American Society of Civil Engineers

Minimum Design Loads for Buildings and Other Structures

This document uses both the International System of Units (SI) and customary units.

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In 2003, the Board of Direction approved the revision to the ASCE Rules for Standards Committees to govern the writing and maintenance of standards developed by the Society. All such standards are developed by a consensus standards process managed by the Society's Codes and Standards Committee (CSC). The consensus process includes balloting by a balanced standards committee made up of Society members and nonmembers, balloting by the membership of the Society as a whole, and balloting by the public. All standards are updated or reaffirmed by the same process at intervals not exceeding five years.

The following standards have been issued:

ANSI/ASCE 1-82 N-725 Guideline for Design and Analysis of Nuclear Safety Related Earth Structures
ASCE/EWRI 2-06 Measurement of Oxygen Transfer in Clean Water
ANSI/ASCE 3-91 Standard for the Structural Design of Composite Slabs and ANSI/ASCE 9-91 Standard Practice for the Construction and Inspection of Composite Slabs
ASCE 4-98 Seismic Analysis of Safety-Related Nuclear Structures
Building Code Requirements for Masonry Structures (ACI 530-02/ASCE 5-02/TMS 402-02) and Specifications for Masonry Structures (ACI 530.1-02/ASCE 6-02/TMS 602-02)
ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures
SEI/ASCE 8-02 Standard Specification for the Design of Cold-Formed Stainless Steel Structural Members
ANSI/ASCE 9-91 listed with ASCE 3-91
ASCE 10-97 Design of Latticed Steel Transmission Structures
SEI/ASCE 11-99 Guideline for Structural Condition Assessment of Existing Buildings
ASCE/EWRI 12-05 Guideline for the Design of Urban Subsurface Drainage
ASCE/EWRI 13-05 Standard Guidelines for Installation of Urban Subsurface Drainage
ASCE/EWRI 14-05 Standard Guidelines for Operation and Maintenance of Urban Subsurface Drainage
ASCE 15-98 Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations (SIDD)
ASCE 16-95 Standard for Load Resistance Factor Design (LRFD) of Engineered Wood Construction
ASCE 17-96 Air-Supported Structures
ASCE 18-96 Standard Guidelines for In-Process Oxygen Transfer Testing
ASCE 19-96 Structural Applications of Steel Cables for Buildings
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ANSI/ASCE/T&DI 21-05 Automated People Mover Standards—Part 1
ANSI/ASCE/T&DI 21.2-08 Automated People Mover Standards—Part 2
ANSI/ASCE/T&DI 21.3-08 Automated People Mover Standards—Part 3
ANSI/ASCE/T&DI 21.4-08 Automated People Mover Standards—Part 4
SEI/ASCE 23-97 Specification for Structural Steel Beams with Web Openings
ASCE/SEI 24-05 Flood Resistant Design and Construction
ASCE/SEI 25-06 Earthquake-Actuated Automatic Gas Shutoff Devices
ASCE 26-97 Standard Practice for Design of Buried Precast Concrete Box Sections
ASCE 27-00 Standard Practice for Direct Design of Precast Concrete Pipe for Jacking in Trenchless Construction
ASCE 28-00 Standard Practice for Direct Design of Precast Concrete Box Sections for Jacking in Trenchless Construction
ASCE/SEI/SFPE 29-05 Standard Calculation Methods for Structural Fire Protection
SEI/ASCE 30-00 Guideline for Condition Assessment of the Building Envelope
SEI/ASCE 31-03 Seismic Evaluation of Existing Buildings
SEI/ASCE 32-01 Design and Construction of Frost-Protected Shallow Foundations
EWRI/ASCE 33-01 Comprehensive Transboundary International Water Quality Management Agreement

EWRI/ASCE 34-01 Standard Guidelines for Artificial Recharge of Ground Water
EWRI/ASCE 35-01 Guidelines for Quality Assurance of Installed Fine-Pore Aeration Equipment
CI/ASCE 36-01 Standard Construction Guidelines for Microtunneling
SEI/ASCE 37-02 Design Loads on Structures during Construction
CI/ASCE 38-02 Standard Guideline for the Collection and Depiction of Existing Subsurface Utility Data
EWRI/ASCE 39-03 Standard Practice for the Design and Operation of Hail Suppression Projects
ASCE/EWRI 40-03 Regulated Riparian Model Water Code
ASCE/SEI 41-06 Seismic Rehabilitation of Existing Buildings
ASCE/EWRI 42-04 Standard Practice for the Design and Operation of Precipitation Enhancement Projects
ASCE/SEI 43-05 Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities
ASCE/EWRI 44-05 Standard Practice for the Design and Operation of Supercooled Fog Dispersal Projects
ASCE/EWRI 45-05 Standard Guidelines for the Design of Urban Stormwater Systems
ASCE/EWRI 46-05 Standard Guidelines for the Installation of Urban Stormwater Systems
ASCE/EWRI 47-05 Standard Guidelines for the Operation and Maintenance of Urban Stormwater Systems
ASCE/SEI 48-05 Design of Steel Transmission Pole Structures
ASCE/EWRI 50-08 Standard Guideline for Fitting Saturated Hydraulic Conductivity Using Probability Density Functions
ASCE/EWRI 51-08 Standard Guideline for Calculating the Effective Saturated Hydraulic Conductivity
ASCE/SEI 52-10 Design of Fiberglass-Reinforced Plastic (FRP) Stacks
ASCE/G-I 53-10 Compaction Grouting Consensus Guide

FOREWORD

The material presented in this standard has been prepared in accordance with recognized engineering principles. This standard should not be used without first securing competent advice with respect to its suitability for any given application. The publication of the material contained herein is not intended as a representation or warranty on the part of the American Society of Civil Engineers, or of any other person named herein, that this information is suitable for any general or particular use or promises freedom from infringement of any patent or patents. Anyone making use of this information assumes all liability from such use.

In the margin of Chapters 1 through 23, a bar has been placed to indicate a substantial technical revision in the standard from the 2005 edition. Because of the reorganization of the wind provisions, these bars are not used in Chapters 26 through 31. Likewise, bars are not used to indicate changes in any parts of the Commentary.

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This revision of the standard began in 2006 and incorporates information as described in the commentary.

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DEDICATION

Thomas R. Tyson, P.E., S.E.

The members of the Minimum Design Loads for Buildings and Other Structures Standards Committee of the Structural Engineering Institute respectfully dedicate this Standard in the memory of Thomas R. Tyson, P.E., S.E., who passed away on December 19, 2009.

His structural engineering expertise complemented his dedication to our profession, and these qualities guided the members of the Live Load Subcommittee, which he chaired during the preparation of this Standard. His practical advice, quick smile, and good nature will be greatly missed.

CONTENTS

Chapter 1
GENERAL

1.1 SCOPE

This standard provides minimum load requirements for the design of buildings and other structures that are subject to building code requirements. Loads and appropriate load combinations, which have been developed to be used together, are set forth for strength design and allowable stress design. For design strengths and allowable stress limits, design specifications for conventional structural materials used in buildings and modifications contained in this standard shall be followed.

1.2 DEFINITIONS AND NOTATIONS

1.2.1 Definitions

The following definitions apply to the provisions of the entire standard.

ALLOWABLE STRESS DESIGN: A method of proportioning structural members such that elastically computed stresses produced in the members by nominal loads do not exceed specified allowable stresses (also called "working stress design").

AUTHORITY HAVING JURISDICTION: The organization, political subdivision, office, or individual charged with the responsibility of administering and enforcing the provisions of this standard.

BUILDINGS: Structures, usually enclosed by walls and a roof, constructed to provide support or shelter for an intended occupancy.

DESIGN STRENGTH: The product of the nominal strength and a resistance factor.

ESSENTIAL FACILITIES: Buildings and other structures that are intended to remain operational in the event of extreme environmental loading from flood, wind, snow, or earthquakes.

FACTORED LOAD: The product of the nominal load and a load factor.

HIGHLY TOXIC SUBSTANCE: As defined in 29 CFR 1910.1200 Appendix A with Amendments as of February 1, 2000.

IMPORTANCE FACTOR: A factor that accounts for the degree of risk to human life, health, and welfare associated with damage to property or loss of use or functionality.

LIMIT STATE: A condition beyond which a structure or member becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state).

LOAD EFFECTS: Forces and deformations produced in structural members by the applied loads.

LOAD FACTOR: A factor that accounts for deviations of the actual load from the nominal load, for uncertainties in the analysis that transforms the load into a load effect, and for the probability that more than one extreme load will occur simultaneously.

LOADS: Forces or other actions that result from the weight of all building materials, occupants and their possessions, environmental effects, differential movement, and restrained dimensional changes. Permanent loads are those loads in which variations over time are rare or of small magnitude. All other loads are variable loads (see also "nominal loads").

NOMINAL LOADS: The magnitudes of the loads specified in this standard for dead, live, soil, wind, snow, rain, flood, and earthquake.

NOMINAL STRENGTH: The capacity of a structure or member to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

OCCUPANCY: The purpose for which a building or other structure, or part thereof, is used or intended to be used.

OTHER STRUCTURES: Structures, other than buildings, for which loads are specified in this standard.

P-DELTA EFFECT: The second order effect on shears and moments of frame members induced by axial loads on a laterally displaced building frame.

RESISTANCE FACTOR: A factor that accounts for deviations of the actual strength from the nominal strength and the manner and consequences of failure (also called "strength reduction factor").

RISK CATEGORY: A categorization of buildings and other structures for determination of flood, wind, snow, ice, and earthquake loads based on the risk associated with unacceptable performance. See Table 1.5-1.

STRENGTH DESIGN: A method of proportioning structural members such that the computed forces produced in the members by the factored loads do not

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent a low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life.	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure.	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where their quantity exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.	
Buildings and other structures designated as essential facilities.	IV
Buildings and other structures, the failure of which could pose a substantial hazard to the community.	
Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity exceeds a threshold quantity established by the authority having jurisdiction to be dangerous to the public if released and is sufficient to pose a threat to the public if released.[a]	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures.	

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the substances is commensurate with the risk associated with that Risk Category.

exceed the member design strength (also called “load and resistance factor design”).

TEMPORARY FACILITIES: Buildings or other structures that are to be in service for a limited time and have a limited exposure period for environmental loadings.

TOXIC SUBSTANCE: As defined in 29 CFR 1910.1200 Appendix A with Amendments as of February 1, 2000.

1.1.2 Symbols and Notations

F_x A minimum design lateral force applied to level x of the structure and used for purposes of evaluating structural integrity in accordance with Section 1.4.2.

W_x The portion of the total dead load of the structure, D, located or assigned to Level x.

D Dead load.

L Live load.

L_r Roof live load.

N Notional load used to evaluate conformance with minimum structural integrity criteria.

R Rain load.

S Snow load.

1.3 BASIC REQUIREMENTS

1.3.1 Strength and Stiffness

Buildings and other structures, and all parts thereof, shall be designed and constructed with adequate strength and stiffness to provide structural stability, protect nonstructural components and systems from unacceptable damage, and meet the serviceability requirements of Section 1.3.2.

Acceptable strength shall be demonstrated using one or more of the following procedures:

a. the Strength Procedures of Section 1.3.1.1,
b. the Allowable Stress Procedures of Section 1.3.1.2, or
c. subject to the approval of the authority having jurisdiction for individual projects, the Performance-Based Procedures of Section 1.3.1.3.

It shall be permitted to use alternative procedures for different parts of a structure and for different load combinations, subject to the limitations of Chapter 2. Where resistance to extraordinary events is considered, the procedures of Section 2.5 shall be used.

1.3.1.1 Strength Procedures

Structural and nonstructural components and their connections shall have adequate strength to resist the applicable load combinations of Section 2.3 of this Standard without exceeding the applicable strength limit states for the materials of construction.

1.3.1.2 Allowable Stress Procedures

Structural and nonstructural components and their connections shall have adequate strength to resist the applicable load combinations of Section 2.4 of this Standard without exceeding the applicable allowable stresses for the materials of construction.

1.3.1.3 Performance-Based Procedures

Structural and nonstructural components and their connections shall be demonstrated by analysis or by a combination of analysis and testing to provide a reliability not less than that expected for similar components designed in accordance with the Strength Procedures of Section 1.3.1.1 when subject to the influence of dead, live, environmental, and other loads. Consideration shall be given to uncertainties in loading and resistance.

1.3.1.3.1 Analysis Analysis shall employ rational methods based on accepted principles of engineering mechanics and shall consider all significant sources of deformation and resistance. Assumptions of stiffness, strength, damping, and other properties of components and connections incorporated in the analysis shall be based on approved test data or referenced Standards.

1.3.1.3.2 Testing Testing used to substantiate the performance capability of structural and nonstructural components and their connections under load shall accurately represent the materials, configuration, construction, loading intensity, and boundary conditions anticipated in the structure. Where an approved industry standard or practice that governs the testing of similar components exists, the test program and determination of design values from the test program shall be in accordance with those industry standards and practices. Where such standards or practices do not exist, specimens shall be constructed to a scale similar to that of the intended application unless it can be demonstrated that scale effects are not significant to the indicated performance. Evaluation of test results shall be made on the basis of the values obtained from not less than 3 tests, provided that the deviation of any value obtained from any single test does not vary from the average value for all tests by more than 15%. If such deviaton from the average value for any test exceeds 15%, then additional tests shall be performed until the deviation of any test from the average value does not exceed 15% or a minimum of 6 tests have been performed. No test shall be eliminated unless a rationale for its exclusion is given. Test reports shall document the location, the time and date of the test, the characteristics of the tested specimen, the laboratory facilities, the test configuration, the applied loading and deformation under load, and the occurrence of any damage sustained by the specimen, together with the loading and deformation at which such damage occurred.

1.3.1.3.3 Documentation The procedures used to demonstrate compliance with this section and the results of analysis and testing shall be documented in one or more reports submitted to the authority having jurisdiction and to an independent peer review.

1.3.1.3.4 Peer Review The procedures and results of analysis, testing, and calculation used to demonstrate compliance with the requirements of this section shall be subject to an independent peer review approved by the authority having jurisdiction. The peer review shall comprise one or more persons having the necessary expertise and knowledge to evaluate compliance, including knowledge of the expected performance, the structural and component behavior, the particular loads considered, structural analysis of the type performed, the materials of construction, and laboratory testing of elements and components to determine structural resistance and performance characteristics. The review shall include the assumptions, criteria, procedures, calculations, analytical models, test setup, test data, final drawings, and reports. Upon satisfactory completion, the peer review shall submit a letter to the authority having jurisdiction indicating the scope of their review and their findings.

1.3.2 Serviceability

Structural systems, and members thereof, shall be designed to have adequate stiffness to limit deflections, lateral drift, vibration, or any other deformations that adversely affect the intended use and performance of buildings and other structures.

1.3.3 Self-Straining Forces

Provision shall be made for anticipated self-straining forces arising from differential settlements of foundations and from restrained dimensional changes due to temperature, moisture, shrinkage, creep, and similar effects.

1.3.4 Analysis

Load effects on individual structural members shall be determined by methods of structural analysis that take into account equilibrium, general stability, geometric compatibility, and both short- and long-term material properties. Members that tend to accumulate residual deformations under repeated service loads shall have included in their analysis the added eccentricities expected to occur during their service life.

1.3.5 Counteracting Structural Actions

All structural members and systems, and all components and cladding in a building or other structure, shall be designed to resist forces due to earthquake and wind, with consideration of overturning, sliding, and uplift, and continuous load paths shall be provided for transmitting these forces to the foundation. Where sliding is used to isolate the elements, the effects of friction between sliding elements shall be included as a force. Where all or a portion of the resistance to these forces is provided by dead load, the dead load shall be taken as the minimum dead load likely to be in place during the event causing the considered forces. Consideration shall be given to the effects of vertical and horizontal deflections resulting from such forces.

1.4 GENERAL STRUCTURAL INTEGRITY

All structures shall be provided with a continuous load path in accordance with the requirements of Section 1.4.1 and shall have a complete lateral force-resisting system with adequate strength to resist the forces indicated in Section 1.4.2. All members of the structural system shall be connected to their supporting members in accordance with Section 1.4.3. Structural walls shall be anchored to diaphragms and supports in accordance with Section 1.4.4. The effects on the structure and its components due to the forces stipulated in this section shall be taken as the notional load, N, and combined with the effects of other loads in accordance with the load combinations of Section of Section 1.4.1. Where material resistance is dependent on load duration, notional loads are permitted to be taken as having a duration of 10 minutes. Structures designed in conformance with the requirements of this Standard for Seismic Design Categories B, C, D, E, or F shall be deemed to comply with the requirements of Sections 1.4.1, 1.4.2, 1.4.3, 1.4.4 and 1.4.5.

1.4.1 Load Combinations of Integrity Loads

The notional loads, N, specified in Sections 1.4.2 through 1.4.5 shall be combined with dead and live loads in accordance with Section 1.4.1.1 for strength design and 1.4.1.2 for allowable stress design.

1.4.1.1 Strength Design Notional Load Combinations

a. $1.2D + 1.0N + L + 0.2S$
b. $0.9D + 1.0N$

1.4.1.2 Allowable Stress Design Notional Load Combinations

a. $D\ 0.7N$
b. $D + 0.75(0.7N) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
c. $0.6D + 0.7N$

1.4.2 Load Path Connections

All parts of the structure between separation joints shall be interconnected to form a continuous path to the lateral force-resisting system, and the connections shall be capable of transmitting the lateral forces induced by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure with elements having strength to resist a force of not less than 5% of the portion's weight.

1.4.3 Lateral Forces

Each structure shall be analyzed for the effects of static lateral forces applied independently in each of two orthogonal directions. In each direction, the static lateral forces at all levels shall be applied simultaneously. For purposes of analysis, the force at each level shall be determined using Eq. 1.4-1 as follows:

$$F_x = 0.01\ W_x \tag{1.4-1}$$

where

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

Structures explicitly designed for stability, including second-order effects, shall be deemed to comply with the requirements of this section.

1.4.4 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, the member's supporting element shall also be connected to the diaphragm. The connection shall have the strength to resist a force of 5 percent of the unfactored dead load plus live load reaction imposed by the supported member on the supporting member.

1.4.5 Anchorage of Structural Walls

Walls that provide vertical load bearing or lateral shear resistance for a portion of the structure shall be anchored to the roof and all floors and members that provide lateral support for the wall or that are supported by the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be capable of resisting a strength level horizontal force perpendicular to the plane of the wall equal to 0.2 times the weight of the wall tributary to the connection, but not less than 5 psf (0.24 kN/m^2).

1.4.6 Extraordinary Loads and Events

When considered, design for resistance to extraordinary loads and events shall be in accordance with the procedures of Section 2.5.

1.5 CLASSIFICATION OF BUILDINGS AND OTHER STRUCTURES

1.5.1 Risk Categorization

Buildings and other structures shall be classified, based on the risk to human life, health, and welfare associated with their damage or failure by nature of their occupancy or use, according to Table 1.5-1 for the purposes of applying flood, wind, snow, earthquake, and ice provisions. Each building or other structure shall be assigned to the highest applicable risk category or categories. Minimum design loads for structures shall incorporate the applicable importance factors given in Table 1.5-2, as required by other sections of this Standard. Assignment of a building or other structure to multiple risk categories based on the type of load condition being evaluated (e.g., snow or seismic) shall be permitted.

When the building code or other referenced standard specifies an Occupancy Category, the Risk Category shall not be taken as lower than the Occupancy Category specified therein.

1.5.2 Multiple Risk Categories

Where buildings or other structures are divided into portions with independent structural systems, the classification for each portion shall be permitted to be determined independently. Where building systems, such as required egress, HVAC, or electrical power, for a portion with a higher risk category pass through or depend on other portions of the building or other structure having a lower risk category, those portions shall be assigned to the higher risk category.

1.5.3 Toxic, Highly Toxic, and Explosive Substances

Buildings and other structures containing toxic, highly toxic, or explosive substances are permitted to be classified as Risk Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as part of an overall risk management plan (RMP) that a release of the toxic, highly toxic, or explosive substances is not sufficient to pose a threat to the public.

To qualify for this reduced classification, the owner or operator of the buildings or other structures

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads[a]

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.25	1.00	1.25
IV	1.20	1.25	1.00	1.50

[a]The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it is dependent on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

containing the toxic, highly toxic, or explosive substances shall have an RMP that incorporates three elements as a minimum: a hazard assessment, a prevention program, and an emergency response plan.

As a minimum, the hazard assessment shall include the preparation and reporting of worst-case release scenarios for each structure under consideration, showing the potential effect on the public for each. As a minimum, the worst-case event shall include the complete failure (instantaneous release of entire contents) of a vessel, piping system, or other storage structure. A worst-case event includes (but is not limited to) a release during the design wind or design seismic event. In this assessment, the evaluation of the effectiveness of subsequent measures for accident mitigation shall be based on the assumption that the complete failure of the primary storage structure has occurred. The offsite impact shall be defined in terms of population within the potentially affected area. To qualify for the reduced classification, the hazard assessment shall demonstrate that a release of the toxic, highly toxic, or explosive substances from a worst-case event does not pose a threat to the public outside the property boundary of the facility.

As a minimum, the prevention program shall consist of the comprehensive elements of process safety management, which is based upon accident prevention through the application of management controls in the key areas of design, construction, operation, and maintenance. Secondary containment of the toxic, highly toxic, or explosive substances (including, but not limited to, double wall tank, dike of sufficient size to contain a spill, or other means to contain a release of the toxic, highly toxic, or explosive substances within the property boundary of the facility and prevent release of harmful quantities of contaminants to the air, soil, ground water, or surface water) are permitted to be used to mitigate the risk of release. Where secondary containment is provided, it shall be designed for all environmental loads and is not eligible for this reduced classification. In hurricane-prone regions, mandatory practices and procedures that effectively diminish the effects of wind on critical structural elements or that alternatively protect against harmful releases during and after hurricanes are permitted to be used to mitigate the risk of release.

As a minimum, the emergency response plan shall address public notification, emergency medical treatment for accidental exposure to humans, and procedures for emergency response to releases that have consequences beyond the property boundary of the facility. The emergency response plan shall address the potential that resources for response could be compromised by the event that has caused the emergency.

1.6 ADDITIONS AND ALTERATIONS TO EXISTING STRUCTURES

When an existing building or other structure is enlarged or otherwise altered, structural members affected shall be strengthened if necessary so that the factored loads defined in this document will be supported without exceeding the specified design strength for the materials of construction. When using allowable stress design, strengthening is required when the stresses due to nominal loads exceed the specified allowable stresses for the materials of construction.

1.7 LOAD TESTS

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

1.8 CONSENSUS STANDARDS AND OTHER REFERENCED DOCUMENTS

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

OSHA
Occupational Safety and Health Administration
200 Constitution Avenue, NW
Washington, DC 20210

29 CFR 1910.1200 Appendix A with Amendments as of February 1, 2000.
Section 1.2
OSHA Standards for General Industry, 29 CFR (Code of Federal Regulations) Part 1910.1200
Appendix A, United States Department of Labor, Occupational Safety and Health Administration, Washington, DC, 2005

Chapter 2
COMBINATIONS OF LOADS

2.1 GENERAL

Buildings and other structures shall be designed using the provisions of either Section 2.3 or 2.4. Where elements of a structure are designed by a particular material standard or specification, they shall be designed exclusively by either Section 2.3 or 2.4.

2.2 SYMBOLS

A_k = load or load effect arising from extra ordinary event A
D = dead load
D_i = weight of ice
E = earthquake load
F = load due to fluids with well-defined pressures and maximum heights
F_a = flood load
H = load due to lateral earth pressure, ground water pressure, or pressure of bulk materials
L = live load
L_r = roof live load
R = rain load
S = snow load
T = self-straining load
W = wind load
W_i = wind-on-ice determined in accordance with Chapter 10

2.3 COMBINING FACTORED LOADS USING STRENGTH DESIGN

2.3.1 Applicability

The load combinations and load factors given in Section 2.3.2 shall be used only in those cases in which they are specifically authorized by the applicable material design standard.

2.3.2 Basic Combinations

Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.0W$
7. $0.9D + 1.0E$

EXCEPTIONS:

1. The load factor on L in combinations 3, 4, and 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, with the exception of garages or areas occupied as places of public assembly.
2. In combinations 2, 4, and 5, the companion load S shall be taken as either the flat roof snow load (p_f) or the sloped roof snow load (p_s).

Where fluid loads F are present, they shall be included with the same load factor as dead load D in combinations 1 through 5 and 7.

Where load H are present, they shall be included as follows:

1. where the effect of H adds to the primary variable load effect, include H with a load factor of 1.6;
2. where the effect of H resists the primary variable load effect, include H with a load factor of 0.9 where the load is permanent or a load factor of 0 for all other conditions.

Effects of one or more loads not acting shall be investigated. The most unfavorable effects from both wind and earthquake loads shall be investigated, where appropriate, but they need not be considered to act simultaneously. Refer to Section 12.4 for specific definition of the earthquake load effect E.[1]

Each relevant strength limit state shall be investigated.

2.3.3 Load Combinations Including Flood Load

When a structure is located in a flood zone (Section 5.3.1), the following load combinations shall be considered in addition to the basic combinations in Section 2.3.2:

1. In V-Zones or Coastal A-Zones, $1.0W$ in combinations 4 and 6 shall be replaced by $1.0W + 2.0F_a$.
2. In noncoastal A-Zones, $1.0W$ in combinations 4 and 6 shall be replaced by $0.5W + 1.0F_a$.

[1]The same E from Sections 1.4 and 12.4 is used for both Sections 2.3.2 and 2.4.1. Refer to the Chapter 11 Commentary for the Seismic Provisions.

2.3.4. Load Combinations Including Atmospheric Ice Loads

When a structure is subjected to atmospheric ice and wind-on-ice loads, the following load combinations shall be considered:

1. $0.5(L_r$ or S or $R)$ in combination 2 shall be replaced by $0.2D_i + 0.5S$.
2. $1.0W + 0.5(L_r$ or S or $R)$ in combination 4 shall be replaced by $D_i + W_i + 0.5S$.
3. $1.0W$ in combination 6 shall be replaced by $D_i + W_i$.

