The exception is not written in mandatory language – i.e. the engineer may eliminate accidental torsion, but does not have to, so the clause is only potentially helpful.

B.2.4.5  Technical and Editorial Simplifications in Chapter 13 (Section 24.15)

No comments for this topic.

B.2.5  Specific Review Comments for Secondary Items in Chapter 24

B.2.5.1  Removal of Plastic Analysis Exception in Section 12.4.3.1 (24.5.3)

The removal of the Plastic Analysis Exception is appropriate for the alternative Chapter 24. The practitioner has the option to use Chapter 12.
**B.2.5.2 Removal of Chapter 19 Foundation Modeling Option in Section 12.7.1 (24.8.1) and Associated Removal of Sections 12.8.1.2 and 12.9.7**

The removal of the foundation modeling option and soil-structure interaction reductions is appropriate for the alternative chapter. The practitioner has the option to use Chapter 12.

**B.2.5.3 Streamlined $C_s$ Calculation in Section 12.8.1.1 (24.9.1)**

The removal of the long period transition provisions makes sense for the vast majority of practitioners in SDC B. It had no effect on this trial design because the period used for design was 0.43 seconds, well below the mapped $T_1$ of 6 seconds. The calculation probably only affects very tall buildings (e.g. 500 feet or taller). Since Table 24.3-1 does not limit heights, it would be advisable to limit the applicability of Chapter 24 to buildings under 500 feet tall. This would be important for large cities typically in SDC B such as Boston and New York City. Note: this may not be an issue if the seismic force-resisting systems tabulated in Table 24.3-1 are not typical for very tall buildings.

**B.2.5.4 Streamlined Approximate Fundamental Period Determination in Section 12.8.2.1 (24.9.2.1)**

Since this trial design problem is a wood-framed building, the elimination of the alternate methods for calculating approximate fundamental periods had no effect on the design. In general, the elimination of the alternate methods is good for simplicity and brevity. The practitioner has the option to design per Chapter 12.

**B.2.5.5 Removal of the Simplified Design Procedure of Section 12.4**

The Chapter 24 Alternative Seismic Design Requirements are consistent with the baseline requirements developed in Chapter 12, thus they promote more consistency in design, and allow for more thoughtful discussion about reasonable differences for low seismic regions. The simplified method was often not useful due to its limitations, and I don’t anticipate that anyone will miss it.

**B.2.6 Summary**

An analysis of a four/five story wood frame structure with flexible diaphragms showed that the proposed differences between Chapters 12 and 24 had little to no impact on the design of the structure, but were advantageous to the designer for ease of use. There appear to be a few opportunities to further simplify Chapter 24 in future cycles with additional research, but the new Chapter is easy to follow, consistent with the layout of
provisions for moderate and high seismic categories, and applicable to a much broader scope of buildings than previously published simplified methods have been.
B.3.1 **Trial Design by Martin/Martin Inc.**

B.3.1 **Overview of Trial Design Team**

The study was led by Elizabeth A. Jones, PE, SE, Principal at Martin/Martin, Inc. She was assisted by Adam Boswell, PE, the original design team Project Engineer and Tamara Worker, PE, Project Engineer.

B.3.2 **Overview of Example Building(s) used as Basis of the Trial Design**

The example building is a residential and dining hall located at Colorado School of Mines, in Golden, Colorado, a suburb of Denver. This Occupancy Category III (referred to as Risk Category in ASCE 7-10) project includes two, four-story buildings connected by a bridge over a one-level common base, which is partially buried. An expansion joint occurs at one end of the bridge. The concrete structure consists of two-way, flat plates supported on concrete columns with drilled pier foundations. The lateral system consists of ordinary reinforced concrete shear walls (bearing), with an R value of 4. The $S_{DS}$ for the site is 19%. For comparison, the wind speed is 120 mph with Exposure C, based on ASCE 7-05 (and approximately 170 mph, using ASCE 7-10 strength/risk level design.). Seismic controls the lateral design over wind, due to the mass and shape of the building.

B.3.3 **General Review Comments for the New Chapter 24 Document**

By having the Appendix Chapter 24 that only addresses the requirements for Seismic Design Category B, the lateral design process is greatly simplified, particularly for someone not experienced with the use of ASCE 7 for seismic design. This would include those new to the practice, as well as those engineers who live in areas where wind typically controls the design. It is also helpful for those who often design buildings in multiple design categories. The simplification primarily occurs by having the paragraphs deleted that do not apply, as well as some formulas and tables simplified since only Category B has to be covered.