2.3.5 Load Combinations Including Self-Straining Loads

Where applicable, the structural effects of load T shall be considered in combination with other loads. The load factor on load T shall be established considering the uncertainty associated with the likely magnitude of the load, the probability that the maximum effect of T will occur simultaneously with other applied loadings, and the potential adverse consequences if the effect of T is greater than assumed. The load factor on T shall not have a value less than 1.0.

2.3.6 Load Combinations for Nonspecified Loads

Where approved by the Authority Having Jurisdiction, the Responsible Design Professional is permitted to determine the combined load effect for strength design using a method that is consistent with the method on which the load combination requirements in Section 2.3.2 are based. Such a method must be probability-based and must be accompanied by documentation regarding the analysis and collection of supporting data that is acceptable to the Authority Having Jurisdiction.

2.4 COMBINING NOMINAL LOADS USING ALLOWABLE STRESS DESIGN

2.4.1 Basic Combinations

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

1. D
2. $D + L$
3. $D + (L_r$ or S or $R)$
4. $D + 0.75L + 0.75(L_r$ or S or $R)$
5. $D + (0.6W$ or $0.7E)$

6a. $D + 0.75L + 0.75(0.6W) + 0.75(L_r$ or S or $R)$

6b. $D + 0.75L + 0.75(0.7E) + 0.75S$

7. $0.6D + 0.6W$
8. $0.6D + 0.7E$

EXCEPTIONS:

1. In combinations 4 and 6, the companion load S shall be taken as either the flat roof snow load (p_f) or the sloped roof snow load (p_s).
2. For nonbuilding structures, in which the wind load is determined from force coefficients, C_f, identified in Figures 29.5-1, 29.5-2 and 29.5-3 and the projected area contributing wind force to a foundation element exceeds 1,000 square feet on either a vertical or a horizontal plane, it shall be permitted to replace W with $0.9W$ in combination 7 for design of the foundation, excluding anchorage of the structure to the foundation.
3. It shall be permitted to replace $0.6D$ with $0.9D$ in combination 8 for the design of Special Reinforced Masonry Shear Walls, where the walls satisfy the requirement of Section 14.4.2.

Where fluid loads F are present, they shall be included in combinations 1 through 6 and 8 with the same factor as that used for dead load D.

Where load H is present, it shall be included as follows:

1. where the effect of H adds to the primary variable load effect, include H with a load factor of 1.0;
2. where the effect of H resists the primary variable load effect, include H with a load factor of 0.6 where the load is permanent or a load factor of 0 for all other conditions.

The most unfavorable effects from both wind and earthquake loads shall be considered, where appropriate, but they need not be assumed to act simultaneously. Refer to Section 1.4 and 12.4 for the specific definition of the earthquake load effect E.[2]

Increases in allowable stress shall not be used with the loads or load combinations given in this standard unless it can be demonstrated that such an increase is justified by structural behavior caused by rate or duration of load.

[2]The same E from Sections 1.4 and 12.4 is used for both Sections 2.3.2 and 2.4.1. Refer to the Chapter 11 Commentary for the Seismic Provisions.

2.4.2 Load Combinations Including Flood Load

When a structure is located in a flood zone, the following load combinations shall be considered in addition to the basic combinations in Section 2.4.1:

1. In V-Zones or Coastal A-Zones (Section 5.3.1), $1.5F_a$ shall be added to other loads in combinations 5, 6, and 7, and E shall be set equal to zero in 5 and 6.
2. In non-coastal A-Zones, $0.75F_a$ shall be added to combinations 5, 6, and 7, and E shall be set equal to zero in 5 and 6.

2.4.3 Load Combinations Including Atmospheric Ice Loads

When a structure is subjected to atmospheric ice and wind-on-ice loads, the following load combinations shall be considered:

1. $0.7D_i$ shall be added to combination 2.
2. (L_r or S or R) in combination 3 shall be replaced by $0.7D_i + 0.7W_i + S$.
3. $0.6W$ in combination 7 shall be replaced by $0.7D_i + 0.7W_i$.

2.4.4 Load Combinations Including Self-Straining Loads

Where applicable, the structural effects of load T shall be considered in combination with other loads. Where the maximum effect of load T is unlikely to occur simultaneously with the maximum effects of other variable loads, it shall be permitted to reduce the magnitude of T considered in combination with these other loads. The fraction of T considered in combination with other loads shall not be less than 0.75.

2.5 LOAD COMBINATIONS FOR EXTRAORDINARY EVENTS

2.5.1 Applicability

Where required by the owner or applicable code, strength and stability shall be checked to ensure that structures are capable of withstanding the effects of extraordinary (i.e., low-probability) events, such as fires, explosions, and vehicular impact without disproportionate collapse.

2.5.2 Load Combinations

2.5.2.1 Capacity

For checking the capacity of a structure or structural element to withstand the effect of an extraordinary event, the following gravity load combination shall be considered:

$$(0.9 \text{ or } 1.2)D + A_k + 0.5L + 0.2S \qquad (2.5\text{-}1)$$

in which A_k = the load or load effect resulting from extraordinary event A.

2.5.2.2 Residual Capacity

For checking the residual load-carrying capacity of a structure or structural element following the occurrence of a damaging event, selected load-bearing elements identified by the Responsible Design Professional shall be notionally removed, and the capacity of the damaged structure shall be evaluated using the following gravity load combination:

$$(0.9 \text{ or } 1.2)D + 0.5L + 0.2(L_r \text{ or } S \text{ or } R) \qquad (2.5\text{-}2)$$

2.5.3 Stability Requirements

Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of second-order effects is permitted.

Chapter 3
DEAD LOADS, SOIL LOADS, AND HYDROSTATIC PRESSURE

3.1 DEAD LOADS

3.1.1 Definition

Dead loads consist of the weight of all materials of construction incorporated into the building including, but not limited to, walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding, and other similarly incorporated architectural and structural items, and fixed service equipment including the weight of cranes.

3.1.2 Weights of Materials and Constructions

In determining dead loads for purposes of design, the actual weights of materials and constructions shall be used provided that in the absence of definite information, values approved by the authority having jurisdiction shall be used.

3.1.3 Weight of Fixed Service Equipment

In determining dead loads for purposes of design, the weight of fixed service equipment, such as plumbing stacks and risers, electrical feeders, and heating, ventilating, and air conditioning systems shall be included.

3.2 SOIL LOADS AND HYDROSTATIC PRESSURE

3.2.1 Lateral Pressures

In the design of structures below grade, provision shall be made for the lateral pressure of adjacent soil. If soil loads are not given in a soil investigation report approved by the authority having jurisdiction, then the soil loads specified in Table 3.2-1 shall be used as the

Table 3.2-1 Design Lateral Soil Load

Description of Backfill Material	Unified Soil Classification	Design Lateral Soil Load[a] psf per foot of depth (kN/m^2 per meter of depth)
Well-graded, clean gravels; gravel–sand mixes	GW	35 (5.50)[b]
Poorly graded clean gravels; gravel–sand mixes	GP	35 (5.50)[b]
Silty gravels, poorly graded gravel–sand mixes	GM	35 (5.50)[b]
Clayey gravels, poorly graded gravel-and-clay mixes	GC	45 (7.07)[b]
Well-graded, clean sands; gravelly–sand mixes	SW	35 (5.50)[b]
Poorly graded clean sands; sand–gravel mixes	SP	35 (5.50)[b]
Silty sands, poorly graded sand–silt mixes	SM	45 (7.07)[b]
Sand–silt clay mix with plastic fines	SM–SC	85 (13.35)[c]
Clayey sands, poorly graded sand–clay mixes	SC	85 (13.35)[c]
Inorganic silts and clayey silts	ML	85 (13.35)[c]
Mixture of inorganic silt and clay	ML–CL	85 (13.35)[c]
Inorganic clays of low to medium plasticity	CL	100 (15.71)
Organic silts and silt–clays, low plasticity	OL	[d]
Inorganic clayey silts, elastic silts	MH	[d]
Inorganic clays of high plasticity	CH	[d]
Organic clays and silty clays	OH	[d]

[a]Design lateral soil loads are given for moist conditions for the specified soils at their optimum densities. Actual field conditions shall govern. Submerged or saturated soil pressures shall include the weight of the buoyant soil plus the hydrostatic loads.

[c]For relatively rigid walls, as when braced by floors, the design lateral soil load shall be increased for sand and gravel type soils to 60 psf (9.43 kN/m^2) per foot (meter) of depth. Basement walls extending not more than 8 ft (2.44 m) below grade and supporting light floor systems are not considered as being relatively rigid walls.

[d]For relatively rigid walls, as when braced by floors, the design lateral load shall be increased for silt and clay type soils to 100 psf (15.71 kN/m^2) per foot (meter) of depth. Basement walls extending not more than 8 ft (2.44 m) below grade and supporting light floor systems are not considered as being relatively rigid walls.

[b]Unsuitable as backfill material.

minimum design lateral loads. Due allowance shall be made for possible surcharge from fixed or moving loads. When a portion or the whole of the adjacent soil is below a free-water surface, computations shall be based upon the weight of the soil diminished by buoyancy, plus full hydrostatic pressure.

The lateral pressure shall be increased if soils with expansion potential are present at the site as determined by a geotechnical investigation.

3.2.2 Uplift on Floors and Foundations

In the design of basement floors and similar approximately horizontal elements below grade, the upward pressure of water, where applicable, shall be taken as the full hydrostatic pressure applied over the entire area. The hydrostatic load shall be measured from the underside of the construction. Any other upward loads shall be included in the design.

Where expansive soils are present under foundations or slabs-on-ground, the foundations, slabs, and other components shall be designed to tolerate the movement or resist the upward loads caused by the expansive soils, or the expansive soil shall be removed or stabilized around and beneath the structure.

Chapter 4
LIVE LOADS

4.1 DEFINITIONS

FIXED LADDER: A ladder that is permanently attached to a structure, building, or equipment.

GRAB BAR SYSTEM: A bar and associated anchorages and attachments to the structural system, for the support of body weight in locations such as toilets, showers, and tub enclosures.

GUARDRAIL SYSTEM: A system of components, including anchorages and attachments to the structural system, near open sides of an elevated surface for the purpose of minimizing the possibility of a fall from the elevated surface by people, equipment, or material.

HANDRAIL SYSTEM: A rail grasped by hand for guidance and support, and associated anchorages and attachments to the structural system.

HELIPAD: A structural surface that is used for landing, taking off, taxiing, and parking of helicopters.

LIVE LOAD: A load produced by the use and occupancy of the building or other structure that does not include construction or environmental loads, such as wind load, snow load, rain load, earthquake load, flood load, or dead load.

ROOF LIVE LOAD: A load on a roof produced (1) during maintenance by workers, equipment, and materials and (2) during the life of the structure by movable objects, such as planters or other similar small decorative appurtenances that are not occupancy related.

SCREEN ENCLOSURE: A building or part thereof, in whole or in part self-supporting, having walls and a roof of insect or sun screening using fiberglass, aluminum, plastic, or similar lightweight netting material, which enclose an occupancy or use such as outdoor swimming pools, patios or decks, and horticultural and agricultural production facilities.

VEHICLE BARRIER SYSTEM: A system of components, including anchorages and attachments to the structural system near open sides or walls of garage floors or ramps, that acts as a restraint for vehicles.

4.2 LOADS NOT SPECIFIED

For occupancies or uses not designated in this chapter, the live load shall be determined in accordance with a method approved by the authority having jurisdiction.

4.3 UNIFORMLY DISTRIBUTED LIVE LOADS

4.3.1 Required Live Loads

The live loads used in the design of buildings and other structures shall be the maximum loads expected by the intended use or occupancy, but shall in no case be less than the minimum uniformly distributed unit loads required by Table 4-1, including any permissible reduction.

4.3.2 Provision for Partitions

In office buildings or other buildings where partitions will be erected or rearranged, provision for partition weight shall be made, whether or not partitions are shown on the plans. Partition load shall not be less than 15 psf (0.72 kN/m^2).

EXCEPTION: A partition live load is not required where the minimum specified live load exceeds 80 psf (3.83 kN/m^2).

4.3.3 Partial Loading

The full intensity of the appropriately reduced live load applied only to a portion of a structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member. Roof live loads shall be distributed as specified in Table 4-1.

4.4 CONCENTRATED LIVE LOADS

Floors, roofs, and other similar surfaces shall be designed to support safely the uniformly distributed live loads prescribed in Section 4.3 or the concentrated load, in pounds or kilonewtons (kN), given in Table 4-1, whichever produces the greater load effects. Unless otherwise specified, the indicated concentration shall be assumed to be uniformly distributed over an area 2.5 ft (762 mm) by 2.5 ft (762 mm) and shall be located so as to produce the maximum load effects in the members.

4.5 LOADS ON HANDRAIL, GUARDRAIL, GRAB BAR, VEHICLE BARRIER SYSTEMS, AND FIXED LADDERS

4.5.1 Loads on Handrail and Guardrail Systems

All handrail and guardrail systems shall be designed to resist a single concentrated load of 200 lb (0.89 kN) applied in any direction at any point on the handrail or top rail and to transfer this load through the supports to the structure to produce the maximum load effect on the element being considered.

Further, all handrail and guardrail systems shall be designed to resist a load of 50 lb/ft (pound-force per linear foot) (0.73 kN/m) applied in any direction along the handrail or top rail. This load need not be assumed to act concurrently with the load specified in the preceding paragraph, and this load need not be considered for the following occupancies:

1. One- and two-family dwellings.
2. Factory, industrial, and storage occupancies, in areas that are not accessible to the public and that serve an occupant load not greater than 50.

Intermediate rails (all those except the handrail), and panel fillers shall be designed to withstand a horizontally applied normal load of 50 lb (0.22 kN) on an area not to exceed 12 in. by 12 in. (305 mm by 305 mm) including openings and space between rails and located so as to produce the maximum load effects. Reactions due to this loading are not required to be superimposed with the loads specified in either preceding paragraph.

4.5.2 Loads on Grab Bar Systems

Grab bar systems shall be designed to resist a single concentrated load of 250 lb (1.11 kN) applied in any direction at any point on the grab bar to produce the maximum load effect.

4.5.3 Loads on Vehicle Barrier Systems

Vehicle barrier systems for passenger vehicles shall be designed to resist a single load of 6,000 lb (26.70 kN) applied horizontally in any direction to the barrier system, and shall have anchorages or attachments capable of transferring this load to the structure. For design of the system, the load shall be assumed to act at heights between 1 ft 6 in. (460 mm) and 2 ft 3 in. (686 mm) above the floor or ramp surface, selected to produce the maximum load effect. The load shall be applied on an area not to exceed 12 in. by 12 in. (305 mm by 305 mm) and located so as to produce the maximum load effects. This load is not required to act concurrently with any handrail or guardrail system loadings specified in Section 4.5.1. Vehicle barrier systems in garages accommodating trucks and buses shall be designed in accordance with *AASHTO LRFD Bridge Design Specifications*.

4.5.4 Loads on Fixed Ladders

The minimum design live load on fixed ladders with rungs shall be a single concentrated load of 300 lb (1.33 kN), and shall be applied at any point to produce the maximum load effect on the element being considered. The number and position of additional concentrated live load units shall be a minimum of 1 unit of 300 lb (1.33 kN) for every 10 ft (3.05 m) of ladder height.

Where rails of fixed ladders extend above a floor or platform at the top of the ladder, each side rail extension shall be designed to resist a single concentrated live load of 100 lb (0.445 kN) in any direction at any height up to the top of the side rail extension. Ship ladders with treads instead of rungs shall have minimum design loads as stairs, defined in Table 4-1.

4.6 IMPACT LOADS

4.6.1 General

The live loads specified in Sections 4.3 through 4.5 shall be assumed to include adequate allowance for ordinary impact conditions. Provision shall be made in the structural design for uses and loads that involve unusual vibration and impact forces.

4.6.2 Elevators

All elements subject to dynamic loads from elevators shall be designed for impact loads and deflection limits prescribed by ASME A17.1.

4.6.3 Machinery

For the purpose of design, the weight of machinery and moving loads shall be increased as follows to allow for impact: (1) light machinery, shaft- or motor-driven, 20 percent; and (2) reciprocating machinery or power-driven units, 50 percent. All percentages shall be increased where specified by the manufacturer.

4.7 REDUCTION IN LIVE LOADS

4.7.1 General

Except for roof uniform live loads, all other minimum uniformly distributed live loads, L_o in

Table 4-1, shall be permitted to be reduced in accordance with the requirements of Sections 4.7.2 through 4.7.6.

4.7.2 Reduction in Uniform Live Loads

Subject to the limitations of Sections 4.7.3 through 4.7.6, members for which a value of $K_{LL}A_T$ is 400 ft^2 (37.16 m^2) or more are permitted to be designed for a reduced live load in accordance with the following formula:

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \qquad (4.7\text{-}1)$$

In SI:

$$L = L_o\left(0.25 + \frac{4.57}{\sqrt{K_{LL}A_T}}\right)$$

where

L = reduced design live load per ft^2 (m^2) of area supported by the member
L_o = unreduced design live load per ft^2 (m^2) of area supported by the member (see Table 4-1)
K_{LL} = live load element factor (see Table 4-2)
A_T = tributary area in ft^2 (m^2)

L shall not be less than $0.50L_o$ for members supporting one floor and L shall not be less than $0.40L_o$ for members supporting two or more floors.

EXCEPTION: For structural members in one- and two-family dwellings supporting more than one floor load, the following floor live load reduction shall be permitted as an alternative to Eq. 4.7-1:

$$L = 0.7 \times (L_{o1} + L_{o2} + \ldots)$$

L_{o1}, L_{o2}, … are the unreduced floor live loads applicable to each of multiple supported story levels regardless of tributary area. The reduced floor live load effect, L, shall not be less than that produced by the effect of the largest unreduced floor live load on a given story level acting alone.

4.7.3 Heavy Live Loads

Live loads that exceed 100 lb/ft^2 (4.79 kN/m^2) shall not be reduced.

EXCEPTION: Live loads for members supporting two or more floors shall be permitted to be reduced by 20 percent.

4.7.4 Passenger Vehicle Garages

The live loads shall not be reduced in passenger vehicle garages.

EXCEPTION: Live loads for members supporting two or more floors shall be permitted to be reduced by 20 percent.

4.7.5 Assembly Uses

Live loads shall not be reduced in assembly uses.

4.7.6 Limitations on One-Way Slabs

The tributary area, A_T, for one-way slabs shall not exceed an area defined by the slab span times a width normal to the span of 1.5 times the slab span.

4.8 REDUCTION IN ROOF LIVE LOADS

4.8.1 General

The minimum uniformly distributed roof live loads, L_o in Table 4-1, are permitted to be reduced in accordance with the requirements of Sections 4.8.2 and 4.8.3.

4.8.2 Flat, Pitched, and Curved Roofs

Ordinary flat, pitched, and curved roofs, and awning and canopies other than those of fabric construction supported by a skeleton structure, are permitted to be designed for a reduced roof live load, as specified in Eq. 4.8-1 or other controlling combinations of loads, as specified in Chapter 2, whichever produces the greater load effect. In structures such as greenhouses, where special scaffolding is used as a work surface for workers and materials during maintenance and repair operations, a lower roof load than specified in Eq. 4.8-1 shall not be used unless approved by the authority having jurisdiction. On such structures, the minimum roof live load shall be 12 psf (0.58 kN/m^2).

$$L_r = L_oR_1R_2 \quad \text{where} \quad 12 \le L_r \le 20 \qquad (4.8\text{-}1)$$

In SI:

$$L_r = L_oR_1R_2 \quad \text{where} \quad 0.58 \le L_r \le 0.96$$

where

L_r = reduced roof live load per ft^2 (m^2) of horizontal projection supported by the member
L_o = unreduced design roof live load per ft^2 (m^2) of horizontal projection supported by the member (see Table 4-1)

The reduction factors R_1 and R_2 shall be determined as follows:

$$R_1 = \begin{array}{ll} 1 & \text{for } A_T \le 200 \text{ ft}^2 \\ 1.2 - 0.001A_t & \text{for } 200 \text{ ft}^2 < A_T < 600 \text{ ft}^2 \\ 0.6 & \text{for } A_T \ge 600 \text{ ft}^2 \end{array}$$

in SI:

$$R_1 = \begin{cases} 1 & \text{for } A_T \le 18.58\ \text{m}^2 \\ 1.2 - 0.011A_t & \text{for } 18.58\ \text{m}^2 < A_T < 55.74\ \text{m}^2 \\ 0.6 & \text{for } A_T \ge 55.74\ \text{m}^2 \end{cases}$$

where A_T = tributary area in ft^2 (m^2) supported by the member and

$$R_2 = \begin{cases} 1 & \text{for } F \le 4 \\ 1.2 - 0.05F & \text{for } 4 < F < 12 \\ 0.6 & \text{for } F \ge 12 \end{cases}$$

where, for a pitched roof, F = number of inches of rise per foot (in SI: $F = 0.12 \times$ slope, with slope expressed in percentage points) and, for an arch or dome, F = rise-to-span ratio multiplied by 32.

4.8.3 Special Purpose Roofs

Roofs that have an occupancy function, such as roof gardens, assembly purposes, or other special purposes are permitted to have their uniformly distributed live load reduced in accordance with the requirements of Section 4.7.

4.9 CRANE LOADS

4.9.1 General

The crane live load shall be the rated capacity of the crane. Design loads for the runway beams, including connections and support brackets, of moving bridge cranes and monorail cranes shall include the maximum wheel loads of the crane and the vertical impact, lateral, and longitudinal forces induced by the moving crane.

4.9.2 Maximum Wheel Load

The maximum wheel loads shall be the wheel loads produced by the weight of the bridge, as applicable, plus the sum of the rated capacity and the weight of the trolley with the trolley positioned on its runway at the location where the resulting load effect is maximum.

4.9.3 Vertical Impact Force

The maximum wheel loads of the crane shall be increased by the percentages shown in the following text to determine the induced vertical impact or vibration force:

Monorail cranes (powered)	25
Cab-operated or remotely operated bridge cranes (powered)	25
Pendant-operated bridge cranes (powered)	10
Bridge cranes or monorail cranes with hand-geared bridge, trolley, and hoist	0

4.9.4 Lateral Force

The lateral force on crane runway beams with electrically powered trolleys shall be calculated as 20 percent of the sum of the rated capacity of the crane and the weight of the hoist and trolley. The lateral force shall be assumed to act horizontally at the traction surface of a runway beam, in either direction perpendicular to the beam, and shall be distributed with due regard to the lateral stiffness of the runway beam and supporting structure.

4.9.5 Longitudinal Force

The longitudinal force on crane runway beams, except for bridge cranes with hand-geared bridges, shall be calculated as 10 percent of the maximum wheel loads of the crane. The longitudinal force shall be assumed to act horizontally at the traction surface of a runway beam in either direction parallel to the beam.

4.10 CONSENSUS STANDARDS AND OTHER REFERENCED DOCUMENTS

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

AASHTO
American Association of State Highway and Transportation Officials
444 North Capitol Street, NW, Suite 249
Washington, DC 20001

Sections 4.4.3, Table 4-1

AASHTO LRFD Bridge Design Specifications, 4th edition, 2007, with 2008 Interim Revisions
Sections 4.5.3, Table 4-1

ASME
American Society of Mechanical Engineers
Three Park Avenue
New York, NY 10016-5900

ASME A17.1
Section 4.6.2
American National Standard Safety Code for Elevators and Escalators, 2007.

Table 4-1 Minimum Uniformly Distributed Live Loads, L_o, and Minimum Concentrated Live Loads

Occupancy or Use	Uniform psf (kN/m^2)	Conc. lb (kN)
Apartments (see Residential)		
Access floor systems		
Office use	50 (2.4)	2,000 (8.9)
Computer use	100 (4.79)	2,000 (8.9)
Armories and drill rooms	150 (7.18)[a]	
Assembly areas and theaters		
Fixed seats (fastened to floor)	60 (2.87)[a]	
Lobbies	100 (4.79)[a]	
Movable seats	100 (4.79)[a]	
Platforms (assembly)	100 (4.79)[a]	
Stage floors	150 (7.18)[a]	
Balconies and decks	1.5 times the live load for the occupancy served. Not required to exceed 100 psf (4.79 kN/m^2)	
Catwalks for maintenance access	40 (1.92)	300 (1.33)
Corridors		
First floor	100 (4.79)	
Other floors, same as occupancy served except as indicated		
Dining rooms and restaurants	100 (4.79)[a]	
Dwellings (see Residential)		
Elevator machine room grating (on area of 2 in. by 2 in. (50 mm by 50 mm))		300 (1.33)
Finish light floor plate construction (on area of 1 in. by 1 in. (25 mm by 25 mm))		200 (0.89)
Fire escapes	100 (4.79)	
On single-family dwellings only	40 (1.92)	
Fixed ladders	See Section 4.5	
Garages		
Passenger vehicles only	40 (1.92)[a,b,c]	
Trucks and buses	[c]	
Handrails, guardrails, and grab bars	See Section 4.5	
Helipads	60 (2.87)[d,e] Nonreducible	[e,f,g]
Hospitals		
Operating rooms, laboratories	60 (2.87)	1,000 (4.45)
Patient rooms	40 (1.92)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Hotels (see Residential)		
Libraries		
Reading rooms	60 (2.87)	1,000 (4.45)
Stack rooms	150 (7.18)[a,h]	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Manufacturing		
Light	125 (6.00)[a]	2,000 (8.90)
Heavy	250 (11.97)[a]	3,000 (13.40)

Continued

Table 4-1 (*Continued*)

Occupancy or Use	Uniform psf (kN/m²)	Conc. lb (kN)
Office buildings		
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Lobbies and first-floor corridors	100 (4.79)	2,000 (8.90)
Offices	50 (2.40)	2,000 (8.90)
Corridors above first floor	80 (3.83)	2,000 (8.90)
Penal institutions		
Cell blocks	40 (1.92)	
Corridors	100 (4.79)	
Recreational uses		
Bowling alleys, poolrooms, and similar uses	75 (3.59)[a]	
Dance halls and ballrooms	100 (4.79)[a]	
Gymnasiums	100 (4.79)[a]	
Reviewing stands, grandstands, and bleachers	100 (4.79)[a,k]	
Stadiums and arenas with fixed seats (fastened to the floor)	60 (2.87)[a,k]	
Residential		
One- and two-family dwellings		
Uninhabitable attics without storage	10 (0.48)[l]	
Uninhabitable attics with storage	20 (0.96)[m]	
Habitable attics and sleeping areas	30 (1.44)	
All other areas except stairs	40 (1.92)	
All other residential occupancies		
Private rooms and corridors serving them	40 (1.92)	
Public rooms[a] and corridors serving them	100 (4.79)	
Roofs		
Ordinary flat, pitched, and curved roofs	20 (0.96)[n]	
Roofs used for roof gardens	100 (4.79)	
Roofs used for assembly purposes	Same as occupancy served	
Roofs used for other occupancies	[o]	[o]
Awnings and canopies		
Fabric construction supported by a skeleton structure	5 (0.24) nonreducible	300 (1.33) applied to skeleton structure
Screen enclosure support frame	5 (0.24) nonreducible and applied to the roof frame members only, not the screen	200 (0.89) applied to supporting roof frame members only
All other construction	20 (0.96)	
Primary roof members, exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages		2,000 (8.9)
All other primary roof members		300 (1.33)
All roof surfaces subject to maintenance workers		300 (1.33)
Schools		
Classrooms	40 (1.92)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
First-floor corridors	100 (4.79)	1,000 (4.45)
Scuttles, skylight ribs, and accessible ceilings		200 (0.89)
Sidewalks, vehicular driveways, and yards subject to trucking	250 (11.97)[a,p]	8,000 (35.60)[q]
Stairs and exit ways	100 (4.79)	300[r]
One- and two-family dwellings only	40 (1.92)	300[r]

Table 4-1 (*Continued*)

Occupancy or Use	Uniform psf (kN/m²)	Conc. lb (kN)
Storage areas above ceilings	20 (0.96)	
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Light	125 (6.00)[a]	
Heavy	250 (11.97)[a]	
Stores		
Retail		
First floor	100 (4.79)	1,000 (4.45)
Upper floors	75 (3.59)	1,000 (4.45)
Wholesale, all floors	125 (6.00)[a]	1,000 (4.45)
Vehicle barriers	See Section 4.5	
Walkways and elevated platforms (other than exit ways)	60 (2.87)	
Yards and terraces, pedestrian	100 (4.79)[a]	

[a]Live load reduction for this use is not permitted by Section 4.7 unless specific exceptions apply.

[b]Floors in garages or portions of a building used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 4-1 or the following concentrated load: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 lb (13.35 kN) acting on an area of 4.5 in. by 4.5 in. (114 mm by 114 mm); and (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 lb (10 kN) per wheel.

[c]Design for trucks and buses shall be per AASHTO LRFD Bridge Design Specifications; however, provisions for fatigue and dynamic load allowance are not required to be applied.

[d]Uniform load shall be 40 psf (1.92 kN/m²) where the design basis helicopter has a maximum take-off weight of 3,000 lbs (13.35 kN) or less. This load shall not be reduced.

[e]Labeling of helicopter capacity shall be as required by the authority having jurisdiction.

[f]Two single concentrated loads, 8 ft (2.44 m) apart shall be applied on the landing area (representing the helicopter's two main landing gear, whether skid type or wheeled type), each having a magnitude of 0.75 times the maximum take-off weight of the helicopter and located to produce the maximum load effect on the structural elements under consideration. The concentrated loads shall be applied over an area of 8 in. by 8 in. (200 mm by 200 mm) and shall not be concurrent with other uniform or concentrated live loads.

[g]A single concentrated load of 3,000 lbs (13.35 kN) shall be applied over an area 4.5 in. by 4.5 in. (114 mm by 114 mm), located so as to produce the maximum load effects on the structural elements under consideration. The concentrated load need not be assumed to act concurrently with other uniform or concentrated live loads.

[h]The loading applies to stack room floors that support nonmobile, double-faced library book stacks subject to the following limitations: (1) The nominal book stack unit height shall not exceed 90 in. (2,290 mm); (2) the nominal shelf depth shall not exceed 12 in. (305 mm) for each face; and (3) parallel rows of double-faced book stacks shall be separated by aisles not less than 36 in. (914 mm) wide.

[k]In addition to the vertical live loads, the design shall include horizontal swaying forces applied to each row of the seats as follows: 24 lb per linear ft of seat applied in a direction parallel to each row of seats and 10 lb per linear ft of seat applied in a direction perpendicular to each row of seats. The parallel and perpendicular horizontal swaying forces need not be applied simultaneously.

[l]Uninhabitable attic areas without storage are those where the maximum clear height between the joist and rafter is less than 42 in. (1,067 mm), or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 in. (1,067 mm) in height by 24 in. (610 mm) in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirement.

[m]Uninhabitable attic areas with storage are those where the maximum clear height between the joist and rafter is 42 in. (1,067 mm) or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 in. (1,067 mm) in height by 24 in. (610 mm) in width, or greater, within the plane of the trusses. At the trusses, the live load need only be applied to those portions of the bottom chords where both of the following conditions are met:

i. The attic area is accessible from an opening not less than 20 in. (508 mm) in width by 30 in. (762 mm) in length that is located where the clear height in the attic is a minimum of 30 in. (762 mm); and
ii. The slope of the truss bottom chord is no greater than 2 units vertical to 12 units horizontal (9.5% slope).

The remaining portions of the bottom chords shall be designed for a uniformly distributed nonconcurrent live load of not less than 10 lb/ft² (0.48 kN/m²).

[n]Where uniform roof live loads are reduced to less than 20 lb/ft² (0.96 kN/m²) in accordance with Section 4.8.1 and are applied to the design of structural members arranged so as to create continuity, the reduced roof live load shall be applied to adjacent spans or to alternate spans, whichever produces the greatest unfavorable load effect.

[o]Roofs used for other occupancies shall be designed for appropriate loads as approved by the authority having jurisdiction.

[p]Other uniform loads in accordance with an approved method, which contains provisions for truck loadings, shall also be considered where appropriate.

[q]The concentrated wheel load shall be applied on an area of 4.5 in. by 4.5 in. (114 mm by 114 mm).

[r]Minimum concentrated load on stair treads (on area of 2 in. by 2 in. [50 mm by 50 mm]) is to be applied nonconcurrent with the uniform load.

Table 4-2 Live Load Element Factor, K_{LL}

Element	K_{LL}[a]
Interior columns	4
Exterior columns without cantilever slabs	4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs	2
Edge beams without cantilever slabs	2
Interior beams	2
All other members not identified, including:	1
Edge beams with cantilever slabs	
Cantilever beams	
One-way slabs	
Two-way slabs	
Members without provisions for continuous shear transfer normal to their span	

[a]In lieu of the preceding values, K_{LL} is permitted to be calculated.

Chapter 5
FLOOD LOADS

5.1 GENERAL

The provisions of this section apply to buildings and other structures located in areas prone to flooding as defined on a flood hazard map.

5.2 DEFINITIONS

The following definitions apply to the provisions of this chapter:

APPROVED: Acceptable to the authority having jurisdiction.

BASE FLOOD: The flood having a 1 percent chance of being equaled or exceeded in any given year.

BASE FLOOD ELEVATION (BFE): The elevation of flooding, including wave height, having a 1 percent chance of being equaled or exceeded in any given year.

BREAKAWAY WALL: Any type of wall subject to flooding that is not required to provide structural support to a building or other structure and that is designed and constructed such that, under base flood or lesser flood conditions, it will collapse in such a way that: (1) it allows the free passage of floodwaters, and (2) it does not damage the structure or supporting foundation system.

COASTAL A-ZONE: An area within a special flood hazard area, landward of a V-Zone or landward of an open coast without mapped V-Zones. To be classified as a Coastal A-Zone, the principal source of flooding must be astronomical tides, storm surges, seiches, or tsunamis, not riverine flooding, and the potential for breaking wave heights greater than or equal to 1.5 ft (0.46 m) must exist during the base flood.

COASTAL HIGH HAZARD AREA (V-ZONE): An area within a Special Flood Hazard Area, extending from offshore to the inland limit of a primary frontal dune along an open coast, and any other area that is subject to high-velocity wave action from storms or seismic sources. This area is designated on Flood Insurance Rate Maps (FIRMs) as V, VE, VO, or V1-30.

DESIGN FLOOD: The greater of the following two flood events: (1) the Base Flood, affecting those areas identified as Special Flood Hazard Areas on the community's FIRM; or (2) the flood corresponding to the area designated as a Flood Hazard Area on a community's Flood Hazard Map or otherwise legally designated.

DESIGN FLOOD ELEVATION (DFE): The elevation of the design flood, including wave height, relative to the datum specified on a community's flood hazard map.

FLOOD HAZARD AREA: The area subject to flooding during the design flood.

FLOOD HAZARD MAP: The map delineating Flood Hazard Areas adopted by the authority having jurisdiction.

FLOOD INSURANCE RATE MAP (FIRM): An official map of a community on which the Federal Insurance and Mitigation Administration has delineated both special flood hazard areas and the risk premium zones applicable to the community.

SPECIAL FLOOD HAZARD AREA (AREA OF SPECIAL FLOOD HAZARD): The land in the floodplain subject to a 1 percent or greater chance of flooding in any given year. These areas are delineated on a community's FIRM as A-Zones (A, AE, A1-30, A99, AR, AO, or AH) or V-Zones (V, VE, VO, or V1-30).

5.3 DESIGN REQUIREMENTS

5.3.1 Design Loads

Structural systems of buildings or other structures shall be designed, constructed, connected, and anchored to resist flotation, collapse, and permanent lateral displacement due to action of flood loads associated with the design flood (see Section 5.3.3) and other loads in accordance with the load combinations of Chapter 2.

5.3.2 Erosion and Scour

The effects of erosion and scour shall be included in the calculation of loads on buildings and other structures in flood hazard areas.

5.3.3 Loads on Breakaway Walls

Walls and partitions required by ASCE/SEI 24 to break away, including their connections to the structure, shall be designed for the largest of the following loads acting perpendicular to the plane of the wall:

1. The wind load specified in Chapter 26.
2. The earthquake load specified in Chapter 12.
3. 10 psf (0.48 kN/m^2).

The loading at which breakaway walls are intended to collapse shall not exceed 20 psf (0.96 kN/m^2) unless the design meets the following conditions:

1. Breakaway wall collapse is designed to result from a flood load less than that which occurs during the base flood.
2. The supporting foundation and the elevated portion of the building shall be designed against collapse, permanent lateral displacement, and other structural damage due to the effects of flood loads in combination with other loads as specified in Chapter 2.

5.4 LOADS DURING FLOODING

5.4.1 Load Basis

In flood hazard areas, the structural design shall be based on the design flood.

5.4.2 Hydrostatic Loads

Hydrostatic loads caused by a depth of water to the level of the DFE shall be applied over all surfaces involved, both above and below ground level, except that for surfaces exposed to free water, the design depth shall be increased by 1 ft (0.30 m).

Reduced uplift and lateral loads on surfaces of enclosed spaces below the DFE shall apply only if provision is made for entry and exit of floodwater.

5.4.3 Hydrodynamic Loads

Dynamic effects of moving water shall be determined by a detailed analysis utilizing basic concepts of fluid mechanics.

EXCEPTION: Where water velocities do not exceed 10 ft/s (3.05 m/s), dynamic effects of moving water shall be permitted to be converted into equivalent hydrostatic loads by increasing the DFE for design purposes by an equivalent surcharge depth, d_h, on the headwater side and above the ground level only, equal to

$$d_h = \frac{aV^2}{2g} \tag{5.4-1}$$

where

V = average velocity of water in ft/s (m/s)
g = acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2)
a = coefficient of drag or shape factor (not less than 1.25)

The equivalent surcharge depth shall be added to the DFE design depth and the resultant hydrostatic pressures applied to, and uniformly distributed across, the vertical projected area of the building or structure that is perpendicular to the flow. Surfaces parallel to the flow or surfaces wetted by the tail water shall be subject to the hydrostatic pressures for depths to the DFE only.

5.4.4 Wave Loads

Wave loads shall be determined by one of the following three methods: (1) by using the analytical procedures outlined in this section, (2) by more advanced numerical modeling procedures, or (3) by laboratory test procedures (physical modeling).

Wave loads are those loads that result from water waves propagating over the water surface and striking a building or other structure. Design and construction of buildings and other structures subject to wave loads shall account for the following loads: waves breaking on any portion of the building or structure; uplift forces caused by shoaling waves beneath a building or structure, or portion thereof; wave runup striking any portion of the building or structure; wave-induced drag and inertia forces; and wave-induced scour at the base of a building or structure, or its foundation. Wave loads shall be included for both V-Zones and A-Zones. In V-Zones, waves are 3 ft (0.91 m) high, or higher; in coastal floodplains landward of the V-Zone, waves are less than 3 ft high (0.91 m).

Nonbreaking and broken wave loads shall be calculated using the procedures described in Sections 5.4.2 and 5.4.3 that show how to calculate hydrostatic and hydrodynamic loads.

Breaking wave loads shall be calculated using the procedures described in Sections 5.4.4.1 through 5.4.4.4. Breaking wave heights used in the procedures described in Sections 5.4.4.1 through 5.4.4.4 shall be calculated for V-Zones and Coastal A-Zones using Eqs. 5.4-2 and 5.4-3.

$$H_b = 0.78d_s \tag{5.4-2}$$

where

H_b = breaking wave height in ft (m)
d_s = local still water depth in ft (m)

The local still water depth shall be calculated using Eq. 5.4-3, unless more advanced procedures or laboratory tests permitted by this section are used.

$$d_s = 0.65(\mathrm{BFE} - \mathrm{G}) \qquad (5.4\text{-}3)$$

where

BFE = BFE in ft (m)
G = ground elevation in ft (m)

5.4.4.1 Breaking Wave Loads on Vertical Pilings and Columns

The net force resulting from a breaking wave acting on a rigid vertical pile or column shall be assumed to act at the still water elevation and shall be calculated by the following:

$$F_D = 0.5\gamma_w C_D D H_b^2 \qquad (5.4\text{-}4)$$

where

F_D = net wave force, in lb (kN)
γ_w = unit weight of water, in lb per cubic ft (kN/m^3), = 62.4 pcf (9.80 kN/m^3) for fresh water and 64.0 pcf (10.05 kN/m^3) for salt water
C_D = coefficient of drag for breaking waves, = 1.75 for round piles or columns and = 2.25 for square piles or columns
D = pile or column diameter, in ft (m) for circular sections, or for a square pile or column, 1.4 times the width of the pile or column in ft (m)
H_b = breaking wave height, in ft (m)

5.4.4.2 Breaking Wave Loads on Vertical Walls

Maximum pressures and net forces resulting from a normally incident breaking wave (depth-limited in size, with $H_b = 0.78d_s$) acting on a rigid vertical wall shall be calculated by the following:

$$P_{\max} = C_p\gamma_w d_s + 1.2\gamma_w d_s \qquad (5.4\text{-}5)$$

and

$$F_t = 1.1C_p\gamma_w d_s^2 + 2.4\gamma_w d_s^2 \qquad (5.4\text{-}6)$$

where

$P_{\max}$ = maximum combined dynamic ($C_p\gamma_w d_s$) and static ($1.2\gamma_w d_s$) wave pressures, also referred to as shock pressures in lb/ft^2 (kN/m^2)
F_t = net breaking wave force per unit length of structure, also referred to as shock, impulse, or wave impact force in lb/ft (kN/m), acting near the still water elevation
C_p = dynamic pressure coefficient ($1.6 < C_p < 3.5$) (see Table 5.4-1)

Table 5.4-1 Value of Dynamic Pressure Coefficient, C_p

Risk Category[a]	C_p
I	1.6
II	2.8
III	3.2
IV	3.5

[a]For Risk Category, see Table 1.5-1.

γ_w = unit weight of water, in lb per cubic ft (kN/m^3), = 62.4 pcf (9.80 kN/m^3) for fresh water and 64.0 pcf (10.05 kN/m^3) for salt water
d_s = still water depth in ft (m) at base of building or other structure where the wave breaks

This procedure assumes the vertical wall causes a reflected or standing wave against the waterward side of the wall with the crest of the wave at a height of $1.2d_s$ above the still water level. Thus, the dynamic static and total pressure distributions against the wall are as shown in Fig. 5.4-1.

This procedure also assumes the space behind the vertical wall is dry, with no fluid balancing the static component of the wave force on the outside of the wall. If free water exists behind the wall, a portion of the hydrostatic component of the wave pressure and force disappears (see Fig. 5.4-2) and the net force shall be computed by Eq. 5.4-7 (the maximum combined wave pressure is still computed with Eq. 5.4-5).

$$F_t = 1.1C_p\gamma_w d_s^2 + 1.9\gamma_w d_s^2 \qquad (5.4\text{-}7)$$

where

F_t = net breaking wave force per unit length of structure, also referred to as shock, impulse, or wave impact force in lb/ft (kN/m), acting near the still water elevation
C_p = dynamic pressure coefficient ($1.6 < C_p < 3.5$) (see Table 5.4-1)
γ_w = unit weight of water, in lb per cubic ft (kN/m^3), = 62.4 pcf (9.80 kN/m^3) for fresh water and 64.0 pcf (10.05 kN/m^3) for salt water
d_s = still water depth in ft (m) at base of building or other structure where the wave breaks

5.4.4.3 Breaking Wave Loads on Nonvertical Walls

Breaking wave forces given by Eqs. 5.4-6 and 5.4-7 shall be modified in instances where the walls or surfaces upon which the breaking waves act are

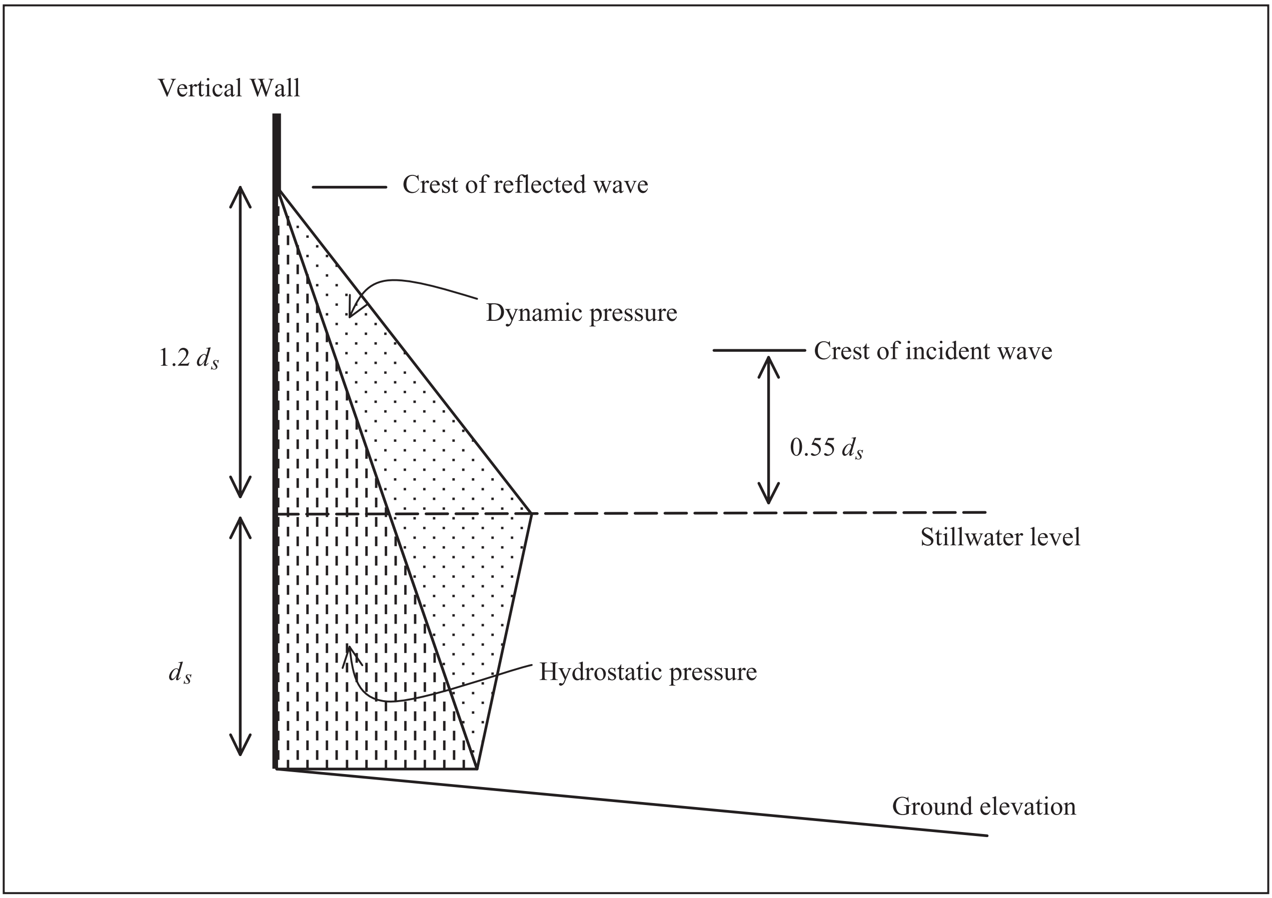


FIGURE 5.4-1 Normally Incident Breaking Wave Pressures against a Vertical Wall (Space behind Vertical Wall is Dry).

nonvertical. The horizontal component of breaking wave force shall be given by

$$F_{nv} = F_t \sin^2\alpha \qquad (5.4\text{-}8)$$

where

F_{nv} = horizontal component of breaking wave force in lb/ft (kN/m)
F_t = net breaking wave force acting on a vertical surface in lb/ft (kN/m)
α = vertical angle between nonvertical surface and the horizontal

5.4.4.4 Breaking Wave Loads from Obliquely Incident Waves

Breaking wave forces given by Eqs. 5.4-6 and 5.4-7 shall be modified in instances where waves are obliquely incident. Breaking wave forces from non-normally incident waves shall be given by

$$F_{oi} = F_t \sin^2\alpha \qquad (5.4\text{-}9)$$

where

F_{oi} = horizontal component of obliquely incident breaking wave force in lb/ft (kN/m)
F_t = net breaking wave force (normally incident waves) acting on a vertical surface in lb/ft (kN/m)
α = horizontal angle between the direction of wave approach and the vertical surface

5.4.5 Impact Loads

Impact loads are those that result from debris, ice, and any object transported by floodwaters striking against buildings and structures, or parts thereof. Impact loads shall be determined using a rational approach as concentrated loads acting horizontally at the most critical location at or below the DFE.

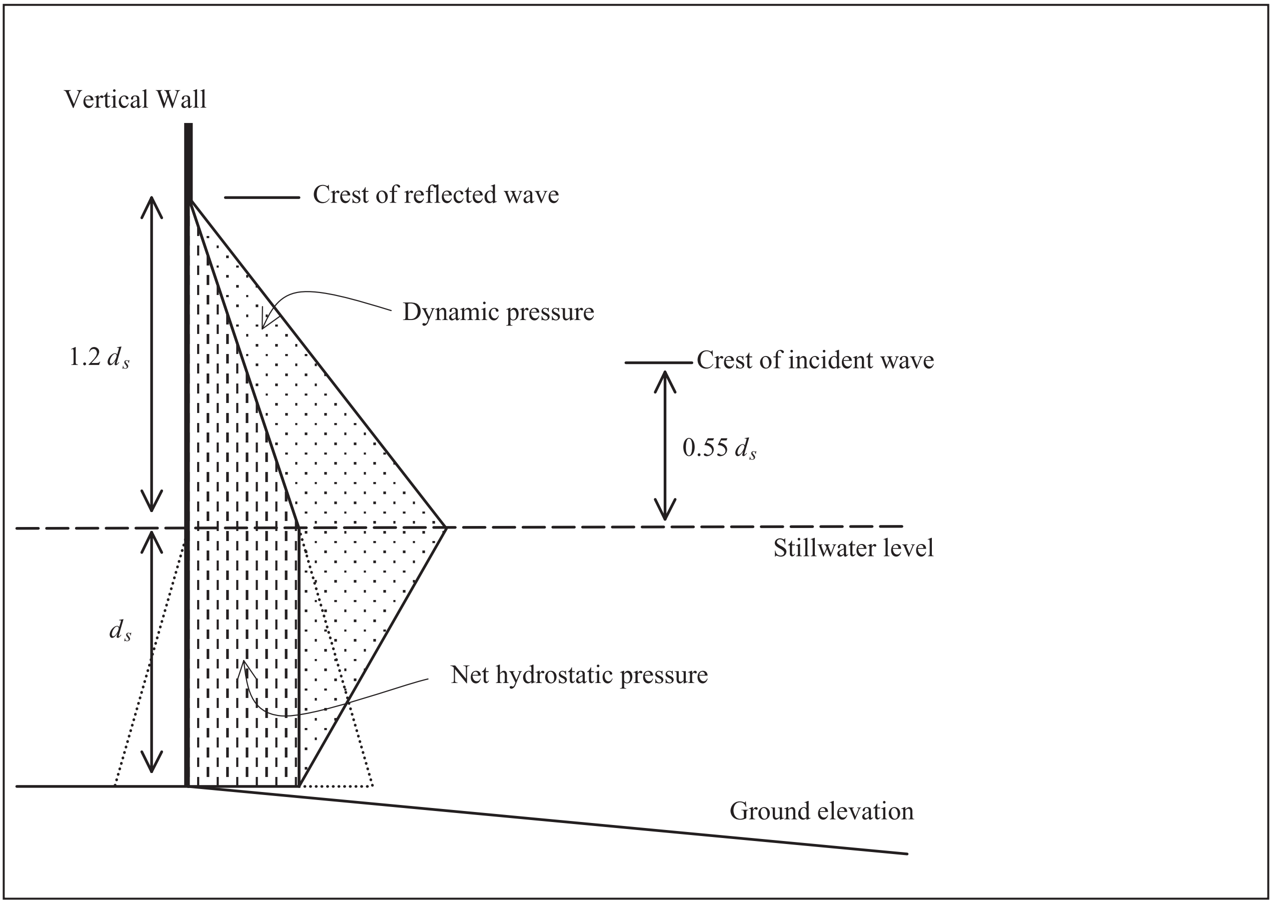


FIGURE 5.4-2 Normally Incident Breaking Wave Pressures against a Vertical Wall (Still Water Level Equal on Both Sides of Wall).