I recommend that the section for modal response spectrum analysis, part 24.10, be removed. I believe it is very rare to do this analysis for Seismic Design Category B. If an engineer were using modal response spectrum analysis, he or she should be able to easily navigate Chapter 12 of the main portion of ASCE 7-10.
B.3.4 Specific Review Comments for Primary Items in Chapter 24

B.3.4.1 Revisions to Table 12.2-1 (Table 24.3-1 in Chapter 24)

I agree with the systems which were retained in the list and those that were deleted.

B.3.4.2 Editorial Changes to Irregularity Section 12.3.2 (Section 24.4.2 in Chapter 24) and Tables 12.3-1 and 12.3-2 (Tables 24.4-1 and 24.4-2)

For Horizontal and Vertical Irregularities, there are no technical changes, just simplifications to not include the items that do not apply to SDC B. This is helpful.

B.3.4.3 Technical Change to use $Ev = 0$ in Sections 12.4.2-12.4.3 (24.5.2-24.5.3)

I agree with the simplification to not include the effect of a vertical earthquake component. The load combinations in sections 12.4.2 and 12.4.3 are cumbersome enough with the different factors on $S_{DS}$ for vertical EQ as well as all the footnotes. The current ASCE 7 already allows an exception to neglect vertical earthquake component when checking soil-structure interface, per section 12.4.2.2.

In our example building, the reactions to the drilled piers supporting the shear walls increased 1 to 2% with the inclusion of the vertical earthquake effect. This had no effect on the design of the piers, as dead plus live load combination controlled the design for maximum downward load. There was a 2% to 6% increase in uplift at a pier, which did not control the reinforcing needed in the pier. As noted before, the exception for soil-structure interface already in the code meant that pier end bearing and skin friction did not have to consider vertical earthquake effect.

B.3.4.4 Technical Change to Only Require Accidental Torsion if a Type 1b Horizontal Irregularity Exists (Section 12.8.4.2/24.9.4.2)

I do not believe this change is helpful. The engineer still must perform diaphragm analysis that includes the effects of accidental torsion in order to determine whether torsional irregularities occur. If the extreme torsion irregularity (1b) does not exist, engineer may delete the consideration of accidental torsion. This may actually cause additional effort, rather than less effort. If the check for torsional irregularity could be determined without the analysis for accidental torsion, then there may be the ability to gain some simplification. However, for a rigid diaphragm, since the engineer must consider rigidities of the lateral resisting elements and consider the natural torsion created by distance between center of rigidity and center of mass,
some sort of diaphragm analysis must be performed. There is not a large amount of additional work to add in effects of accidental torsion.

In our example building, the west building barely exceeded the trigger for torsional irregularity Type 1a. The value of the drift at a corner was 1.21 times the average drift for both directions. We compared the wall shear and overturning moment at the four shear walls for the cases with and without accidental torsion considered. The shear and moment at the base were 2 to 12 percent higher considering the effect of accidental torsion. I recommend keeping the consideration of accidental torsion.

**B.3.4.5 Technical and Editorial Simplifications in Chapter 13 (Section 24.15)**

Chapter 13 is a very confusing chapter of ASCE 7 particularly related to determining when the chapter must be used and when exceptions apply. On Seismic Design Category B projects, I believe that many times the chapter is ignored, or it is applied to components that do not need to follow it. Since very few parts of Chapter 13 actually apply to SDC B, it is very convenient to have those requirements in the new Chapter 24. There appears to be a conflict between Chapter 13 and Chapter 24 for parapets. Chapter 13 is clear that the parapets in question are those that are supported by bearing walls or shear walls, whereas Chapter 24 seems to apply to all parapets.

**B.3.5 Specific Review Comments for Secondary Items in Chapter 24**

**B.3.5.1 Removal of Plastic Analysis Exception in Section 12.4.3.1 (24.5.3)**

I agree with the deletion of this exception as it is extremely unlikely it would be used in Seismic Design Category B.

**B.3.5.2 Removal of Chapter 19 Foundation Modeling Option in Section 12.7.1 (24.8.1) and Associated Removal of Sections 12.8.1.2 and 12.9.7**

I agree with removal of the reference to Chapter 19 for modeling of foundation flexibility, and I recommend that section 24.14.3 also be deleted. Foundation modeling is generally an advanced analysis used to capture a potential change in dynamic characteristics or a change in load distribution due to high variability in foundation stiffness. Advanced analyses such as these are uncommon and not consistent with a simplified procedure. I recommend that section 24.8.1 be reworded to redirect the user to Chapter 12 when foundation flexibility is a design consideration.
**B.3.5.3 Streamlined Cs Calculation in Section 12.8.1.1 (24.9.1)**

The only actual change is the elimination of the long period region of the spectrum. This will be conservative for structures with very long periods, but should affect very few projects.

**B.3.5.4 Streamlined Approximate Fundamental Period Determination in Section 12.8.2.1 (24.9.2.1)**

Three methods for computing fundamental period were reduced to one, which is the most common method; I agree with this simplification.

**B.3.5.5 Removal of the Simplified Design Procedure of Section 12.4**

The proposed Chapter 24 is a simplification itself, so it does not need to include the separate Simplified Alternative method. An engineer can still go to the main portion of ASCE 7 to use this method.

**B.3.6 Summary**

I believe that the seismic analysis and design process for buildings in Seismic Design Category B is simplified and the engineer’s effort reduced by the use of the proposed Chapter 24 in place of Chapters 12 and 13. There is one change I disagree with (neglecting accidental torsion), and there are some additional items that I think could be removed from Chapter 24, including response spectrum analysis and foundation modeling criteria.
B.4.1  **TRIAL DESIGN BY STANLEY D. LINDSAY & ASSOCIATES, LTD.**

B.4.1  **Overview of Trial Design Team**

This trial design and review effort was performed by William P. Jacobs V, P.E. Mr. Jacobs is a Principal at Stanley D. Lindsey & Associates, Ltd., in Atlanta, GA, where he has been involved in the design of building structures in low and moderate seismicity regions throughout the southeast United States. Mr. Jacobs serves as a member of the AISC Committee on Manuals and is vice-chair of AISC’s task committee on composite construction. He is also a member of the ASCE National Technical Planning Committee, the ASCE committee on composite design, and is an associate member of ASCE 37.

B.4.2  **Overview of Example Building(s) used as Basis of the Trial Design**

The trial design building is a six-story, 180,000 square foot structural steel building located near Charlotte, NC. The typical floor plate is approximately 230 ft. x 120 ft., and the roof level is at approximately 90 ft. above grade. Typical gravity framing consists of structural steel beams and girders acting compositely with a metal deck and concrete floor system. The lateral force resisting system consists of dual braced-frame cores containing multiple two-story tall “x” braces. The cores are located near the perimeter of each end of the building, thus this building is not prone to torsional irregularities. The lateral force resisting system is detailed as an R=3 “Steel Systems Not Specifically Detailed for Seismic Resistance” system.

B.4.3  **General Review Comments for the New Chapter 24 Document**

The readability and organization of the proposed chapter matches or exceeds that of the remainder of ASCE 7. Removal and reorganization of the material within the chapter increases clarity for the building types targeted by this proposal. The reviewer feels that additional reductions could be made to enhance the clarity of the chapter further. These reductions include the removal of the Modal Response Spectrum Analysis procedure, which is not likely to be used by a designer invoking simplified provisions, as well as the removal of additional systems in Table 12.2-1 (24.3-1) as discussed in Section B.4.4.1 of this review. Additionally, the reviewer recommends several minor revisions be made to the technical changes within the proposal as discussed in Sections B.4.4.3-B.4.4.5.
B.4.4 Specific Review Comments for Primary Items in Chapter 24

B.4.4.1 Revisions to Table 12.2-1 (Table 24.3-1 in Chapter 24)

The clarity of the table is enhanced due to the removal of the extraneous material. The reviewer agrees with all material that has been removed. Additionally, the reviewer feels that it is highly unlikely that a user that elects to implement a “Dual System with Intermediate Moment Frames…” would be the type of user in need of a simplified version of the Provisions and therefore recommends the removal of “E. Dual Systems…” as well. For steel systems in particular, it is the reviewer’s experience that it is highly unlikely and uneconomical for any other system than “H. Steel Systems Not Specifically Details for Seismic Resistance…” to be used for a SDC “B” building. Consideration should be given to removing all of the other ordinary and intermediate steel systems (with the exception the cantilevered column systems in Section G).

B.4.4.2 Editorial Changes to Irregularity Section 12.3.2 (Section 24.4.2 in Chapter 24) and Tables 12.3-1 and 12.3-2 (Tables 24.4-1 and 24.4-2)

The reviewer takes no exception to this editorial revision. The edits to the tables are clear and readability is unaffected. The removal of the numerous inapplicable irregularities helps greatly in simplifying the feel of this section.