5.5 CONSENSUS STANDARDS AND OTHER REFERENCED DOCUMENTS

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

ASCE/SEI
American Society of Civil Engineers
Structural Engineering Institute
1801 Alexander Bell Drive
Reston, VA 20191-4400

ASCE/SEI 24
Section 5.3.3
Flood Resistant Design and Construction, 1998

Chapter 6
RESERVED FOR FUTURE PROVISIONS

In preparing the wind load provisions contained within this standard, the Wind Load Subcommittee (WLSC) of ASCE 7 established as one of its primary goals the improvement of the clarity and use of the standard. As a result of the efforts of the WLSC, the wind load provisions of ASCE 7 are presented in Chapters 26 through 31 as opposed to prior editions wherein the wind load provisions were contained in a single section (previously Chapter 6).

Chapter 7
SNOW LOADS

7.1 SYMBOLS

C_e = exposure factor as determined from Table 7-2
C_s = slope factor as determined from Fig. 7-2
C_t = thermal factor as determined from Table 7-3
h = vertical separation distance in feet (m) between the edge of a higher roof including any parapet and the edge of a lower adjacent roof excluding any parapet
h_b = height of balanced snow load determined by dividing p_s by γ, in ft (m)
h_c = clear height from top of balanced snow load to (1) closest point on adjacent upper roof, (2) top of parapet, or (3) top of a projection on the roof, in ft (m)
h_d = height of snow drift, in ft (m)
h_o = height of obstruction above the surface of the roof, in ft (m)
I_s = importance factor as prescribed in Section 7.3.3
l_u = length of the roof upwind of the drift, in ft (m)
p_d = maximum intensity of drift surcharge load, in lb/ft^2 (kN/m^2)
p_f = snow load on flat roofs ("flat" = roof slope ≤ 5°), in lb/ft^2 (kN/m^2)
p_g = ground snow load as determined from Fig. 7-1 and Table 7-1; or a site-specific analysis, in lb/ft^2 (kN/m^2)
p_m = minimum snow load for low-slope roofs, in lb/ft^2 (kN/m^2)
p_s = sloped roof (balanced) snow load, in lb/ft^2 (kN/m^2)
s = horizontal separation distance in feet (m) between the edges of two adjacent buildings
S = roof slope run for a rise of one
θ = roof slope on the leeward side, in degrees
w = width of snow drift, in ft (m)
W = horizontal distance from eave to ridge, in ft (m)
γ = snow density, in lb/ft^3 (kN/m^3) as determined from Eq. 7.7-1

7.2 GROUND SNOW LOADS, p_g

Ground snow loads, p_g, to be used in the determination of design snow loads for roofs shall be as set forth in Fig. 7-1 for the contiguous United States and Table 7-1 for Alaska. Site-specific case studies shall be made to determine ground snow loads in areas designated CS in Fig. 7-1. Ground snow loads for sites at elevations above the limits indicated in Fig. 7-1 and for all sites within the CS areas shall be approved by the authority having jurisdiction. Ground snow load determination for such sites shall be based on an extreme value statistical analysis of data available in the vicinity of the site using a value with a 2 percent annual probability of being exceeded (50-year mean recurrence interval).

Snow loads are zero for Hawaii, except in mountainous regions as determined by the authority having jurisdiction.

7.3 FLAT ROOF SNOW LOADS, p_f

The flat roof snow load, p_f, shall be calculated in lb/ft^2 (kN/m^2) using the following formula:

$$p_f = 0.7 C_e C_t I_s p_g \qquad (7.3\text{-}1)$$

7.3.1 Exposure Factor, C_e

The value for C_e shall be determined from Table 7-2.

7.3.2 Thermal Factor, C_t

The value for C_t shall be determined from Table 7-3.

7.3.3 Importance Factor, I_s

The value for I_s shall be determined from Table 1.5-2 based on the Risk Category from Table 1.5-1.

7.3.4 Minimum Snow Load for Low-Slope Roofs, p_m

A minimum roof snow load, p_m, shall only apply to monoslope, hip and gable roofs with slopes less than 15°, and to curved roofs where the vertical angle from the eaves to the crown is less than 10°. The minimum roof snow load for low-slope roofs shall be obtained using the following formula:

Where p_g is 20 lb/ft^2 (0.96 kN/m^2) or less:

$$p_m = I_s p_g \qquad \text{(Importance Factor times } p_g\text{)}$$

Where p_g exceeds 20 lb/ft^2 (0.96 kN/m^2):

$$p_m = 20\ (I_s) \qquad \text{(20 lb/ft}^2\text{ times Importance Factor)}$$

This minimum roof snow load is a separate uniform load case. It need not be used in determining

Table 7-1 Ground Snow Loads, p_g, for Alaskan Locations

Location	p_g lb/ft^2	p_g kN/m^2	Location	p_g lb/ft^2	p_g kN/m^2	Location	p_g lb/ft^2	p_g kN/m^2
Adak	30	1.4	Galena	60	2.9	Petersburg	150	7.2
Anchorage	50	2.4	Gulkana	70	3.4	St. Paul	40	1.9
Angoon	70	3.4	Homer	40	1.9	Seward	50	2.4
Barrow	25	1.2	Juneau	60	2.9	Shemya	25	1.2
Barter	35	1.7	Kenai	70	3.4	Sitka	50	2.4
Bethel	40	1.9	Kodiak	30	1.4	Talkeetna	120	5.8
Big Delta	50	2.4	Kotzebue	60	2.9	Unalakleet	50	2.4
Cold Bay	25	1.2	McGrath	70	3.4	Valdez	160	7.7
Cordova	100	4.8	Nenana	80	3.8	Whittier	300	14.4
Fairbanks	60	2.9	Nome	70	3.4	Wrangell	60	2.9
Fort Yukon	60	2.9	Palmer	50	2.4	Yakutat	150	7.2

Table 7-2 Exposure Factor, C_e

Terrain Category	Exposure of Roof[a] Fully Exposed	Partially Exposed	Sheltered
B (see Section 26.7)	0.9	1.0	1.2
C (see Section 26.7)	0.9	1.0	1.1
D (see Section 26.7)	0.8	0.9	1.0
Above the treeline in windswept mountainous areas.	0.7	0.8	N/A
In Alaska, in areas where trees do not exist within a 2-mile (3-km) radius of the site.	0.7	0.8	N/A

The terrain category and roof exposure condition chosen shall be representative of the anticipated conditions during the life of the structure. An exposure factor shall be determined for each roof of a structure.

[a]Definitions: Partially Exposed: All roofs except as indicated in the following text. Fully Exposed: Roofs exposed on all sides with no shelter[b] afforded by terrain, higher structures, or trees. Roofs that contain several large pieces of mechanical equipment, parapets that extend above the height of the balanced snow load (h_b), or other obstructions are not in this category. Sheltered: Roofs located tight in among conifers that qualify as obstructions.

[b]Obstructions within a distance of $10h_o$ provide "shelter," where h_o is the height of the obstruction above the roof level. If the only obstructions are a few deciduous trees that are leafless in winter, the "fully exposed" category shall be used. Note that these are heights above the roof. Heights used to establish the Exposure Category in Section 26.7 are heights above the ground.

Table 7-3 Thermal Factor, C_t

Thermal Condition[a]	C_t
All structures except as indicated below	1.0
Structures kept just above freezing and others with cold, ventilated roofs in which the thermal resistance (R-value) between the ventilated space and the heated space exceeds 25 °F × h × ft^2/Btu (4.4 K × m^2/W).	1.1
Unheated and open air structures	1.2
Structures intentionally kept below freezing	1.3
Continuously heated greenhouses[b] with a roof having a thermal resistance (R-value) less than 2.0 °F × h × ft^2/Btu (0.4 K × m^2/W)	0.85

[a]These conditions shall be representative of the anticipated conditions during winters for the life of the structure.

[b]Greenhouses with a constantly maintained interior temperature of 50 °F (10 °C) or more at any point 3 ft above the floor level during winters and having either a maintenance attendant on duty at all times or a temperature alarm system to provide warning in the event of a heating failure.

or in combination with drift, sliding, unbalanced, or partial loads.

7.4 SLOPED ROOF SNOW LOADS, p_s

Snow loads acting on a sloping surface shall be assumed to act on the horizontal projection of that surface. The sloped roof (balanced) snow load, p_s, shall be obtained by multiplying the flat roof snow load, p_f, by the roof slope factor, C_s:

$$p_s = C_s p_f \tag{7.4-1}$$

Values of C_s for warm roofs, cold roofs, curved roofs, and multiple roofs are determined from Sections 7.4.1 through 7.4.4. The thermal factor, C_t, from Table 7-3 determines if a roof is "cold" or "warm." "Slippery surface" values shall be used only where the roof's surface is unobstructed and sufficient space is available below the eaves to accept all the sliding snow. A roof shall be considered unobstructed if no objects exist on it that prevent snow on it from sliding. Slippery surfaces shall include metal, slate, glass, and bituminous, rubber, and plastic membranes with a smooth surface. Membranes with an imbedded aggregate or mineral granule surface shall not be considered smooth. Asphalt shingles, wood shingles, and shakes shall not be considered slippery.

7.4.1 Warm Roof Slope Factor, C_s

For warm roofs ($C_t \leq 1.0$ as determined from Table 7-3) with an unobstructed slippery surface that will allow snow to slide off the eaves, the roof slope factor C_s shall be determined using the dashed line in Fig. 7-2a, provided that for nonventilated warm roofs, their thermal resistance (R-value) equals or exceeds 30 ft^2 hr °F/Btu (5.3 °C m^2/W) and for warm ventilated roofs, their R-value equals or exceeds 20 ft^2 hr °F/Btu (3.5 °C m^2/W). Exterior air shall be able to circulate freely under a ventilated roof from its eaves to its ridge. For warm roofs that do not meet the aforementioned conditions, the solid line in Fig. 7-2a shall be used to determine the roof slope factor C_s.

7.4.2 Cold Roof Slope Factor, C_s

Cold roofs are those with a $C_t > 1.0$ as determined from Table 7-3. For cold roofs with $C_t = 1.1$ and an unobstructed slippery surface that will allow snow to slide off the eaves, the roof slope factor C_s shall be determined using the dashed line in Fig. 7-2b. For all other cold roofs with $C_t = 1.1$, the solid line in Fig. 7-2b shall be used to determine the roof slope factor C_s. For cold roofs with $C_t = 1.2$ and an unobstructed slippery surface that will allow snow to slide off the eaves, the roof slope factor C_s shall be determined using the dashed line on Fig. 7-2c. For all other cold roofs with $C_t = 1.2$, the solid line in Fig. 7-2c shall be used to determine the roof slope factor C_s.

7.4.3 Roof Slope Factor for Curved Roofs

Portions of curved roofs having a slope exceeding 70° shall be considered free of snow load (i.e., $C_s = 0$). Balanced loads shall be determined from the balanced load diagrams in Fig. 7-3 with C_s determined from the appropriate curve in Fig. 7-2.

7.4.4 Roof Slope Factor for Multiple Folded Plate, Sawtooth, and Barrel Vault Roofs

Multiple folded plate, sawtooth, or barrel vault roofs shall have a $C_s = 1.0$, with no reduction in snow load because of slope (i.e., $p_s = p_f$).

7.4.5 Ice Dams and Icicles Along Eaves

Two types of warm roofs that drain water over their eaves shall be capable of sustaining a uniformly distributed load of $2p_f$ on all overhanging portions: those that are unventilated and have an R-value less than 30 ft^2 hr °F/Btu (5.3 °C m^2/W) and those that are ventilated and have an R-value less than 20 ft^2 hr °F/Btu (3.5 °C m^2/W). The load on the overhang shall be based upon the flat roof snow load for the heated portion of the roof up-slope of the exterior wall. No other loads except dead loads shall be present on the roof when this uniformly distributed load is applied.

7.5 PARTIAL LOADING

The effect of having selected spans loaded with the balanced snow load and remaining spans loaded with half the balanced snow load shall be investigated as follows:

7.5.1 Continuous Beam Systems

Continuous beam systems shall be investigated for the effects of the three loadings shown in Fig. 7-4:

Case 1: Full balanced snow load on either exterior span and half the balanced snow load on all other spans.
Case 2: Half the balanced snow load on either exterior span and full balanced snow load on all other spans.
Case 3: All possible combinations of full balanced snow load on any two adjacent spans and half the balanced snow load on all other spans. For this case there will be (n –1) possible combinations where n equals the number of spans in the continuous beam system.

If a cantilever is present in any of the above cases, it shall be considered to be a span.

Partial load provisions need not be applied to structural members that span perpendicular to the ridgeline in gable roofs with slopes of 2.38˚ (½ on 12) and greater.

7.5.2 Other Structural Systems

Areas sustaining only half the balanced snow load shall be chosen so as to produce the greatest effects on members being analyzed.

7.6 UNBALANCED ROOF SNOW LOADS

Balanced and unbalanced loads shall be analyzed separately. Winds from all directions shall be accounted for when establishing unbalanced loads.

7.6.1 Unbalanced Snow Loads for Hip and Gable Roofs

For hip and gable roofs with a slope exceeding 7 on 12 (30.2°) or with a slope less than 2.38° (½ on 12) unbalanced snow loads are not required to be applied. Roofs with an eave to ridge distance, W, of 20 ft (6.1 m) or less, having simply supported prismatic members spanning from ridge to eave shall be designed to resist an unbalanced uniform snow load on the leeward side equal to Ip_g. For these roofs the windward side shall be unloaded. For all other gable roofs, the unbalanced load shall consist of $0.3p_s$ on the windward side, p_s on the leeward side plus a rectangular surcharge with magnitude $h_d\gamma/\sqrt{S}$ and horizontal extent from the ridge $8\sqrt{S}h_d/3$ where h_d is the drift height from Fig. 7-9 with l_u equal to the eave to ridge distance for the windward portion of the roof, W. For W less than 20 ft (6.1 m), use $W = l_u = 20$ ft in Fig 7-9. Balanced and unbalanced loading diagrams are presented in Fig. 7-5.

7.6.2 Unbalanced Snow Loads for Curved Roofs

Portions of curved roofs having a slope exceeding 70° shall be considered free of snow load. If the slope of a straight line from the eaves (or the 70° point, if present) to the crown is less than 10° or greater than 60°, unbalanced snow loads shall not be taken into account.

Unbalanced loads shall be determined according to the loading diagrams in Fig. 7-3. In all cases the windward side shall be considered free of snow. If the ground or another roof abuts a Case II or Case III (see Fig. 7-3) curved roof at or within 3 ft (0.91 m) of its eaves, the snow load shall not be decreased between the 30° point and the eaves, but shall remain constant at the 30° point value. This distribution is shown as a dashed line in Fig. 7-3.

7.6.3 Unbalanced Snow Loads for Multiple Folded Plate, Sawtooth, and Barrel Vault Roofs

Unbalanced loads shall be applied to folded plate, sawtooth, and barrel-vaulted multiple roofs with a slope exceeding 3/8 in./ft (1.79°). According to Section 7.4.4, $C_s = 1.0$ for such roofs, and the balanced snow load equals p_f. The unbalanced snow load shall increase from one-half the balanced load at the ridge or crown (i.e., $0.5p_f$) to two times the balanced load given in Section 7.4.4 divided by C_e at the valley (i.e., $2p_f/C_e$). Balanced and unbalanced loading diagrams for a sawtooth roof are presented in Fig. 7-6. However, the snow surface above the valley shall not be at an elevation higher than the snow above the ridge. Snow depths shall be determined by dividing the snow load by the density of that snow from Eq. 7.7-1, which is in Section 7.7.1.

7.6.4 Unbalanced Snow Loads for Dome Roofs

Unbalanced snow loads shall be applied to domes and similar rounded structures. Snow loads, determined in the same manner as for curved roofs in Section 7.6.2, shall be applied to the downwind 90° sector in plan view. At both edges of this sector, the load shall decrease linearly to zero over sectors of 22.5° each. There shall be no snow load on the remaining 225° upwind sector.

7.7 DRIFTS ON LOWER ROOFS (AERODYNAMIC SHADE)

Roofs shall be designed to sustain localized loads from snowdrifts that form in the wind shadow of
(1) higher portions of the same structure and
(2) adjacent structures and terrain features.

7.7.1 Lower Roof of a Structure

Snow that forms drifts comes from a higher roof or, with the wind from the opposite direction, from the roof on which the drift is located. These two kinds of drifts (“leeward” and “windward” respectively) are shown in Fig. 7-7. The geometry of the surcharge load due to snow drifting shall be approximated by a triangle as shown in Fig. 7-8. Drift loads shall be superimposed on the balanced snow load. If h_c/h_b is less than 0.2, drift loads are not required to be applied.

For leeward drifts, the drift height h_d shall be determined directly from Fig. 7-9 using the length of the upper roof. For windward drifts, the drift height shall be determined by substituting the length of the lower roof for l_u in Fig. 7-9 and using three-quarters of h_d as determined from Fig. 7-9 as the drift height. The larger of these two heights shall be used in design. If this height is equal to or less than h_c, the drift width, w, shall equal $4h_d$ and the drift height shall equal h_d. If this height exceeds h_c, the drift width, w, shall equal $4h_d^2/h_c$ and the drift height shall equal h_c. However, the drift width, w, shall not be greater than $8h_c$. If the drift width, w, exceeds the width of the lower roof, the drift shall be truncated at the far edge of the roof, not reduced to zero there. The maximum intensity of the drift surcharge load, p_d, equals $h_d\gamma$ where snow density, γ, is defined in Eq. 7.7-1:

$$\gamma = 0.13p_g + 14 \text{ but not more than 30 pcf} \quad (7.7\text{-}1)$$

(in SI: $\gamma = 0.426p_g + 2.2$, but not more than 4.7 kN/m^3)

This density shall also be used to determine h_b by dividing p_s by γ (in SI: also multiply by 102 to get the depth in m).

7.7.2 Adjacent Structures

If the horizontal separation distance between adjacent structures, s, is less than 20 ft (6.1 m) and less than six times the vertical separation distance ($s < 6h$), then the requirements for the leeward drift of Section 7.7.1 shall be used to determine the drift load on the lower structure. The height of the snow drift shall be the smaller of h_d, based upon the length of the adjacent higher structure, and $(6h - s)/6$. The horizontal extent of the drift shall be the smaller of $6h_d$ or $(6h - s)$.

For windward drifts, the requirements of Section 7.7.1 shall be used. The resulting drift is permitted to be truncated.

7.8 ROOF PROJECTIONS AND PARAPETS

The method in Section 7.7.1 shall be used to calculate drift loads on all sides of roof projections and at parapet walls. The height of such drifts shall be taken as three-quarters the drift height from Fig. 7-9 (i.e., $0.75h_d$). For parapet walls, l_u shall be taken equal to the length of the roof upwind of the wall. For roof projections, l_u shall be taken equal to the greater of the length of the roof upwind or downwind of the projection. If the side of a roof projection is less than 15 ft (4.6 m) long, a drift load is not required to be applied to that side.

7.9 SLIDING SNOW

The load caused by snow sliding off a sloped roof onto a lower roof shall be determined for slippery upper roofs with slopes greater than ¼ on 12, and for other (i.e., nonslippery) upper roofs with slopes greater than 2 on 12. The total sliding load per unit length of eave shall be $0.4p_fW$, where W is the horizontal distance from the eave to ridge for the sloped upper roof. The sliding load shall be distributed uniformly on the lower roof over a distance of 15 ft (4.6 m) from the upper roof eave. If the width of the lower roof is less than 15 ft (4.6 m), the sliding load shall be reduced proportionally.

The sliding snow load shall not be further reduced unless a portion of the snow on the upper roof is blocked from sliding onto the lower roof by snow already on the lower roof.

For separated structures, sliding loads shall be considered when $h/s > 1$ and $s < 15$ ft (4.6 m). The horizontal extent of the sliding load on the lower roof shall be $15 - s$ with s in feet ($4.6 - s$ with s in meters), and the load per unit length shall be $0.4\, p_f W (15 - s)/15$ with s in feet ($0.4p_fW\,(4.6 - s)/4.6$ with s in meters).

Sliding loads shall be superimposed on the balanced snow load and need not be used in combination with drift, unbalanced, partial, or rain-on-snow loads.

7.10 RAIN-ON-SNOW SURCHARGE LOAD

For locations where p_g is 20 lb/ft^2 (0.96 kN/m^2) or less, but not zero, all roofs with slopes (in degrees) less than $W/50$ with W in ft (in SI: $W/15.2$ with W in m) shall include a 5 lb/ft^2 (0.24 kN/m^2) rain-on-snow surcharge load. This additional load applies only to the sloped roof (balanced) load case and need not be used in combination with drift, sliding, unbalanced, minimum, or partial loads.

7.11 PONDING INSTABILITY

Roofs shall be designed to preclude ponding instability. For roofs with a slope less than ¼ in./ft (1.19°) and roofs where water can be impounded, roof deflections caused by full snow loads shall be evaluated when determining the likelihood of ponding instability (see Section 8.4).

7.12 EXISTING ROOFS

Existing roofs shall be evaluated for increased snow loads caused by additions or alterations. Owners or agents for owners of an existing lower roof shall be advised of the potential for increased snow loads where a higher roof is constructed within 20 ft (6.1 m). See footnote to Table 7-2 and Section 7.7.2.

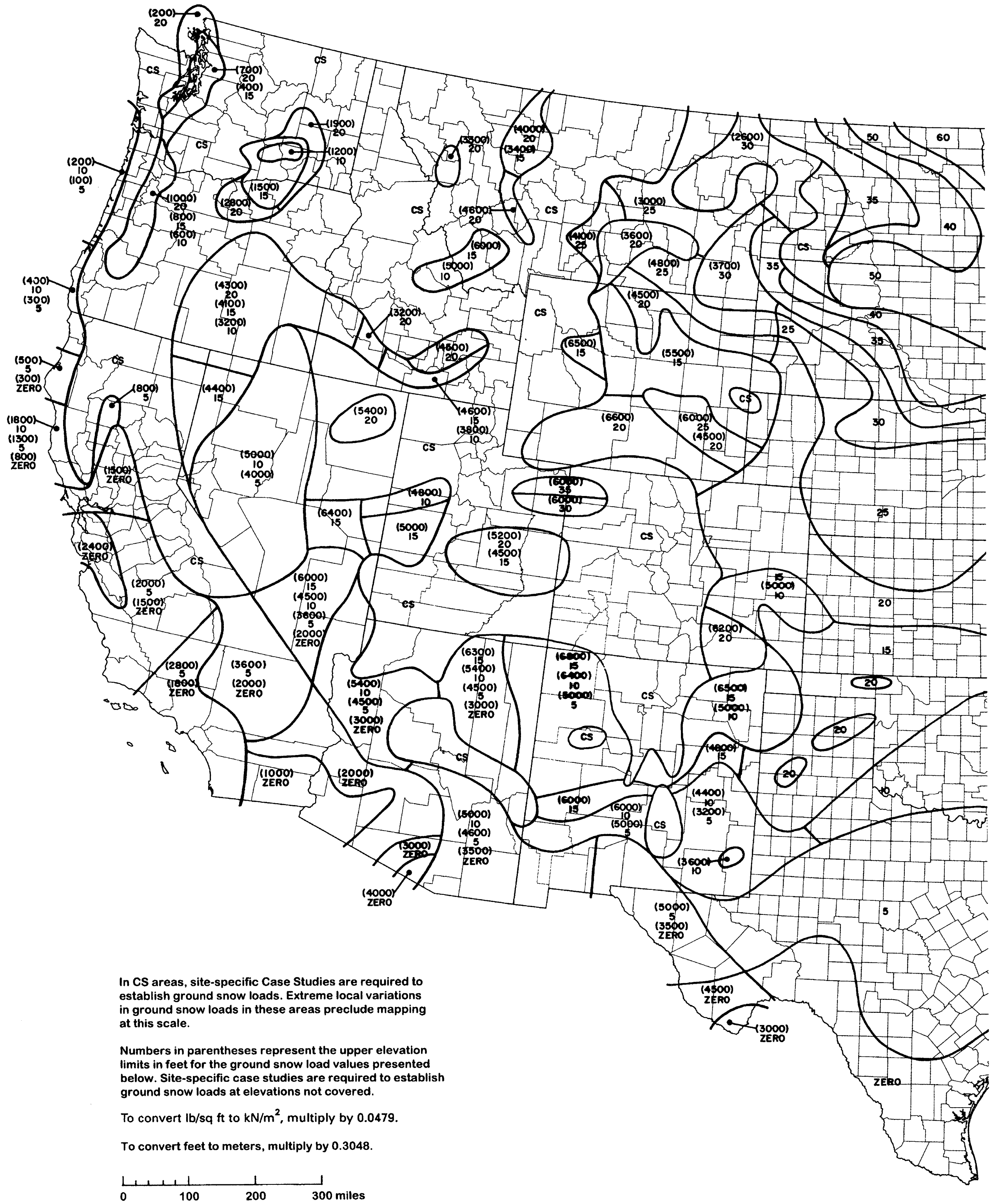


FIGURE 7-1 Ground Snow Loads, P_g, for the United States (Lb/Ft^2).