It is noted that the only effect of a Type 1a torsional irregularity in Table 24.4-1 is to require a 3-d model, and that in general a 3-d model is necessary to determine if a Type 1a irregularity exists. Similarly, the only consequence of a Type 5 Nonparallel System Irregularity is to require an appropriate 3-d model, and in general this is self-evident.

B.4.4.3 Technical Change to use $Ev = 0$ in Sections 12.4.2-12.4.3 (24.5.2-24.5.3)

The reviewer has not yet encountered a substantive revision to a SDC B building due to vertical seismic component, yet its implementation in loading combinations can be quite tedious. This is a most welcome revision.

For the trial design building, the maximum tension in any brace of the braced frame lateral seismic resisting system decreased from 208 kips when including the vertical seismic component to 204 kips without the vertical seismic component (a 1.9% reduction). Similarly, the maximum ultimate-level footing uplift decreased from 366 kips with the vertical seismic component to 348 kips without (a 4.9% reduction). Both revisions are, in the reviewer’s opinion, below the threshold of significance for the final design and detailing of a typical SDC B building.
Though not the focus of this review, it is recommended that this revision be considered for the main body of the provisions as well.

**B.4.4.4 Technical Change to Only Require Accidental Torsion if a Type 1b Horizontal Irregularity Exists (Section 12.8.4.2/24.9.4.2)**

The idea behind this simplification has great merit, but it is the reviewer’s opinion that its implementation falls slightly short of its promise. The concept is that for general SDC B buildings without extreme torsional irregularities, accidental torsion requirements could be eliminated entirely thus drastically reducing the number of required loading combinations.

This is a welcome revision; however, since Section 24.9.4.2 requires the use of the accidental torsional moment to determine if a possible horizontal structural irregularity exists, which then determines if the user can ignore the accidental torsional moment, the perceived greatest strength of this revision (reduction in book keeping/load combinations) is lost. If it could be determined that the use of the accidental torsional moment in SDC B buildings was not required to classify torsional irregularity, the full benefits of this revision could be realized.

One possible means of accomplishing the goal of determining if the structure in question is indeed torsionally sensitive without the application of the accidental torsional moment would be to utilize similar guidance to that provided in Chapter 27, Chapter 28, and most notably Appendix D for Wind design. For instance Chapter 28 (Fig. 28.4-1, Note 5, Exception) prescriptively permits one story buildings less than 30 ft. in height, two story or less buildings of light framed construction, and two story or less buildings with flexible diaphragms to be except from torsional loading. Furthermore Section 27.4.6 refers to Appendix D which provides the same prescriptive exception plus numerous additional methods of classifying buildings based on stiffness and lateral force resistant system distributions.

For the trial building, the maximum tension in any brace of the braced frame lateral seismic resisting system decreased from 208 kips with accidental torsion to 198 kips without (a 4.8% reduction). Similarly, the maximum footing uplift decreased from 366 kips with accidental torsion to 344 kips without (a 6.0% reduction). It is anticipated that for a building configuration more susceptible to torsional loading, these percentages could increase.

Editorially, the reviewer believes that re-writing Section 24.9.4.2 to utilize the requirement for accidental torsion as the exception instead of the rule would make this section flow better. For example “Accidental torsional moments ($M_{ta}$) 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, need only be applied for structures with diaphragms that are not flexible and have Type 1b horizontal structural irregularities in accordance with Table 24.4-1.”

**B.4.4.5 Technical and Editorial Simplifications in Chapter 13 (Section 24.15)**

The reviewer takes no exception to the technical and editorial simplifications to this Chapter/Section. The removal of such a large amount of material lends clarity to the resulting product. This revision could have minimal effect on many designers, including almost all of those performed by the reviewer, due to the practice of delegating specialty façade and/or stair design that is common in many parts of the United States.

**B.4.5 Specific Review Comments for Secondary Items in Chapter 24**

**B.4.5.1 Removal of Plastic Analysis Exception in Section 12.4.3.1 (24.5.3)**

The reviewer takes no exception to this revision. It is highly unlikely that a designer taking advantage of simplified provisions for SDC B buildings would pursue exceptions requiring plastic mechanisms or nonlinear response analyses.

**B.4.5.2 Removal of Chapter 19 Foundation Modeling Option in Section 12.7.1 (24.8.1) and Associated Removal of Sections 12.8.1.2 and 12.9.7**

The reviewer takes no exception to the removal of this modeling option. It is highly unlikely that a designer taking advantage of simplified provisions for SDC B buildings would pursue advanced foundation modeling beyond that still allowed by Section 12.13.3 (24.14.3).