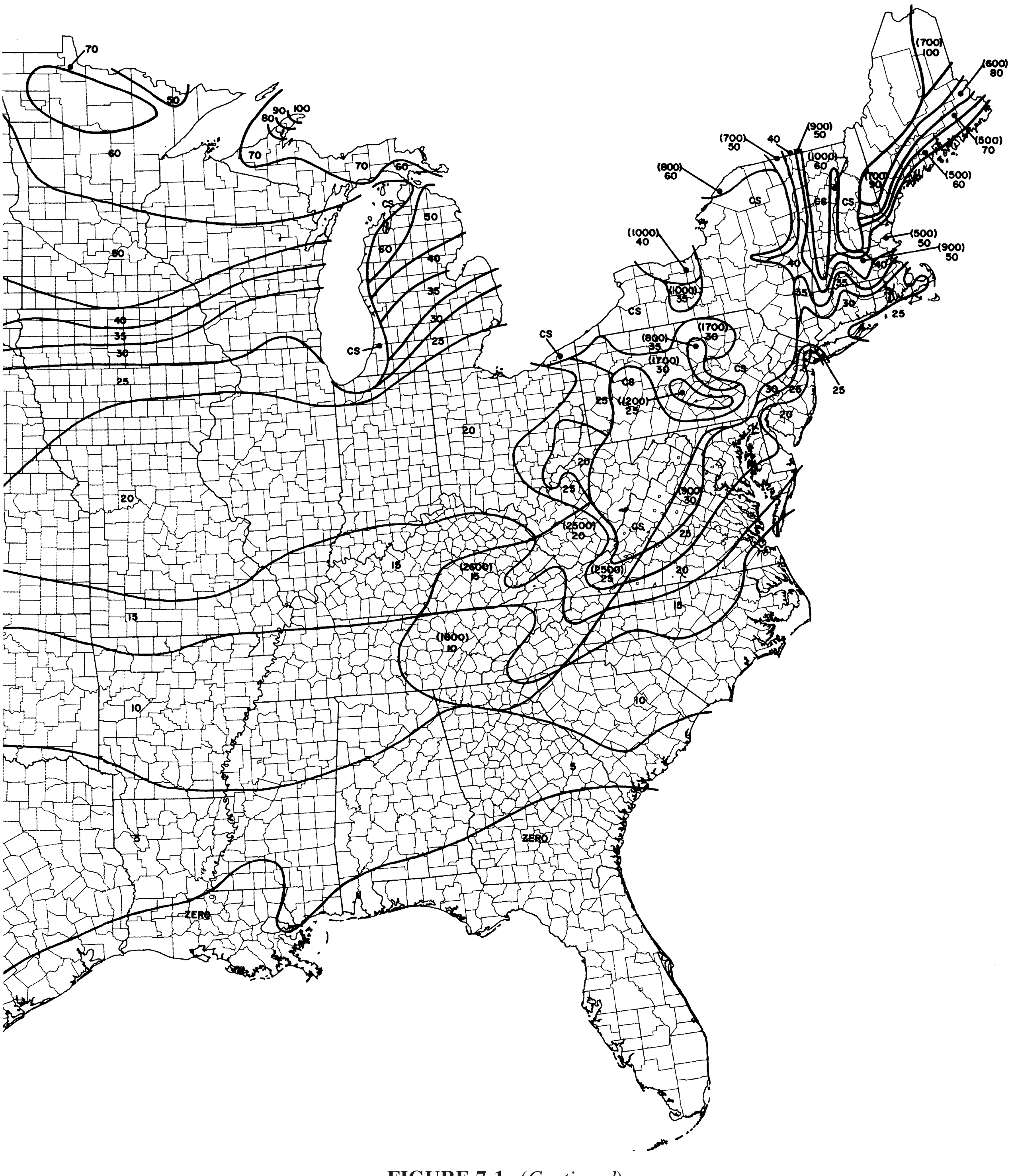
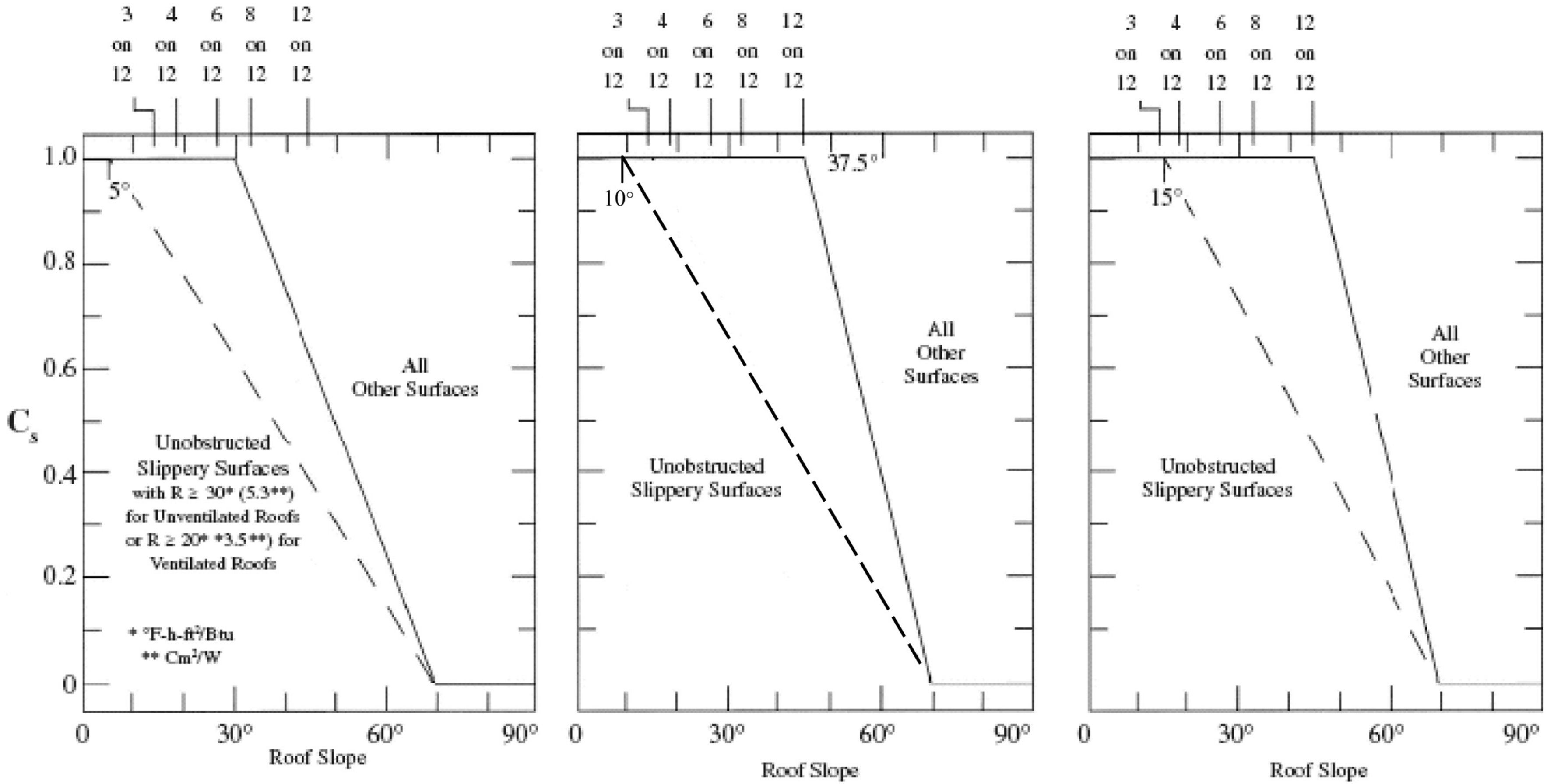


FIGURE 7-1. (*Continued*)



7-2a: Warm roofs with $C_t \leq 1.0$

7-2b: Cold roofs with $C_t = 1.1$

7-2c: Cold roofs with C = 1.2 or larger

FIGURE 7-2 Graphs for Determining Roof Slope Factor C_s, for Warm and Cold Roofs (See Table 7-3 for C_t Definitions).

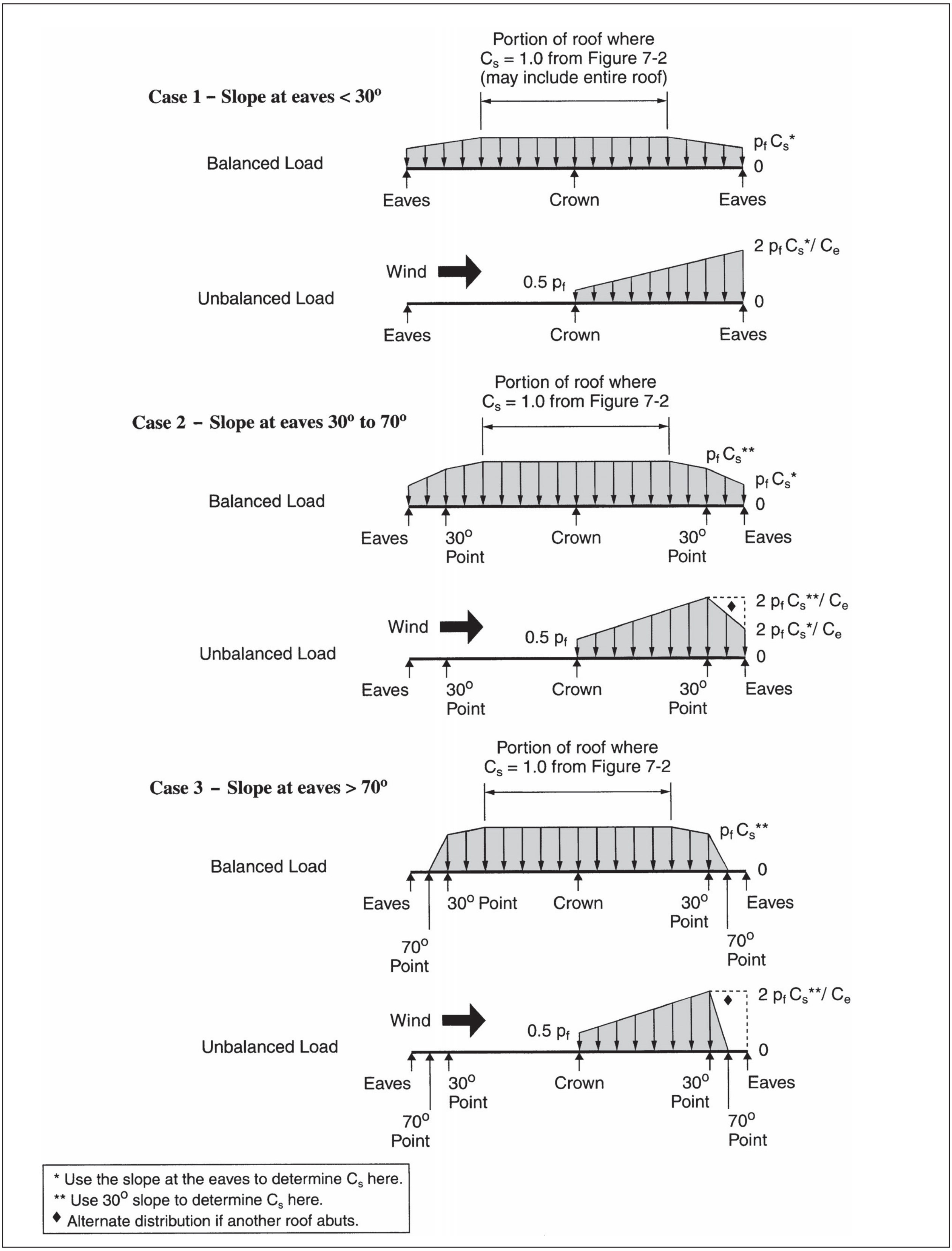
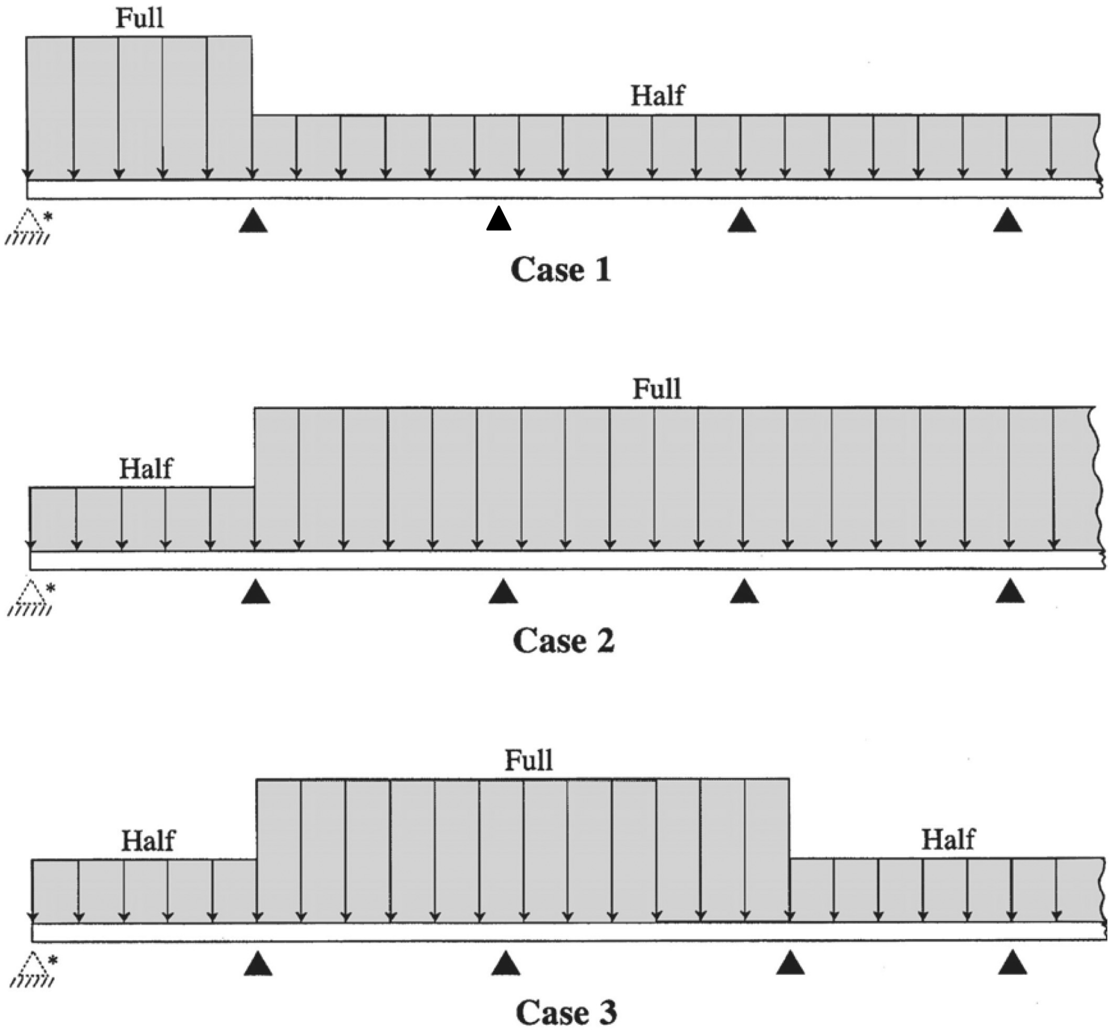


FIGURE 7-3 Balanced and Unbalanced Loads for Curved Roofs.



* The left supports are dashed since they would not exist when a cantilever is present.

FIGURE 7-4 Partial Loading Diagrams for Continuous Beams.

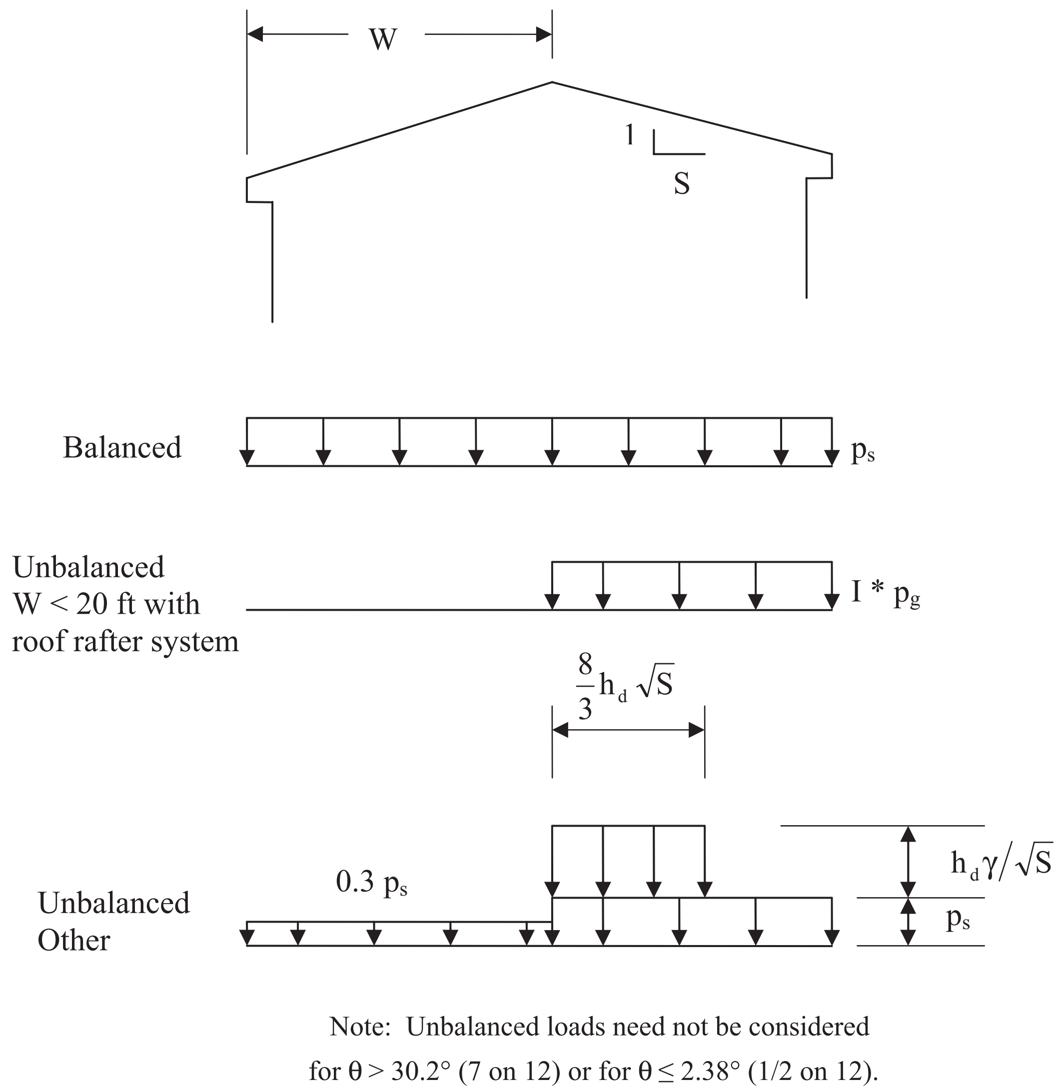


FIGURE 7-5 Balanced and Unbalanced Snow Loads for Hip and Gable Roofs.

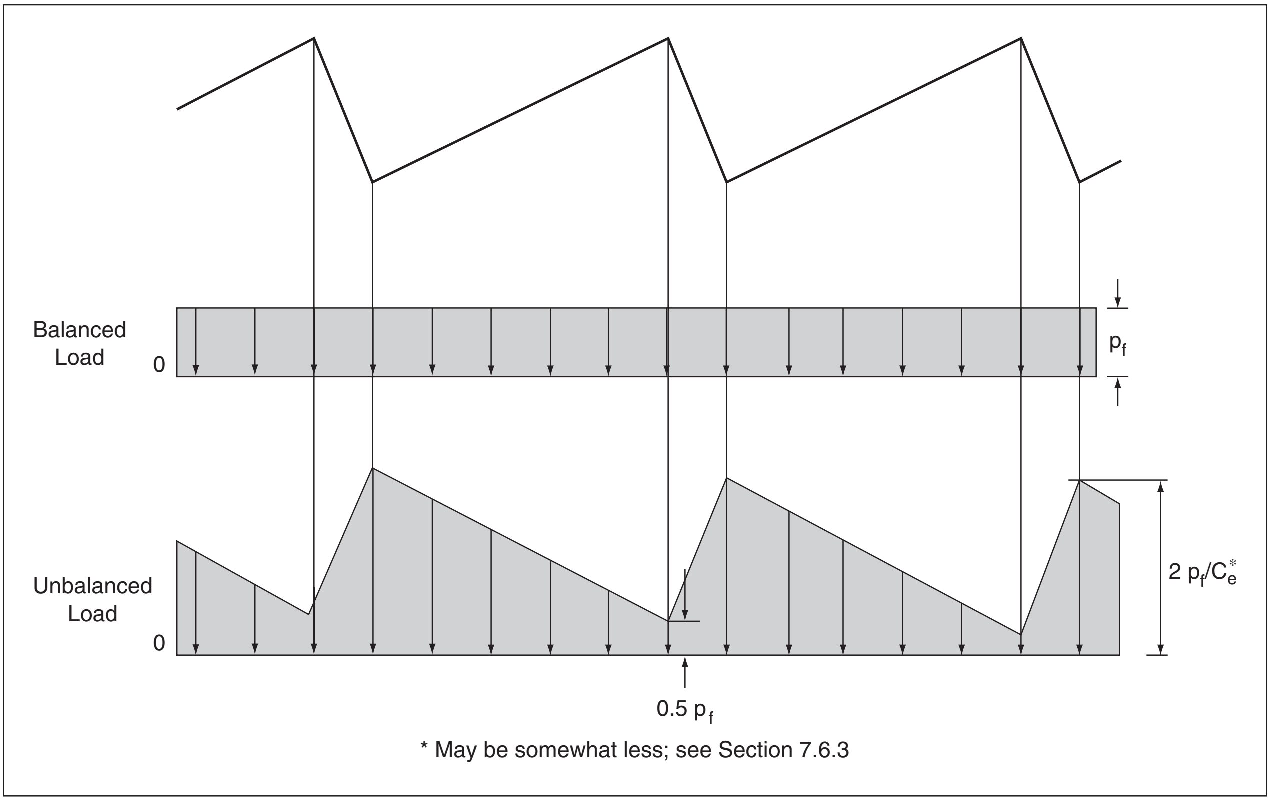


FIGURE 7-6 Balanced and Unbalanced Snow Loads for a Sawtooth Roof.

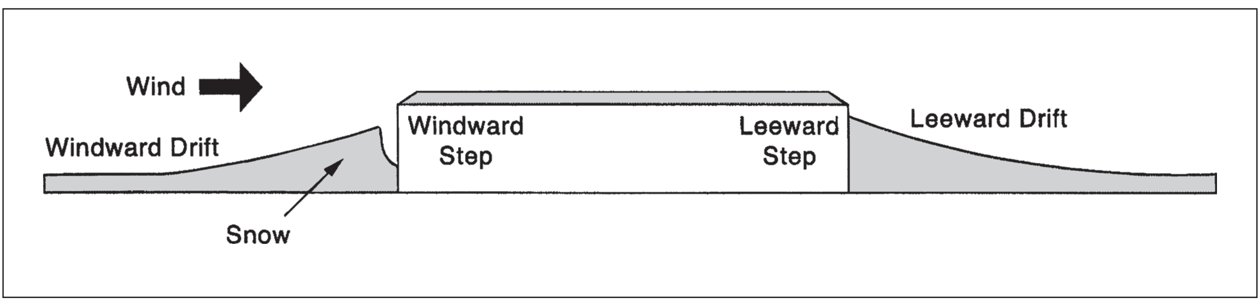


FIGURE 7-7 Drifts Formed at Windward and Leeward Steps.

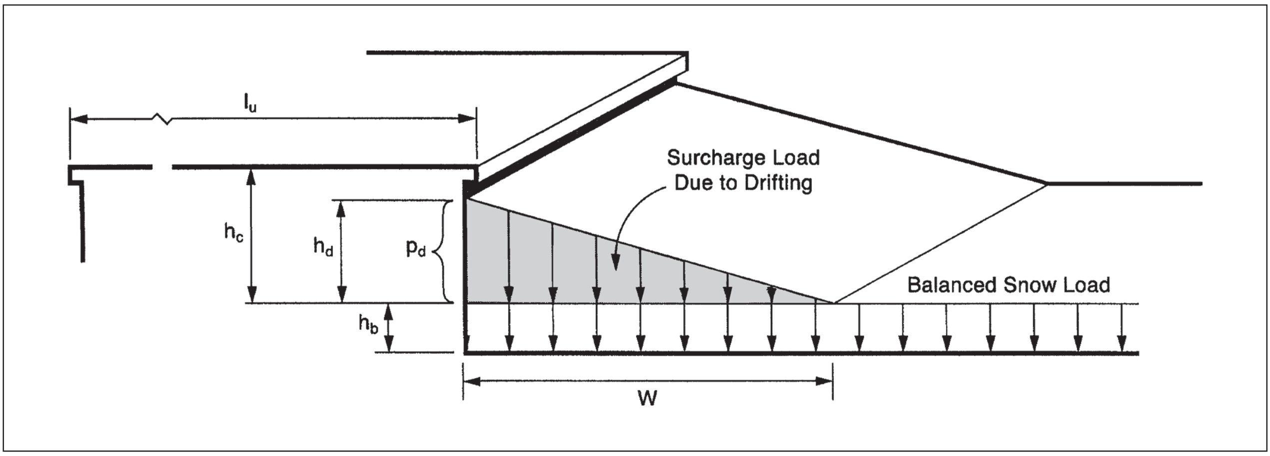


FIGURE 7-8 Configuration of Snow Drifts on Lower Roofs.