**B.4.5.3 Streamlined Calculation in Section 12.8.1.1 (24.9.1)**

The reviewer takes no exception to the simplifications made in this section. The removal of the long period region will have little impact on the vast majority of buildings for which the simplified provisions would be expected to be utilized.

**B.4.5.4 Streamlined Approximate Fundamental Period Determination in Section 12.8.2.1 (24.9.2.1)**

The reviewer disagrees with the removal of Equations 12.8-8 through 12.8-10. In particular, Equation 12.8-8 \( (T_a = 0.1N) \) represents a simplification that, in the reviewer’s opinion, would be more likely used by a designer taking advantage of simplified provisions for SDC B buildings than at any other time. Similarly, the building type for which Equation 12.8-9 is
targeted (low-rise masonry or concrete shear wall buildings) would also constitute a target area for the proposed simplified provisions.

The requirement that $C_u$ be taken as 1.6 in all cases should be reviewed for intent. It is the reviewer’s experience that buildings will often qualify for the maximum available $C_uT_s$; therefore, this provision would result in a force penalty of $(1.7/1.6 = 1.063)$. For the trial design building, this revision results in an increase in the maximum tension in any brace of the braced frame lateral seismic resisting system from 208 kips to 221 kips (a 6.3% increase). Due to the direct increase in applied force in combination with the effect of the building period revision on the vertical distribution exponent “k”, the maximum footing uplift increased from 366 kips to 397 kips (an 8.5% increase). This revision has the effect of partially negating the reduction in forces provided by the revisions to accidental torsion and the vertical seismic load. If offsetting effects is the goal of the revision, the reviewer takes no exception; however, if the intent is solely to simplify the provisions, this change provides a penalty for a minor simplification that does not seem warranted.

**B.4.5.5 Removal of the Simplified Design Procedure of Section 12.14**

It is reasonable to remove existing alternate simplified provisions from a document intending to create a new set of simplified provisions.

**B.4.6 Summary**

The proposed simplified provisions for Seismic Design Category B buildings represent a worthwhile effort that enhances the clarity of the provisions and removes a great deal of extraneous material. Additional revisions are recommended to further enhance the usability of the proposed provisions. These recommendations include the removal of Modal Response Spectrum Procedure, the removal of dual systems, recommendations for clarifying the need to apply accidental eccentricity, minor editorial changes, and the reinstatement of several approximate period methods.

Major technical revisions include the removal of the vertical seismic component as well as revisions to the accidental torsional requirements. It was noted in this trial design effort that the combined effect of these two revisions provides a force reduction of 8-11% for the focus brace and foundation elements as compared to the existing main provisions (refer to Table B.4-1); however, this reduction was offset by the increase in force resulting from the simplification of the $C_u$ equations. Implementation of all of the provisions within the proposal resulted in an overall reduction in tension force for the most highly loaded brace from 208 kips to 206 kips (a
The combined application of the proposed revisions had no appreciable effect on the trial design building, and the effect of the proposed revisions on the design of other SDC B buildings is anticipated to be minimal.

### Table B.4-5  Overview of Technical Revision Effects on Focus Elements

<table>
<thead>
<tr>
<th>Focus Element</th>
<th>Forces Associated with Proposed Revisions</th>
<th>Removal of Vertical Seismic Component 12.4.3 (24.5.3)</th>
<th>Removal of Accidental Eccentricity 12.8.4.2 (24.9.4.2)</th>
<th>Limit Cu to 1.6 12.8.2 (24.9.2)</th>
<th>All Proposed Simplified Provisions (Chapter 24)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brace with Maximum Tension Force (Kips)</td>
<td>208</td>
<td>204</td>
<td>198</td>
<td>221</td>
<td>206</td>
</tr>
<tr>
<td>% Delta to Current Chapter 12</td>
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<td>-1.9%</td>
<td>-4.8%</td>
<td>6.3%</td>
<td>-1.0%</td>
</tr>
<tr>
<td>Footing with Maximum Uplift (Kips)</td>
<td>366</td>
<td>348</td>
<td>344</td>
<td>397</td>
<td>357</td>
</tr>
<tr>
<td>% Delta to Current Chapter 12</td>
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<td>-4.9%</td>
<td>-6.0%</td>
<td>8.5%</td>
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