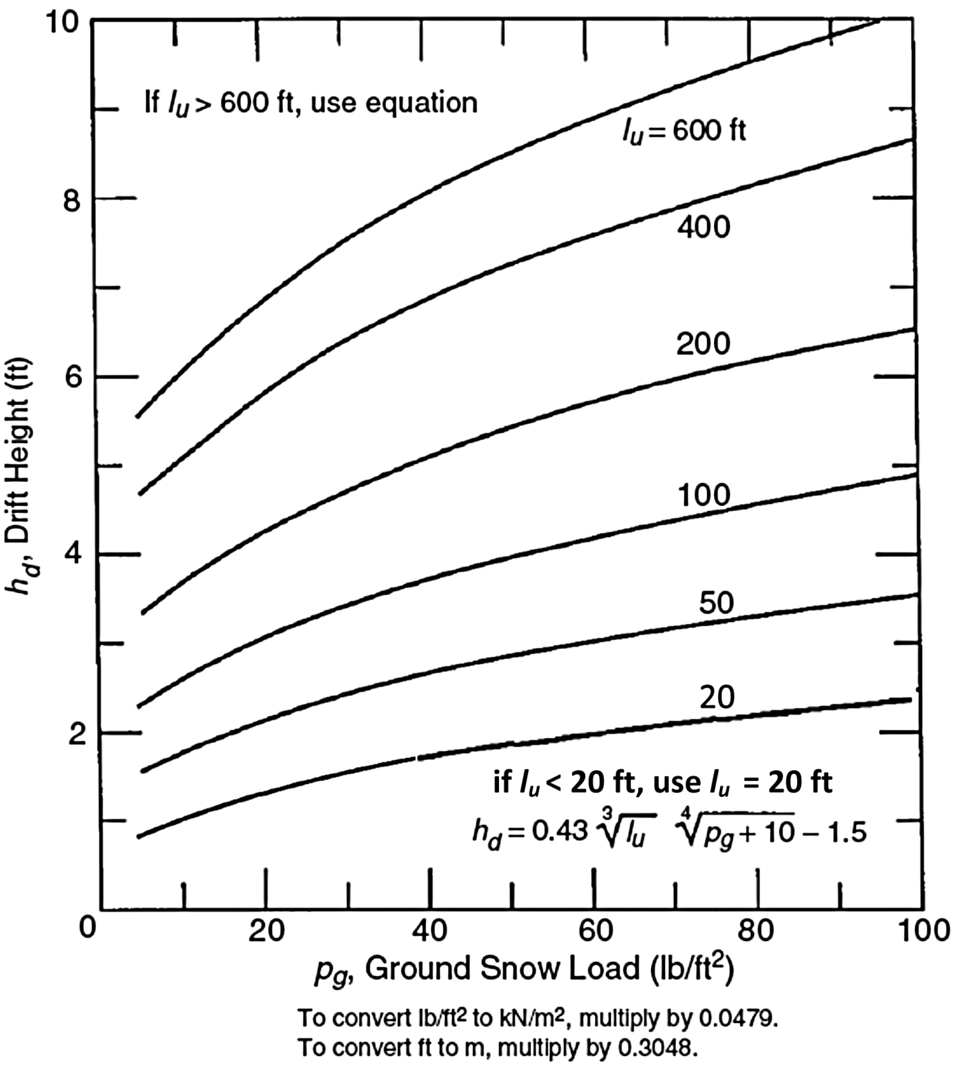


FIGURE 7-9 Graph and Equation for Determining Drift Height, h_d.

Chapter 8
RAIN LOADS

8.1 SYMBOLS

R = rain load on the undeflected roof, in lb/ft^2 (kN/m^2). When the phrase "undeflected roof" is used, deflections from loads (including dead loads) shall not be considered when determining the amount of rain on the roof.

d_s = depth of water on the undeflected roof up to the inlet of the secondary drainage system when the primary drainage system is blocked (i.e., the static head), in in. (mm).

d_h = additional depth of water on the undeflected roof above the inlet of the secondary drainage system at its design flow (i.e., the hydraulic head), in in. (mm).

8.2 ROOF DRAINAGE

Roof drainage systems shall be designed in accordance with the provisions of the code having jurisdiction. The flow capacity of secondary (overflow) drains or scuppers shall not be less than that of the primary drains or scuppers.

8.3 DESIGN RAIN LOADS

Each portion of a roof shall be designed to sustain the load of all rainwater that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow.

$$R = 5.2(d_s + d_h) \qquad (8.3\text{-}1)$$

In SI: $R = 0.0098(d_s + d_h)$

If the secondary drainage systems contain drain lines, such lines and their point of discharge shall be separate from the primary drain lines.

8.4 PONDING INSTABILITY

"Ponding" refers to the retention of water due solely to the deflection of relatively flat roofs. Susceptible bays shall be investigated by structural analysis to assure that they possess adequate stiffness to preclude progressive deflection (i.e., instability) as rain falls on them or meltwater is created from snow on them. Bays with a roof slope less than 1/4 in./ft., or on which water is impounded upon them (in whole or in part) when the primary drain system is blocked, but the secondary drain system is functional, shall be designated as susceptible bays. Roof surfaces with a slope of at least 1/4 in. per ft (1.19°) towards points of free drainage need not be considered a susceptible bay. The larger of the snow load or the rain load equal to the design condition for a blocked primary drain system shall be used in this analysis.

8.5 CONTROLLED DRAINAGE

Roofs equipped with hardware to control the rate of drainage shall be equipped with a secondary drainage system at a higher elevation that limits accumulation of water on the roof above that elevation. Such roofs shall be designed to sustain the load of all rainwater that will accumulate on them to the elevation of the secondary drainage system plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow (determined from Section 8.3).

Such roofs shall also be checked for ponding instability (determined from Section 8.4).

Chapter 9
RESERVED FOR FUTURE PROVISIONS

In preparing the seismic provisions contained within this standard, the Seismic Task Committee of ASCE 7 established a Scope and Format Subcommittee to review the layout and presentation of the seismic provisions and to make recommendations to improve the clarity and use of the standard. As a result of the efforts of this subcommittee, the seismic provisions of ASCE 7 are presented in Chapters 11 through 23 and Appendices 11A and 11B, as opposed to prior editions wherein the seismic provisions were presented in a single section (previously Section 9).

Of foremost concern in the reformat effort was the organization of the seismic provisions in a logical sequence for the general structural design community and the clarification of the various headings to more accurately reflect their content. Accomplishing these two primary goals led to the decision to create 13 separate sections and to relocate provisions into their most logical new sections.

The provisions for buildings and nonbuilding structures are now distinctly separate as are the provisions for nonstructural components. Less commonly used provisions, such as those for seismically isolated structures, have also been located in their own distinct chapter. We hope that the users of ASCE 7 will find the reformatted seismic provisions to be a significant improvement in organization and presentation over prior editions and will be able to more quickly locate applicable provisions. Table C11-1, located in Commentary Chapter C11 of the 2005 edition of ASCE 7 was provided to assist users in locating provisions between the 2002 and 2005 editions of the standard. Table C11-1 is not included in this edition of the standard.

Chapter 10
ICE LOADS—ATMOSPHERIC ICING

10.1 GENERAL

Atmospheric ice loads due to freezing rain, snow, and in-cloud icing shall be considered in the design of ice-sensitive structures. In areas where records or experience indicate that snow or in-cloud icing produces larger loads than freezing rain, site-specific studies shall be used. Structural loads due to hoarfrost are not a design consideration. Roof snow loads are covered in Chapter 7.

10.1.1 Site-Specific Studies

Mountainous terrain and gorges shall be examined for unusual icing conditions. Site-specific studies shall be used to determine the 50-year mean recurrence interval ice thickness, concurrent wind speed, and concurrent temperature in

1. Alaska.
2. Areas where records or experience indicate that snow or in-cloud icing produces larger loads than freezing rain.
3. Special icing regions shown in Figs. 10-2, 10-4, and 10-5.
4. Mountainous terrain and gorges where examination indicates unusual icing conditions exist.

Site-specific studies shall be subject to review and approval by the authority having jurisdiction.

In lieu of using the mapped values, it shall be permitted to determine the ice thickness, the concurrent wind speed, and the concurrent temperature for a structure from local meteorological data based on a 50-year mean recurrence interval provided that

1. The quality of the data for wind and type and amount of precipitation has been taken into account.
2. A robust ice accretion algorithm has been used to estimate uniform ice thicknesses and concurrent wind speeds from these data.
3. Extreme-value statistical analysis procedures acceptable to the authority having jurisdiction have been employed in analyzing the ice thickness and concurrent wind speed data.
4. The length of record and sampling error have been taken into account.

10.1.2 Dynamic Loads

Dynamic loads, such as those resulting from galloping, ice shedding, and aeolian vibrations, that are caused or enhanced by an ice accretion on a flexible structural member, component, or appurtenance are not covered in this section.

10.1.3 Exclusions

Electric transmission systems, communications towers and masts, and other structures for which national standards exist are excluded from the requirements of this section. Applicable standards and guidelines include the NESC, ASCE Manual 74, and ANSI/EIA/TIA-222.

10.2 DEFINITIONS

The following definitions apply only to the provisions of this chapter.

COMPONENTS AND APPURTENANCES: Nonstructural elements that may be exposed to atmospheric icing. Examples are ladders, handrails, antennas, waveguides, Radio Frequency (RF) transmission lines, pipes, electrical conduits, and cable trays.

FREEZING RAIN: Rain or drizzle that falls into a layer of subfreezing air at the earth's surface and freezes on contact with the ground or an object to form glaze ice.

GLAZE: Clear high-density ice.

HOARFROST: An accumulation of ice crystals formed by direct deposition of water vapor from the air onto an object.

ICE-SENSITIVE STRUCTURES: Structures for which the effect of an atmospheric icing load governs the design of part or all of the structure. This includes, but is not limited to, lattice structures, guyed masts, overhead lines, light suspension and cable-stayed bridges, aerial cable systems (e.g., for ski lifts and logging operations), amusement rides, open catwalks and platforms, flagpoles, and signs.

IN-CLOUD ICING: Occurs when supercooled cloud or fog droplets carried by the wind freeze on impact with objects. In-cloud icing usually forms rime, but may also form glaze.

RIME: White or opaque ice with entrapped air.

SNOW: Snow that adheres to objects by some combination of capillary forces, freezing, and sintering.

10.3 SYMBOLS

A_s = surface area of one side of a flat plate or the projected area of complex shapes
A_i = cross-sectional area of ice
D = diameter of a circular structure or member as defined in Chapter 29, in ft (m)
D_c = diameter of the cylinder circumscribing an object
f_z = factor to account for the increase in ice thickness with height
I_i = importance factor for ice thickness from Table 1.5-2 based on the Risk Category from Table 1.5-1
I_w = importance factor for concurrent wind pressure from Table 1.5-2 based on the Risk Category from Table 1.5-1
K_{zt} = topographic factor as defined in Chapter 26
q_z = velocity pressure evaluated at height z above ground, in lb/ft^2 (N/m^2) as defined in Chapter 29
r = radius of the maximum cross-section of a dome or radius of a sphere
t = nominal ice thickness due to freezing rain at a height of 33 ft (10 m) from Figs. 10-2 through 10-6 in inches (mm)
t_d = design ice thickness in in. (mm) from Eq. 10.4-5
V_c = concurrent wind speed mph (m/s) from Figs. 10-2 through 10-6
V_i = volume of ice
z = height above ground in ft (m)
$\in$ = solidity ratio as defined in Chapter 29

10.4 ICE LOADS DUE TO FREEZING RAIN

10.4.1 Ice Weight

The ice load shall be determined using the weight of glaze ice formed on all exposed surfaces of structural members, guys, components, appurtenances, and cable systems. On structural shapes, prismatic members, and other similar shapes, the cross-sectional area of ice shall be determined by

$$A_i = \pi t_d(D_c + t_d) \qquad (10.4\text{-}1)$$

D_c is shown for a variety of cross-sectional shapes in Fig. 10-1.

On flat plates and large three-dimensional objects such as domes and spheres, the volume of ice shall be determined by

$$V_i = \pi t_d A_s \qquad (10.4\text{-}2)$$

For a flat plate A_s shall be the area of one side of the plate, for domes and spheres A_s shall be determined by

$$A_s = \pi r^2 \qquad (10.4\text{-}3)$$

It is acceptable to multiply V_i by 0.8 for vertical plates and 0.6 for horizontal plates.

The ice density shall be not less than 56 pcf (900 kg/m^3).

10.4.2 Nominal Ice Thickness

Figs. 10-2 through 10-6 show the equivalent uniform radial thicknesses t of ice due to freezing rain at a height of 33 ft (10 m) over the contiguous 48 states and Alaska for a 50-year mean recurrence interval. Also shown are concurrent 3-s gust wind speeds. Thicknesses for Hawaii, and for ice accretions due to other sources in all regions, shall be obtained from local meteorological studies.

10.4.3 Height Factor

The height factor f_z used to increase the radial thickness of ice for height above ground z shall be determined by

$$\begin{aligned} f_z &= \left(\frac{z}{33}\right)^{0.10} && \text{for } 0\text{ ft} < z \le 900\text{ ft} \\ f_z &= 1.4 && \text{for } z > 900\text{ ft} \end{aligned} \qquad (10.4\text{-}4)$$

In SI:

$$\begin{aligned} f_z &= \left(\frac{z}{10}\right)^{0.10} && \text{for } 0\text{ m} < z \le 275\text{ m} \\ f_z &= 1.4 && \text{for } z > 275\text{ m} \end{aligned}$$

10.4.4 Importance Factors

Importance factors to be applied to the radial ice thickness and wind pressure shall be determined from Table 1.5-2 based on the Risk Category from Table 1.5-1. The importance factor I_i shall be applied to the ice thickness, not the ice weight, because the ice weight is not a linear function of thickness.

10.4.5 Topographic Factor

Both the ice thickness and concurrent wind speed for structures on hills, ridges, and escarpments are higher than those on level terrain because of wind speed-up effects. The topographic factor for the concurrent wind pressure is K_{zt} and the topographic factor for ice thickness is $(K_{zt})^{0.35}$, where K_{zt} is obtained from Eq. 26.8-1.

10.4.6 Design Ice Thickness for Freezing Rain

The design ice thickness t_d shall be calculated from Eq. 10.4-5.

$$t_d = 2.0 t I_i f_z (K_{zt})^{0.35} \qquad (10.4\text{-}5)$$

10.5 WIND ON ICE-COVERED STRUCTURES

Ice accreted on structural members, components, and appurtenances increases the projected area of the structure exposed to wind. The projected area shall be increased by adding t_d to all free edges of the projected area. Wind loads on this increased projected area shall be used in the design of ice-sensitive structures. Figs. 10-2 to 10-6 include 3-s gust wind speeds at 33 ft (10 m) above grade that are concurrent with the ice loads due to freezing rain. Wind loads shall be calculated in accordance with Chapters 26 through 31 as modified by Sections 10.5.1 through 10.5.5.

10.5.1 Wind on Ice-Covered Chimneys, Tanks, and Similar Structures

Force coefficients C_f for structures with square, hexagonal, and octagonal cross-sections shall be as given in Fig. 29.5-1. Force coefficients C_f for structures with round cross-sections shall be as given in Fig. 29.5-1 for round cross-sections with $D\sqrt{q_z} \leq 2.5$ for all ice thicknesses, wind speeds, and structure diameters.

10.5.2 Wind on Ice-Covered Solid Freestanding Walls and Solid Signs

Force coefficients C_f shall be as given in Fig. 29.4 based on the dimensions of the wall or sign including ice.

10.5.3 Wind on Ice-Covered Open Signs and Lattice Frameworks

The solidity ratio $\in$ shall be based on the projected area including ice. The force coefficient C_f for the projected area of flat members shall be as given in Fig. 29.5-2. The force coefficient C_f for rounded members and for the additional projected area due to ice on both flat and rounded members shall be as given in Fig. 29.5-2 for rounded members with $D\sqrt{q_z} \leq 2.5$ for all ice thicknesses, wind speeds, and member diameters.

10.5.4 Wind on Ice-Covered Trussed Towers

The solidity ratio $\in$ shall be based on the projected area including ice. The force coefficients C_f shall be as given in Fig. 29.5-3. It is acceptable to reduce the force coefficients C_f for the additional projected area due to ice on both round and flat members by the factor for rounded members in Note 3 of Fig. 29.5-3.

10.5.5 Wind on Ice-Covered Guys and Cables

The force coefficient C_f (as defined in Chapter 29) for ice-covered guys and cables shall be 1.2.

10.6 Design Temperatures for Freezing Rain

The design temperatures for ice and wind-on-ice due to freezing rain shall be either the temperature for the site shown in Figs. 10-7 and 10-8 or 32°F (0°C), whichever gives the maximum load effect. The temperature for Hawaii shall be 32°F (0°C). For temperature sensitive structures, the load shall include the effect of temperature change from everyday conditions to the design temperature for ice and wind-on-ice. These temperatures are to be used with ice thicknesses for all mean recurrence intervals. The design temperatures are considered to be concurrent with the design ice load and the concurrent wind load.

10.7 PARTIAL LOADING

The effects of a partial ice load shall be considered when this condition is critical for the type of structure under consideration. It is permitted to consider this to be a static load.

10.8 DESIGN PROCEDURE

1. The nominal ice thickness, t, the concurrent wind speed, V_c, and the concurrent temperature for the site shall be determined from Figs. 10-2 to 10-8 or a site-specific study.
2. The topographic factor for the site, K_{zt}, shall be determined in accordance with Section 10.4.5.
3. The importance factor for ice thickness, I_i, shall be determined in accordance with Section 10.4.4.
4. The height factor, f_z, shall be determined in accordance with Section 10.4.3 for each design segment of the structure.
5. The design ice thickness, t_d, shall be determined in accordance with Section 10.4.6, Eq. 10.4-5.
6. The weight of ice shall be calculated for the design ice thickness, t_d, in accordance with Section 10.4.1.
7. The velocity pressure q_z for wind speed V_c shall be determined in accordance with Section 29.3 using the importance factor for concurrent wind pressure I_w determined in accordance with Section 10.4.4.
8. The wind force coefficients C_f shall be determined in accordance with Section 10.5.
9. The gust effect factor shall be determined in accordance with Section 26.9.

10. The design wind force shall be determined in accordance with Chapter 29.
11. The iced structure shall be analyzed for the load combinations in either Section 2.3 or 2.4.

10.9 CONSENSUS STANDARDS AND OTHER REFERENCED DOCUMENTS

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

ASCE
American Society of Civil Engineers
1801 Alexander Bell Drive
Reston, VA 20191

ASCE Manual 74
Section 10.1.3
Guidelines for Electrical Transmission Line Structural Loading, 1991

ANSI
American National Standards Institute
25 West 43rd Street, 4th Floor
New York, NY 10036

ANSI/EIA/TIA-222
Section 10.1.3
Structural Standards for Steel Antenna Towers and Antenna Supporting Structures, 1996

IEEE
445 Hoes Lane
Piscataway, NJ 08854-1331

NESC
Section 10.1.3
National Electrical Safety Code, 2001

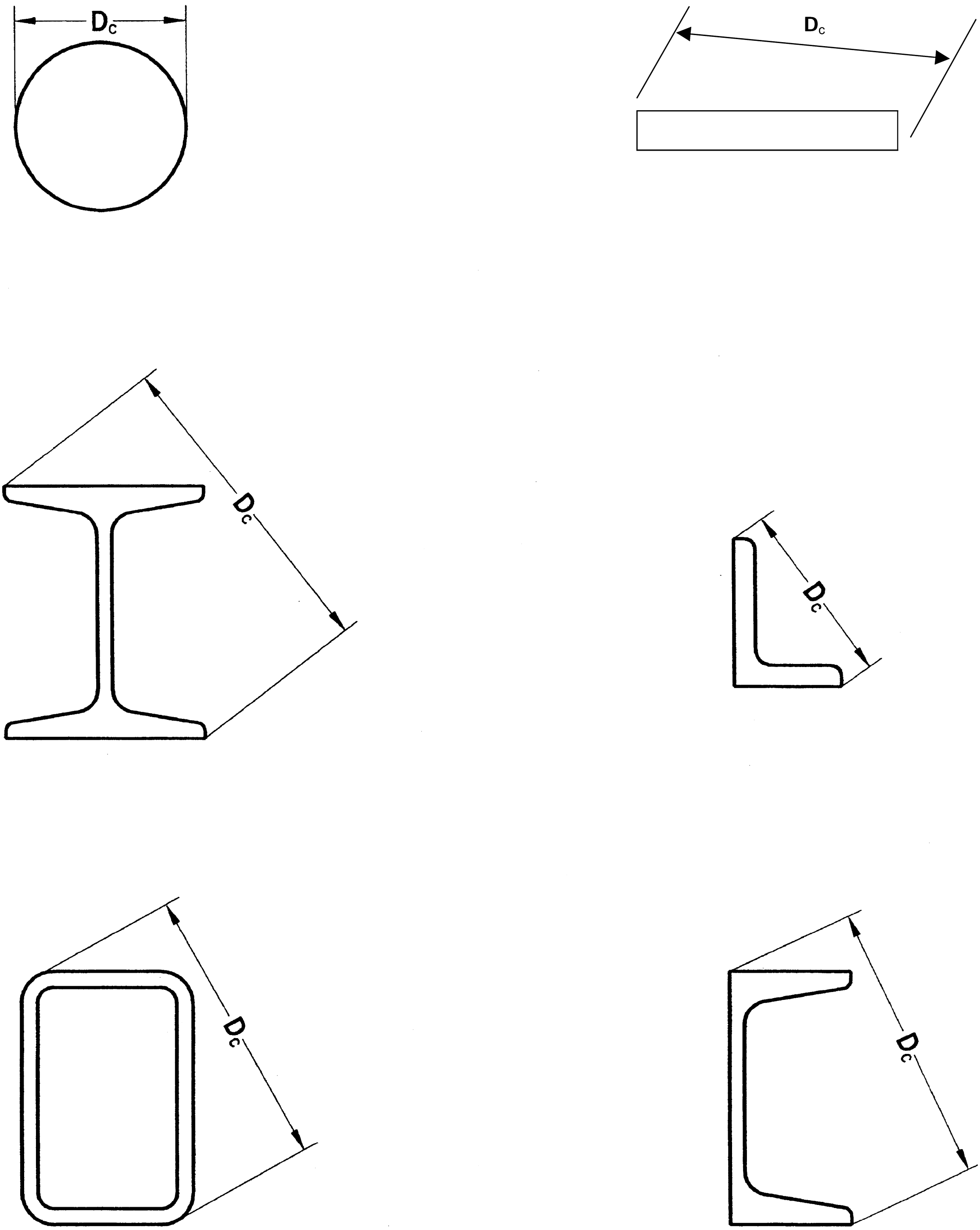
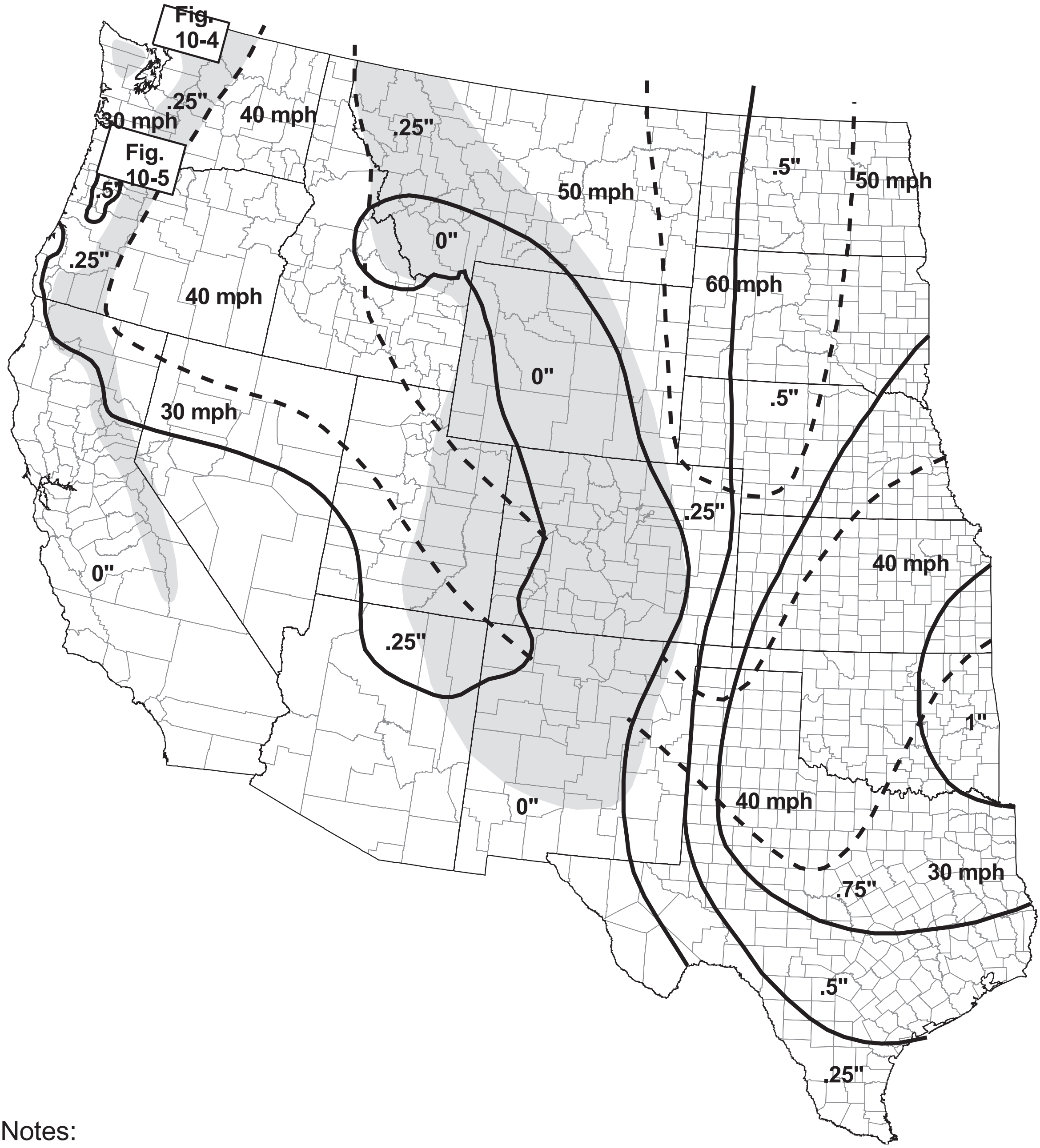
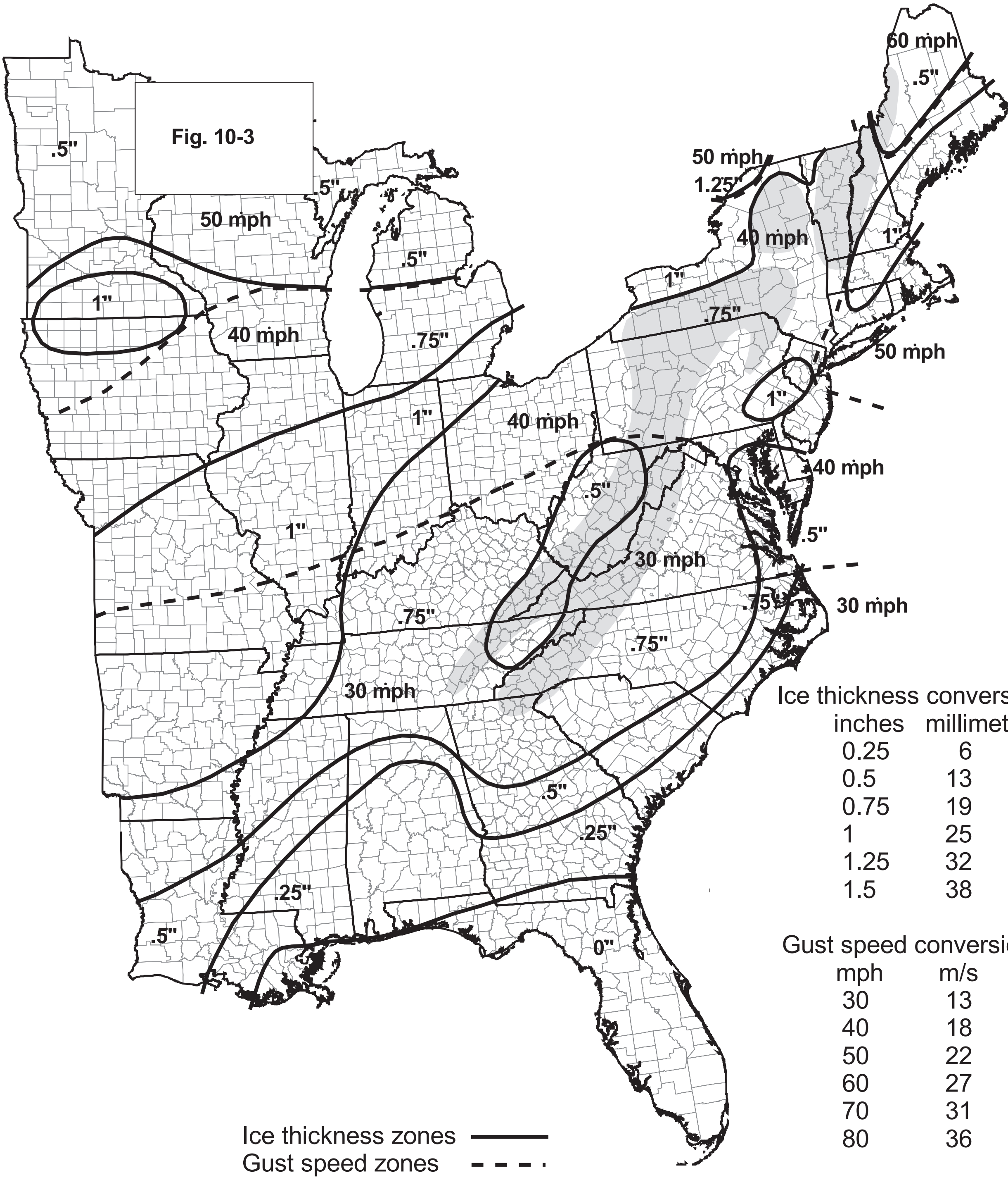


FIGURE 10-1 Characteristic Dimension D_c for Calculating the Ice Area for a Variety of Cross-Sectional Shapes.



Notes:
1. Ice thicknesses on structures in exposed locations at elevations higher than the surrounding terrain and in valleys and gorges may exceed the mapped values.
2. In the mountain west, indicated by the shading, ice thicknesses may exceed the mapped values in the foothills and passes. However, at elevations above 5,000 ft, freezing rain is unlikely.
3. In the Appalachian Mountains, indicated by the shading, ice thicknesses may vary significantly over short distances.

FIGURE 10-2 Equivalent Radial Ice Thicknesses Due to Freezing Rain with Concurrent 3-Second Gust Speeds, for a 50-Year Mean Recurrence Interval.



Ice thickness conversion	
inches	millimeters
0.25	6
0.5	13
0.75	19
1	25
1.25	32
1.5	38

Gust speed conversion	
mph	m/s
30	13
40	18
50	22
60	27
70	31
80	36

FIGURE 10-2 (*Continued*)

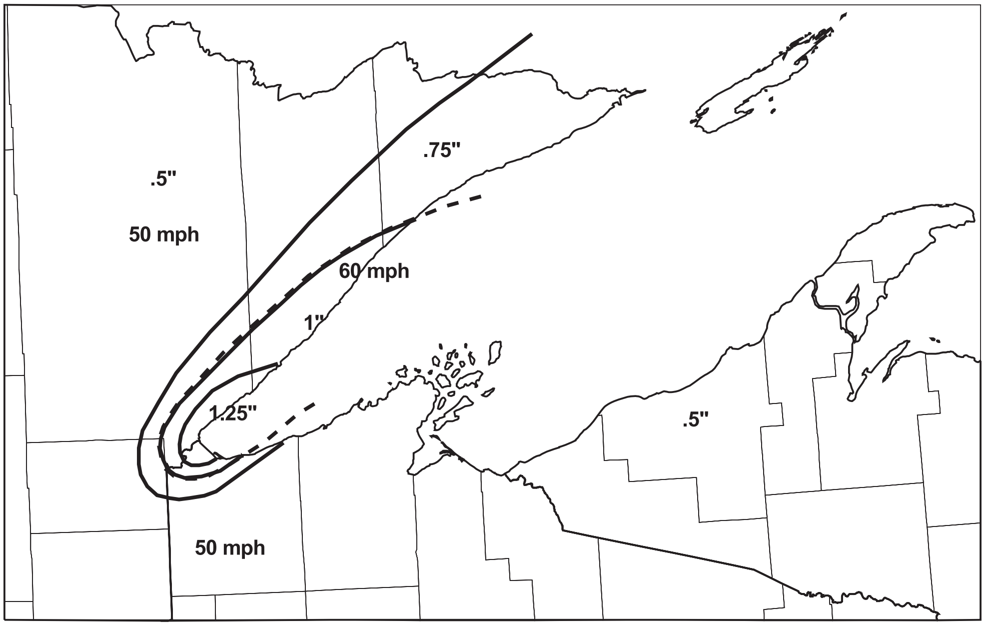


FIGURE 10-3 Lake Superior Detail.

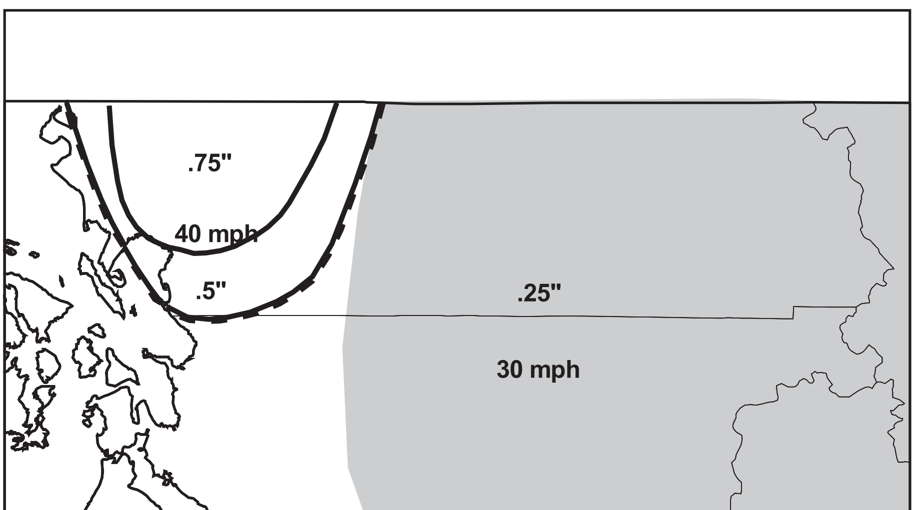


FIGURE 10-4 Fraser Valley Detail.

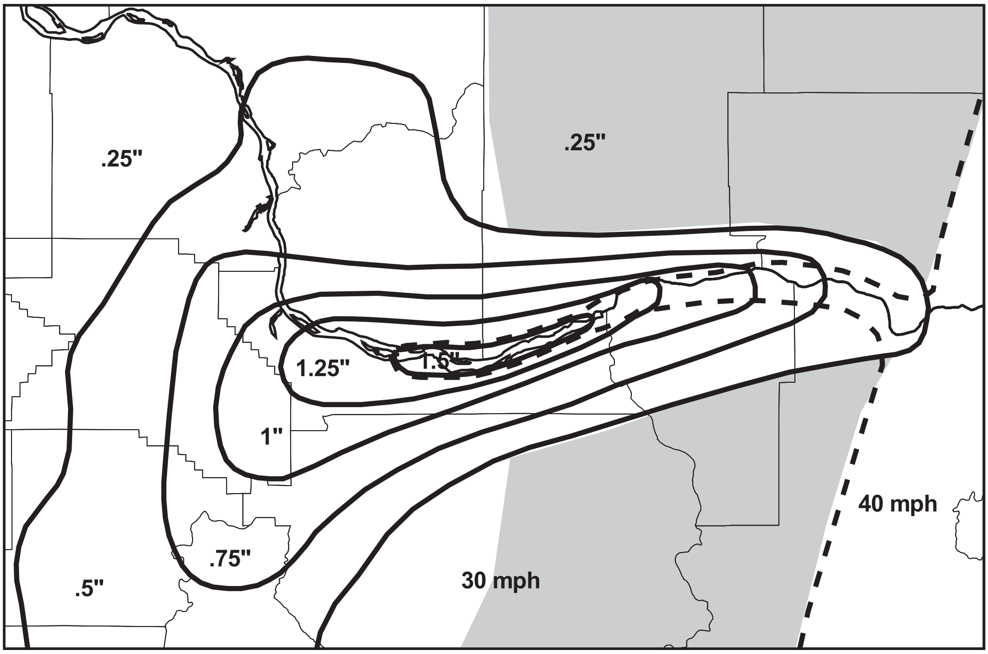


FIGURE 10-5 Columbia River Gorge Detail.

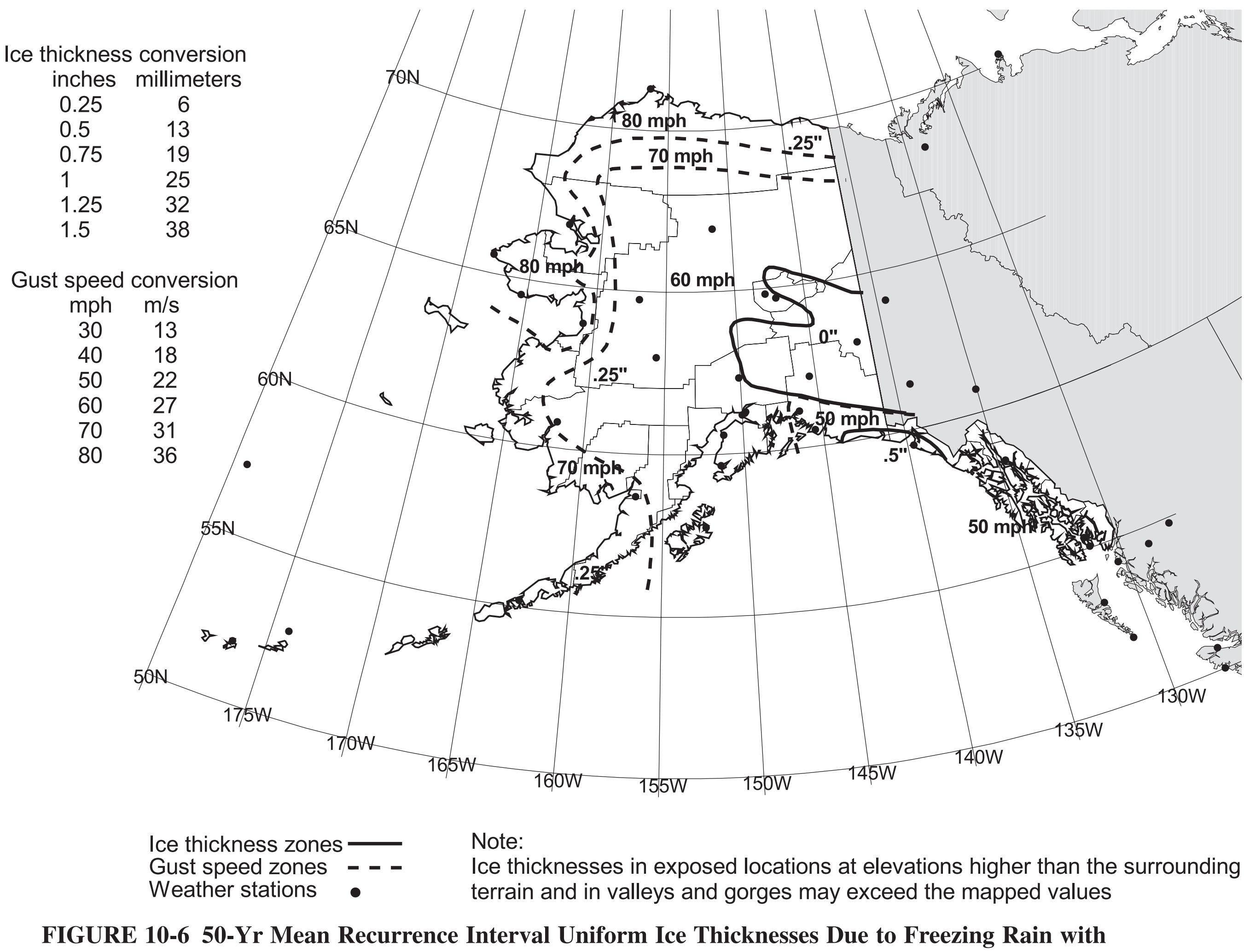


FIGURE 10-6 50-Yr Mean Recurrence Interval Uniform Ice Thicknesses Due to Freezing Rain with Concurrent 3-Second Gust Speeds: Alaska.

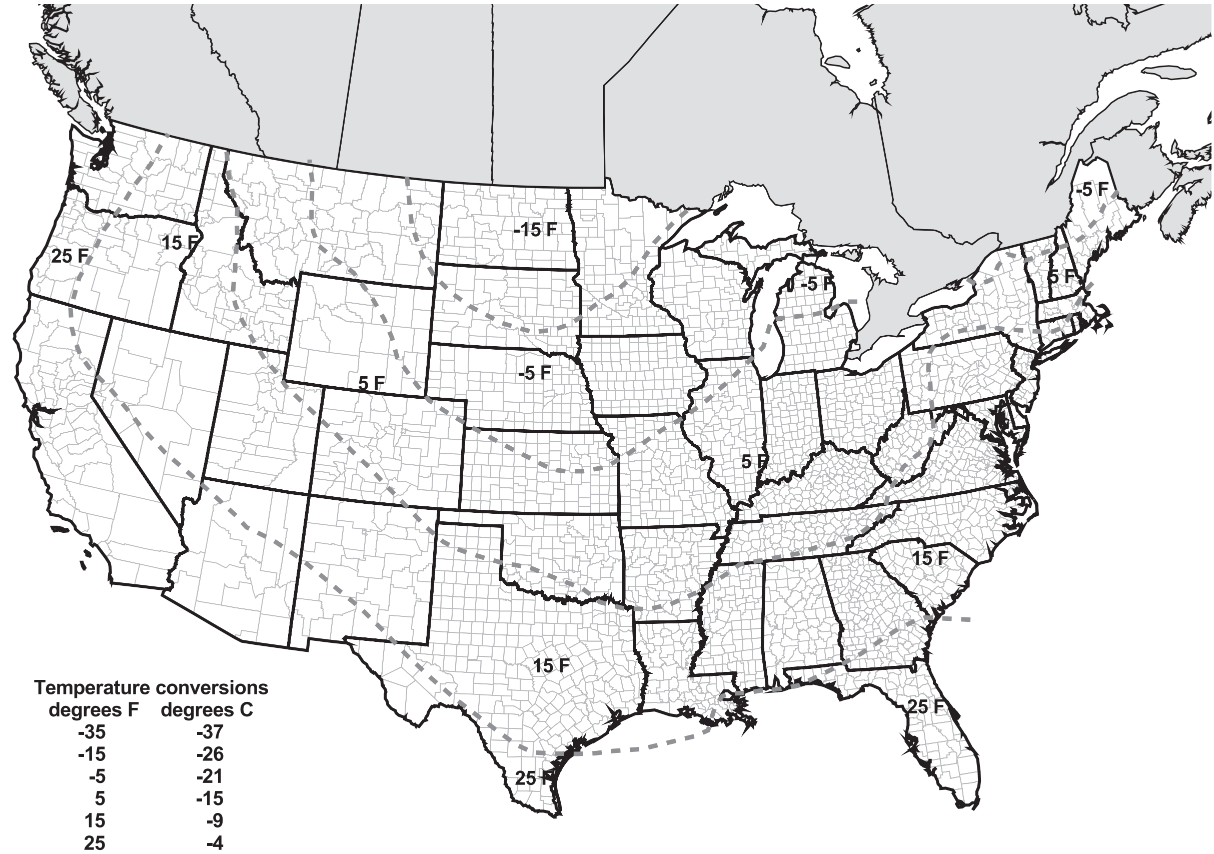


FIGURE 10-7 Temperatures Concurrent with Ice Thicknesses Due to Freezing Rain: Contiguous 48 States.

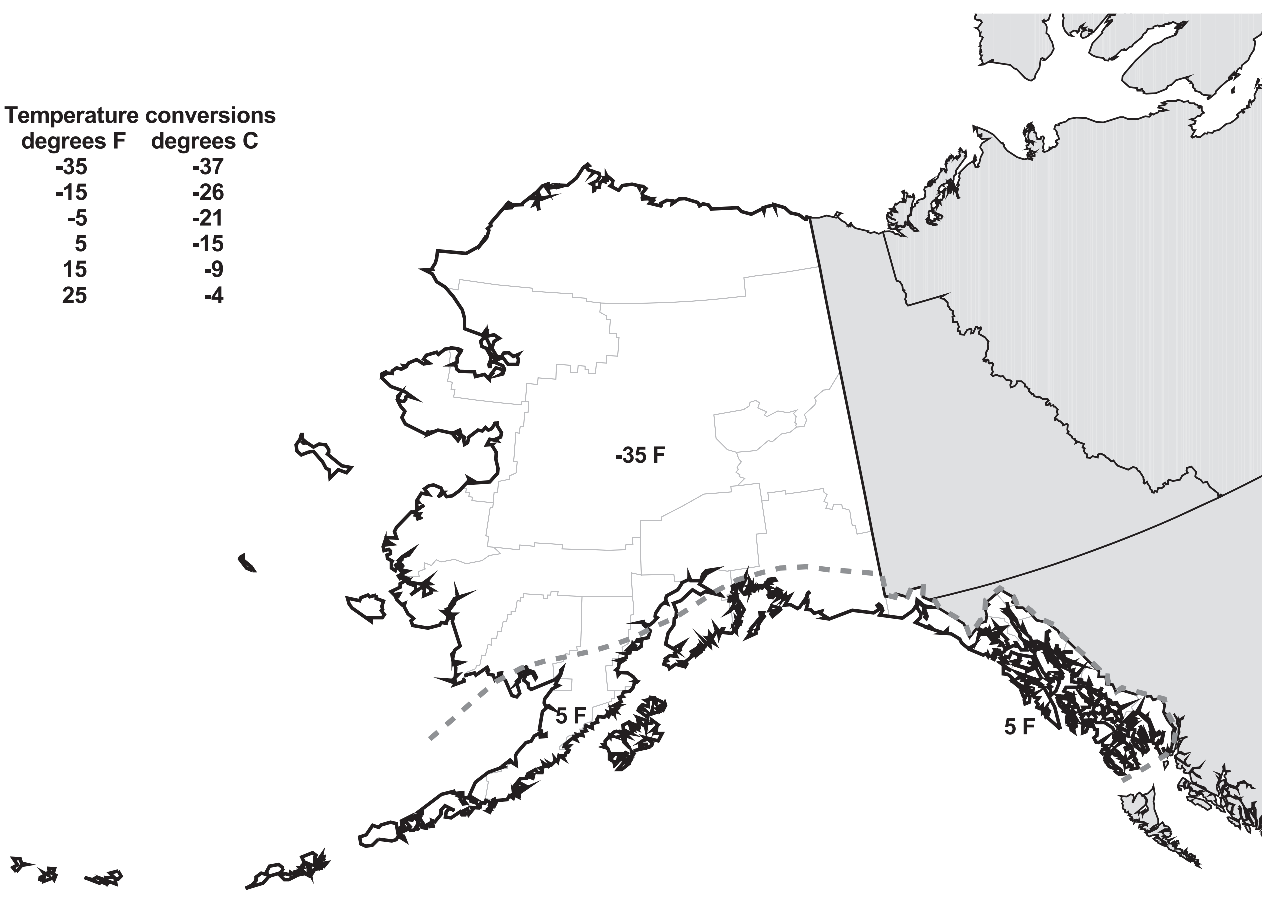


FIGURE 10-8 Temperatures Concurrent with Ice Thicknesses Due to Freezing Rain: Alaska.

Chapter 11
SEISMIC DESIGN CRITERIA

11.1 GENERAL

11.1.1 Purpose

Chapter 11 presents criteria for the design and construction of buildings and other structures subject to earthquake ground motions. The specified earthquake loads are based upon post-elastic energy dissipation in the structure, and because of this fact, the requirements for design, detailing, and construction shall be satisfied even for structures and members for which load combinations that do not contain earthquake loads indicate larger demands than combinations that include earthquake loads. Minimum requirements for quality assurance for seismic force-resisting systems are set forth in Appendix 11A.

11.1.2 Scope

Every structure, and portion thereof, including nonstructural components, shall be designed and constructed to resist the effects of earthquake motions as prescribed by the seismic requirements of this standard. Certain nonbuilding structures, as described in Chapter 15, are also within the scope and shall be designed and constructed in accordance with the requirements of Chapter 15. Requirements concerning alterations, additions, and change of use are set forth in Appendix 11B. Existing structures and alterations to existing structures need only comply with the seismic requirements of this standard where required by Appendix 11B. The following structures are exempt from the seismic requirements of this standard:

1. Detached one- and two-family dwellings that are located where the mapped, short period, spectral response acceleration parameter, S_S, is less than 0.4 or where the Seismic Design Category determined in accordance with Section 11.6 is A, B, or C.
2. Detached one- and two-family wood-frame dwellings not included in Exception 1 with not more than two stories above grade plane, satisfying the limitations of and constructed in accordance with the IRC.
3. Agricultural storage structures that are intended only for incidental human occupancy.
4. Structures that require special consideration of their response characteristics and environment that are not addressed in Chapter 15 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances, and nuclear reactors.
5. Piers and wharves that are not accessible to the general public.

11.1.3 Applicability

Structures and their nonstructural components shall be designed and constructed in accordance with the requirement of the following sections based on the type of structure or component:

a. Buildings: Chapter 12
b. Nonbuilding Structures: Chapter 15
c. Nonstructural Components: Chapter 13
d. Seismically Isolated Structures: Chapter 17
e. Structures with Damping Systems: Chapter 18

Buildings whose purpose is to enclose equipment or machinery and whose occupants are engaged in maintenance or monitoring of that equipment, machinery or their associated processes shall be permitted to be classified as nonbuilding structures designed and detailed in accordance with Section 15.5 of this standard.

11.1.4 Alternate Materials and Methods of Construction

Alternate materials and methods of construction to those prescribed in the seismic requirements of this standard shall not be used unless approved by the authority having jurisdiction. Substantiating evidence shall be submitted demonstrating that the proposed alternate, for the purpose intended, will be at least equal in strength, durability, and seismic resistance.

11.2 DEFINITIONS

The following definitions apply only to the seismic requirements of this standard.

ACTIVE FAULT: A fault determined to be active by the authority having jurisdiction from properly substantiated data (e.g., most recent mapping of active faults by the United States Geological Survey).

ADDITION: An increase in building area, aggregate floor area, height, or number of stories of a structure.

ALTERATION: Any construction or renovation to an existing structure other than an addition.

APPENDAGE: An architectural component such as a canopy, marquee, ornamental balcony, or statuary.

APPROVAL: The written acceptance by the authority having jurisdiction of documentation that establishes the qualification of a material, system, component, procedure, or person to fulfill the requirements of this standard for the intended use.

ATTACHMENTS: Means by which nonstructural components or supports of nonstructural components are secured or connected to the seismic force-resisting system of the structure. Such attachments include anchor bolts, welded connections, and mechanical fasteners.

BASE: The level at which the horizontal seismic ground motions are considered to be imparted to the structure.

BASE SHEAR: Total design lateral force or shear at the base.

BOUNDARY ELEMENTS: Diaphragm and shear wall boundary members to which the diaphragm transfers forces. Boundary members include chords and drag struts at diaphragm and shear wall perimeters, interior openings, discontinuities, and reentrant corners.

BOUNDARY MEMBERS: Portions along wall and diaphragm edges strengthened by longitudinal and transverse reinforcement. Boundary members include chords and drag struts at diaphragm and shear wall perimeters, interior openings, discontinuities, and reentrant corners.

BUILDING: Any structure whose intended use includes shelter of human occupants.

CANTILEVERED COLUMN SYSTEM: A seismic force-resisting system in which lateral forces are resisted entirely by columns acting as cantilevers from the base.

CHARACTERISTIC EARTHQUAKE: An earthquake assessed for an active fault having a magnitude equal to the best estimate of the maximum magnitude capable of occurring on the fault, but not less than the largest magnitude that has occurred historically on the fault.

COMPONENT: A part of an architectural, electrical, or mechanical system.

Component, Nonstructural: A part of an architectural, mechanical, or electrical system within or without a building or nonbuilding structure.

Component, Flexible: Nonstructural component having a fundamental period greater than 0.06 s.

Component, Rigid: Nonstructural component having a fundamental period less than or equal to 0.06 s.

CONCRETE, PLAIN: Concrete that is either unreinforced or contains less reinforcement than the minimum amount specified in ACI 318 for reinforced concrete.

CONCRETE, REINFORCED: Concrete reinforced with no less reinforcement than the minimum amount required by ACI 318 prestressed or nonprestressed, and designed on the assumption that the two materials act together in resisting forces.

CONSTRUCTION DOCUMENTS: The written, graphic, electronic, and pictorial documents describing the design, locations, and physical characteristics of the project required to verify compliance with this standard.

COUPLING BEAM: A beam that is used to connect adjacent concrete wall elements to make them act together as a unit to resist lateral loads.

DEFORMABILITY: The ratio of the ultimate deformation to the limit deformation.

High-Deformability Element: An element whose deformability is not less than 3.5 where subjected to four fully reversed cycles at the limit deformation.

Limited-Deformability Element: An element that is neither a low-deformability nor a high-deformability element.

Low-Deformability Element: An element whose deformability is 1.5 or less.

DEFORMATION:

Limit Deformation: Two times the initial deformation that occurs at a load equal to 40 percent of the maximum strength.

Ultimate Deformation: The deformation at which failure occurs and that shall be deemed to occur if the sustainable load reduces to 80 percent or less of the maximum strength.

DESIGNATED SEISMIC SYSTEMS: Those nonstructural components that require design in accordance with Chapter 13 and for which the component importance factor, I_p, is greater than 1.0.

DESIGN EARTHQUAKE: The earthquake effects that are two-thirds of the corresponding Maximum Considered Earthquake (MCE_R) effects.

DESIGN EARTHQUAKE GROUND MOTION: The earthquake ground motions that are two-thirds of the corresponding MCE_R ground motions.

DIAPHRAGM: Roof, floor, or other membrane or bracing system acting to transfer the lateral forces to the vertical resisting elements.

DIAPHRAGM BOUNDARY: A location where shear is transferred into or out of the diaphragm element. Transfer is either to a boundary element or to another force-resisting element.

DIAPHRAGM CHORD: A diaphragm boundary element perpendicular to the applied load that is assumed to take axial stresses due to the diaphragm moment.

DRAG STRUT (COLLECTOR, TIE, DIAPHRAGM STRUT): A diaphragm or shear wall boundary element parallel to the applied load that collects and transfers diaphragm shear forces to the vertical force-resisting elements or distributes forces within the diaphragm or shear wall.

ENCLOSURE: An interior space surrounded by walls.

EQUIPMENT SUPPORT: Those structural members or assemblies of members or manufactured elements, including braces, frames, legs, lugs, snuggers, hangers, or saddles that transmit gravity loads and operating loads between the equipment and the structure.

FLEXIBLE CONNECTIONS: Those connections between equipment components that permit rotational and/or translational movement without degradation of performance. Examples include universal joints, bellows expansion joints, and flexible metal hose.

FRAME:

Braced Frame: An essentially vertical truss, or its equivalent, of the concentric or eccentric type that is provided in a building frame system or dual system to resist seismic forces.

Concentrically Braced Frame (CBF): A braced frame in which the members are subjected primarily to axial forces. CBFs are categorized as ordinary concentrically braced frames (OCBFs) or special concentrically braced frames (SCBFs).

Eccentrically Braced Frame (EBF): A diagonally braced frame in which at least one end of each brace frames into a beam a short distance from a beam-column or from another diagonal brace.

Moment Frame: A frame in which members and joints resist lateral forces by flexure as well as along the axis of the members. Moment frames are categorized as intermediate moment frames (IMF), ordinary moment frames (OMF), and special moment frames (SMF).

Structural System:

Building Frame System: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by shear walls or braced frames.

Dual System: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by moment-resisting frames and shear walls or braced frames as prescribed in Section 12.2.5.1.

Shear Wall-Frame Interactive System: A structural system that uses combinations of ordinary reinforced concrete shear walls and ordinary reinforced concrete moment frames designed to resist lateral forces in proportion to their rigidities considering interaction between shear walls and frames on all levels.

Space Frame System: A 3-D structural system composed of interconnected members, other than bearing walls, that is capable of supporting vertical loads and, where designed for such an application, is capable of providing resistance to seismic forces.

FRICTION CLIP: A device that relies on friction to resist applied loads in one or more directions to anchor a nonstructural component. Friction is provided mechanically and is not due to gravity loads.

GLAZED CURTAIN WALL: A nonbearing wall that extends beyond the edges of building floor slabs, and includes a glazing material installed in the curtain wall framing.

GLAZED STOREFRONT: A nonbearing wall that is installed between floor slabs, typically including entrances, and includes a glazing material installed in the storefront framing.

GRADE PLANE: A horizontal reference plane representing the average of finished ground level adjoining the structure at all exterior walls. Where the finished ground level slopes away from the exterior walls, the grade plane is established by the lowest points within the area between the structure and the property line or, where the property line is more than 6 ft (1,829 mm) from the structure, between the structure and points 6 ft (1,829 mm) from the structure.

INSPECTION, SPECIAL: The observation of the work by a special inspector to determine compliance with the approved construction documents and these standards in accordance with the quality assurance plan.

Continuous Special Inspection: The full-time observation of the work by a special inspector who is present in the area where work is being performed.

Periodic Special Inspection: The part-time or intermittent observation of the work by a special inspector who is present in the area where work has been or is being performed.

INSPECTOR, SPECIAL (who shall be identified as the owner's inspector): A person approved by the authority having jurisdiction to perform special inspection.

INVERTED PENDULUM-TYPE STRUCTURES: Structures in which more than 50 percent of the structure's mass is concentrated at the top of a slender, cantilevered structure and in which stability of the mass at the top of the structure relies on rotational restraint to the top of the cantilevered element.

JOINT: The geometric volume common to intersecting members.

LIGHT-FRAME CONSTRUCTION: A method of construction where the structural assemblies (e.g., walls, floors, ceilings, and roofs) are primarily formed by a system of repetitive wood or cold-formed steel framing members or subassemblies of these members (e.g., trusses).

LONGITUDINAL REINFORCEMENT RATIO: Area of longitudinal reinforcement divided by the cross-sectional area of the concrete.

MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION: The most severe earthquake effects considered by this standard more specifically defined in the following two terms.

MAXIMUM CONSIDERED EARTHQUAKE GEOMETRIC MEAN (MCE_G) PEAK GROUND ACCELERATION: The most severe earthquake effects considered by this standard determined for geometric mean peak ground acceleration and without adjustment for targeted risk. The MCE_G peak ground acceleration adjusted for site effects (PGA_M) is used in this standard for evaluation of liquefaction, lateral spreading, seismic settlements, and other soil related issues. In this standard, general procedures for determining PGA_M are provided in Section 11.8.3; site-specific procedures are provided in Section 21.5.

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATION: The most severe earthquake effects considered by this standard determined for the orientation that results in the largest maximum response to horizontal ground motions and with adjustment for targeted risk. In this standard, general procedures for determining the MCE_R Ground Motion values are provided in Section 11.4.3; site-specific procedures are provided in Sections 21.1 and 21.2.

MECHANICALLY ANCHORED TANKS OR VESSELS: Tanks or vessels provided with mechanical anchors to resist overturning moments.

NONBUILDING STRUCTURE: A structure, other than a building, constructed of a type included in Chapter 15 and within the limits of Section 15.1.1.

NONBUILDING STRUCTURE SIMILAR TO A BUILDING: A nonbuilding structure that is designed and constructed in a manner similar to buildings, will respond to strong ground motion in a fashion similar to buildings, and has a basic lateral and vertical seismic force-resisting system conforming to one of the types indicated in Tables 12.2-1 or 15.4-1.

ORTHOGONAL: To be in two horizontal directions, at 90° to each other.

OWNER: Any person, agent, firm, or corporation having a legal or equitable interest in the property.

PARTITION: A nonstructural interior wall that spans horizontally or vertically from support to support. The supports may be the basic building frame, subsidiary structural members, or other portions of the partition system.

***P*-DELTA EFFECT:** The secondary effect on shears and moments of structural members due to the action of the vertical loads induced by horizontal displacement of the structure resulting from various loading conditions.

PILE: Deep foundation element, which includes piers, caissons, and piles.

PILE CAP: Foundation elements to which piles are connected including grade beams and mats.

REGISTERED DESIGN PROFESSIONAL: An architect or engineer, registered or licensed to practice professional architecture or engineering, as defined by the statutory requirements of the professional registrations laws of the state in which the project is to be constructed.

SEISMIC DESIGN CATEGORY: A classification assigned to a structure based on its Risk Category and the severity of the design earthquake ground motion at the site as defined in Section 11.4.

SEISMIC FORCE-RESISTING SYSTEM: That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces prescribed herein.

SEISMIC FORCES: The assumed forces prescribed herein, related to the response of the structure to earthquake motions, to be used in the design of the structure and its components.

SELF-ANCHORED TANKS OR VESSELS: Tanks or vessels that are stable under design overturning moment without the need for mechanical anchors to resist uplift.

SHEAR PANEL: A floor, roof, or wall element sheathed to act as a shear wall or diaphragm.

SITE CLASS: A classification assigned to a site based on the types of soils present and their engineering properties as defined in Chapter 20.

STORAGE RACKS: Include industrial pallet racks, moveable shelf racks, and stacker racks made of cold-formed or hot-rolled structural members. Does not include other types of racks such as drive-in and drive-through racks, cantilever racks, portable racks, or racks made of materials other than steel.

STORY: The portion of a structure between the tops of two successive floor surfaces and, for the topmost story, from the top of the floor surface to the top of the roof surface.

STORY ABOVE GRADE PLANE: A story in which the floor or roof surface at the top of the story is more than 6 ft (1,828 mm) above grade plane or is more than 12 ft (3,658 mm) above the finished ground level at any point on the perimeter of the structure.

STORY DRIFT: The horizontal deflection at the top of the story relative to the bottom of the story as determined in Section 12.8.6.

STORY DRIFT RATIO: The story drift, as determined in Section 12.8.6, divided by the story height, h_{sx}.

STORY SHEAR: The summation of design lateral seismic forces at levels above the story under consideration.

STRENGTH:

Design Strength: Nominal strength multiplied by a strength reduction factor, φ.

Nominal Strength: Strength of a member or cross-section calculated in accordance with the requirements and assumptions of the strength design methods of this standard (or the reference documents) before application of any strength-reduction factors.

Required Strength: Strength of a member, cross-section, or connection required to resist factored loads or related internal moments and forces in such combinations as stipulated by this standard.

STRUCTURAL HEIGHT: The vertical distance from the base to the highest level of the seismic force-resisting system of the structure. For pitched or sloped roofs, the structural height is from the base to the average height of the roof.

STRUCTURAL OBSERVATIONS: The visual observations to determine that the seismic force-resisting system is constructed in general conformance with the construction documents.

STRUCTURE: That which is built or constructed and limited to buildings and nonbuilding structures as defined herein.

SUBDIAPHRAGM: A portion of a diaphragm used to transfer wall anchorage forces to diaphragm cross ties.

SUPPORTS: Those members, assemblies of members, or manufactured elements, including braces, frames, legs, lugs, snubbers, hangers, saddles, or struts, and associated fasteners that transmit loads between nonstructural components and their attachments to the structure.

TESTING AGENCY: A company or corporation that provides testing and/or inspection services.

VENEERS: Facings or ornamentation of brick, concrete, stone, tile, or similar materials attached to a backing.

WALL: A component that has a slope of 60° or greater with the horizontal plane used to enclose or divide space.

Bearing Wall: Any wall meeting either of the following classifications:

1. Any metal or wood stud wall that supports more than 100 lb/linear ft (1,459 N/m) of vertical load in addition to its own weight.
2. Any concrete or masonry wall that supports more than 200 lb/linear ft (2,919 N/m) of vertical load in addition to its own weight.

Light Frame Wall: A wall with wood or steel studs.

Light Frame Wood Shear Wall: A wall constructed with wood studs and sheathed with material rated for shear resistance.

Nonbearing Wall: Any wall that is not a bearing wall.

Nonstructural Wall: All walls other than bearing walls or shear walls.

Shear Wall (Vertical Diaphragm): A wall, bearing or nonbearing, designed to resist lateral forces acting in the plane of the wall (sometimes referred to as a "vertical diaphragm").

Structural Wall: Walls that meet the definition for bearing walls or shear walls.

WALL SYSTEM, BEARING: A structural system with bearing walls providing support for all or major portions of the vertical loads. Shear walls or braced frames provide seismic force resistance.

WOOD STRUCTURAL PANEL: A wood-based panel product that meets the requirements of DOC PS1 or DOC PS2 and is bonded with a waterproof adhesive. Included under this designation are plywood, oriented strand board, and composite panels.

11.3 SYMBOLS

The unit dimensions used with the items covered by the symbols shall be consistent throughout except where specifically noted. Symbols presented in this section apply only to the seismic requirements in this standard as indicated.

A_{ch} = cross-sectional area (in.^{2} or mm^2) of a structural member measured out-to-out of transverse reinforcement

A_0 = area of the load-carrying foundation (ft^2 or m^2)

A_{sh} = total cross-sectional area of hoop reinforcement (in.^{2} or mm^2), including supplementary cross-ties, having a spacing of s_h and crossing a section with a core dimension of h_c

A_{vd} = required area of leg (in.^{2} or mm^2) of diagonal reinforcement

A_x = torsional amplification factor (Section 12.8.4.3)

a_i = the acceleration at level i obtained from a modal analysis (Section 13.3.1)

a_p = the amplification factor related to the response of a system or component as affected by the type of seismic attachment, determined in Section 13.3.1

b_p = the width of the rectangular glass panel

C_d = deflection amplification factor as given in Tables 12.2-1, 15.4-1, or 15.4-2

C_R = site-specific risk coefficient at any period; see Section 21.2.1.1

C_{RS} = mapped value of the risk coefficient at short periods as given by Fig. 22-17

C_{R1} = mapped value of the risk coefficient at a period of 1 s as given by Fig. 22-18

C_s = seismic response coefficient determined in Section 12.8.1.1 and 19.3.1 (dimensionless)

C_T = building period coefficient in Section 12.8.2.1

C_{vx} = vertical distribution factor as determined in Section 12.8.3

c = distance from the neutral axis of a flexural member to the fiber of maximum compressive strain (in. or mm)

D = the effect of dead load

D_{clear} = relative horizontal (drift) displacement, measured over the height of the glass panel under consideration, which causes initial glass-to-frame contact. For rectangular glass panels within a rectangular wall frame, D_{clear} is set forth in Section 13.5.9.1

D_{pI} = seismic relative displacement; see Section 13.3.2

D_s = the total depth of stratum in Eq. 19.2-12 (ft or m)

d_C = The total thickness of cohesive soil layers in the top 100 ft (30 m); see Section 20.4.3 (ft or m)

d_i = The thickness of any soil or rock layer i (between 0 and 100 ft [30 m]); see Section 20.4.1 (ft or m)

d_S = The total thickness of cohesionless soil layers in the top 100 ft (30 m); see Section 20.4.2 (ft or m)

E = effect of horizontal and vertical earthquake-induced forces (Section 12.4)

F_a = short-period site coefficient (at 0.2 s-period); see Section 11.4.3

F_i, F_n, F_x = portion of the seismic base shear, V, induced at Level i, n, or x, respectively, as determined in Section 12.8.3

F_p = the seismic force acting on a component of a structure as determined in Sections 12.11.1 and 13.3.1

F_{PGA} = site coefficient for PGA; see Section 11.8.3

F_v = long-period site coefficient (at 1.0 s-period); see Section 11.4.3

f_c' = specified compressive strength of concrete used in design

f_s' = ultimate tensile strength (psi or MPa) of the bolt, stud, or insert leg wires. For ASTM A307 bolts or A108 studs, it is permitted to be assumed to be 60,000 psi (415 MPa)

f_y = specified yield strength of reinforcement (psi or MPa)

f_{yh} = specified yield strength of the special lateral reinforcement (psi or kPa)

$G = \gamma v_s^2/g$ = the average shear modulus for the soils beneath the foundation at large strain levels (psf or Pa)
$G_0 = \gamma v_{s0}^2/g$ = the average shear modulus for the soils beneath the foundation at small strain levels (psf or Pa)
g = acceleration due to gravity
H = thickness of soil
h = height of a shear wall measured as the maximum clear height from top of foundation to bottom of diaphragm framing above, or the maximum clear height from top of diaphragm to bottom of diaphragm framing above
h = average roof height of structure with respect to the base; see Chapter 13
$\bar{h}$ = effective height of the building as determined in Section 19.2.1.1 or 19.3.1 (ft or m)
h_c = core dimension of a component measured to the outside of the special lateral reinforcement (in. or mm)
h_i, h_x = the height above the base to Level i or x, respectively
h_n = structural height as defined in Section 11.2
h_p = the height of the rectangular glass panel
h_{sx} = the story height below Level $x = (h_x - h_{x-1})$
I_e = the importance factor as prescribed in Section 11.5.1
I_0 = the static moment of inertia of the load-carrying foundation; see Section 19.2.1.1 (in.^{4} or mm^4)
I_p = the component importance factor as prescribed in Section 13.3.1
i = the building level referred to by the subscript i; i = 1 designates the first level above the base
K_p = the stiffness of the component or attachment, Section 13.6.2
K_y = the lateral stiffness of the foundation as defined in Section 19.2.1.1 (lb/in. or N/m)
K_θ = the rocking stiffness of the foundation as defined in Section 19.2.1.1 (ft-lb/degree or N-m/rad)
KL/r = the lateral slenderness ratio of a compression member measured in terms of its effective length, KL, and the least radius of gyration of the member cross section, r
k = distribution exponent given in Section 12.8.3
$\bar{k}$ = stiffness of the building as determined in Section 19.2.1.1 (lb/ft or N/m)
k_a = coefficient defined in Sections 12.11.2 and 12.14.7.5
L = overall length of the building (ft or m) at the base in the direction being analyzed
L_0 = overall length of the side of the foundation in the direction being analyzed, Section 19.2.1.2 (ft or m)
M_0, M_{01} = the overturning moment at the foundation–soil interface as determined in Sections 19.2.3 and 19.3.2 (ft-lb or N-m)
M_t = torsional moment resulting from eccentricity between the locations of center of mass and the center of rigidity (Section 12.8.4.1)
M_{ta} = accidental torsional moment as determined in Section 12.8.4.2
m = a subscript denoting the mode of vibration under consideration; that is, m = 1 for the fundamental mode
N = standard penetration resistance, ASTM D-1586
N = number of stories above the base (Section 12.8.2.1)
$\bar{N}$ = average field standard penetration resistance for the top 100 ft (30 m); see Sections 20.3.3 and 20.4.2
$\bar{N}_{ch}$ = average standard penetration resistance for cohesionless soil layers for the top 100 ft (30 m); see Sections 20.3.3 and 20.4.2
N_i = standard penetration resistance of any soil or rock layer i (between 0 and 100 ft [30 m]); see Section 20.4.2
n = designation for the level that is uppermost in the main portion of the building
PGA = mapped MCE_G peak ground acceleration shown in Figs. 22-6 through 22-10
PGA_M = MCE_G peak ground acceleration adjusted for Site Class effects; see Section 11.8.3
P_x = total unfactored vertical design load at and above level x, for use in Section 12.8.7
PI = plasticity index, ASTM D4318
Q_E = effect of horizontal seismic (earthquake-induced) forces
R = response modification coefficient as given in Tables 12.2-1, 12.14-1, 15.4-1, or 15.4-2
R_p = component response modification factor as defined in Section 13.3.1
r = a characteristic length of the foundation as defined in Section 19.2.1.2
r_a = characteristic foundation length as defined by Eq. 19.2-7 (ft or m)

r_m = characteristic foundation length as defined by Eq. 19.2-8 (ft or m)

S_S = mapped MCE_R, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 11.4.1

S_1 = mapped MCE_R, 5 percent damped, spectral response acceleration parameter at a period of 1 s as defined in Section 11.4.1

S_{aM} = the site-specific MCE_R spectral response acceleration parameter at any period

S_{DS} = design, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 11.4.4

S_{D1} = design, 5 percent damped, spectral response acceleration parameter at a period of 1 s as defined in Section 11.4.4

S_{MS} = the MCE_R, 5 percent damped, spectral response acceleration parameter at short periods adjusted for site class effects as defined in Section 11.4.3

S_{M1} = the MCE_R, 5 percent damped, spectral response acceleration parameter at a period of 1 s adjusted for site class effects as defined in Section 11.4.3

s_u = undrained shear strength; see Section 20.4.3

$\overline{s}_u$ = average undrained shear strength in top 100 ft (30 m); see Sections 20.3.3 and 20.4.3, ASTM D2166 or ASTM D2850

s_{ui} = undrained shear strength of any cohesive soil layer i (between 0 and 100 ft [30 m]); see Section 20.4.3

s_h = spacing of special lateral reinforcement (in. or mm)

T = the fundamental period of the building

$\tilde{T}$, $\tilde{T}_1$ = the effective fundamental period(s) of the building as determined in Sections 19.2.1.1 and 19.3.1

T_a = approximate fundamental period of the building as determined in Section 12.8.2

T_L = long-period transition period as defined in Section 11.4.5

T_p = fundamental period of the component and its attachment, Section 13.6.2

$T_0 = 0.2S_{D1}/S_{DS}$

$T_S = S_{D1}/S_{DS}$

T_4 = net tension in steel cable due to dead load, prestress, live load, and seismic load (Section 14.1.7)

V = total design lateral force or shear at the base

V_t = design value of the seismic base shear as determined in Section 12.9.4

V_x = seismic design shear in story x as determined in Section 12.8.4 or 12.9.4

$\tilde{V}$ = reduced base shear accounting for the effects of soil structure interaction as determined in Section 19.3.1

$\tilde{V}_1$ = portion of the reduced base shear, $\tilde{V}$, contributed by the fundamental mode, Section 19.3 (kip or kN)

ΔV = reduction in V as determined in Section 19.3.1 (kip or kN)

ΔV_1 = reduction in V_1 as determined in Section 19.3.1 (kip or kN)

v_s = shear wave velocity at small shear strains (greater than 10^{-3} percent strain); see Section 19.2.1 (ft/s or m/s)

$\overline{v}_s$ = average shear wave velocity at small shear strains in top 100 ft (30 m); see Sections 20.3.3 and 20.4.1

v_{si} = the shear wave velocity of any soil or rock layer i (between 0 and 100 ft [30 m]); see Section 20.4.1

v_{so} = average shear wave velocity for the soils beneath the foundation at small strain levels, Section 19.2.1.1 (ft/s or m/s)

W = effective seismic weight of the building as defined in Section 12.7.2. For calculation of seismic-isolated building period, W is the total effective seismic weight of the building as defined in Sections 19.2 and 19.3 (kip or kN)

$\overline{W}$ = effective seismic weight of the building as defined in Sections 19.2 and 19.3 (kip or kN)

W_c = gravity load of a component of the building

W_p = component operating weight (lb or N)

w = moisture content (in percent), ASTM D2216

w_i, w_n, w_x = portion of W that is located at or assigned to Level i, n, or x, respectively

x = level under consideration, 1 designates the first level above the base

z = height in structure of point of attachment of component with respect to the base; see Section 13.3.1

β = ratio of shear demand to shear capacity for the story between Level x and $x - 1$

$\overline{\beta}$ = fraction of critical damping for the coupled structure-foundation system, determined in Section 19.2.1

β_0 = foundation damping factor as specified in Section 19.2.1.2

γ = average unit weight of soil (lb/ft^3 or N/m^3)

Δ = design story drift as determined in Section 12.8.6

Δ_{fallout} = the relative seismic displacement (drift) at which glass fallout from the curtain wall, storefront, or partition occurs

Δ_a = allowable story drift as specified in Section 12.12.1

$\delta_{\max}$ = maximum displacement at Level x, considering torsion, Section 12.8.4.3

δ_M = maximum inelastic response displacement, considering torsion, Section 12.12.3

δ_{MT} = total separation distance between adjacent structures on the same property, Section 12.12.3

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x, Section 12.8.4.3

δ_x = deflection of Level x at the center of the mass at and above Level x, Eq. 12.8-15

δ_{xe} = deflection of Level x at the center of the mass at and above Level x determined by an elastic analysis, Section 12.8-6

δ_{xm} = modal deflection of Level x at the center of the mass at and above Level x as determined by Section 19.3.2

$\bar{\delta}_x$, $\bar{\delta}_{x1}$ = deflection of Level x at the center of the mass at and above Level x, Eqs. 19.2-13 and 19.3-3 (in. or mm)

θ = stability coefficient for P-delta effects as determined in Section 12.8.7

ρ = a redundancy factor based on the extent of structural redundancy present in a building as defined in Section 12.3.4

ρ_s = spiral reinforcement ratio for precast, prestressed piles in Section 14.2.3.2.6

λ = time effect factor

Ω_0 = overstrength factor as defined in Tables 12.2-1, 15.4-1, and 15.4-2

11.4 SEISMIC GROUND MOTION VALUES

11.4.1 Mapped Acceleration Parameters

The parameters S_S and S_1 shall be determined from the 0.2 and 1 s spectral response accelerations shown on Figs. 22-1, 22-3, 22-5, and 22-6 for S_S and Figs. 22-2, 22-4, 22-5, and 22-6 for S_1. Where S_1 is less than or equal to 0.04 and S_S is less than or equal to 0.15, the structure is permitted to be assigned to Seismic Design Category A and is only required to comply with Section 11.7.

> **User Note:** Electronic values of mapped acceleration parameters, and other seismic design parameters, are provided at the USGS Web site at http://earthquake.usgs.gov/designmaps, or through the SEI Web site at http://content.seinstitute.org.

11.4.2 Site Class

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

11.4.3 Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

The MCE_R spectral response acceleration parameter for short periods (S_{MS}) and at 1 s (S_{M1}), adjusted for Site Class effects, shall be determined by Eqs. 11.4-1 and 11.4-2, respectively.

$$S_{MS} = F_a S_S \tag{11.4-1}$$

$$S_{M1} = F_v S_1 \tag{11.4-2}$$

where

S_S = the mapped MCE_R spectral response acceleration parameter at short periods as determined in accordance with Section 11.4.1, and

S_1 = the mapped MCE_R spectral response acceleration parameter at a period of 1 s as determined in accordance with Section 11.4.1

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

11.4.4 Design Spectral Acceleration Parameters

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

$$S_{DS} = \frac{2}{3} S_{MS} \tag{11.4-3}$$

$$S_{D1} = \frac{2}{3} S_{M1} \tag{11.4-4}$$

Table 11.4-1 Site Coefficient, F_a

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of S_S.

Table 11.4-2 Site Coefficient, F_v

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of S_1.

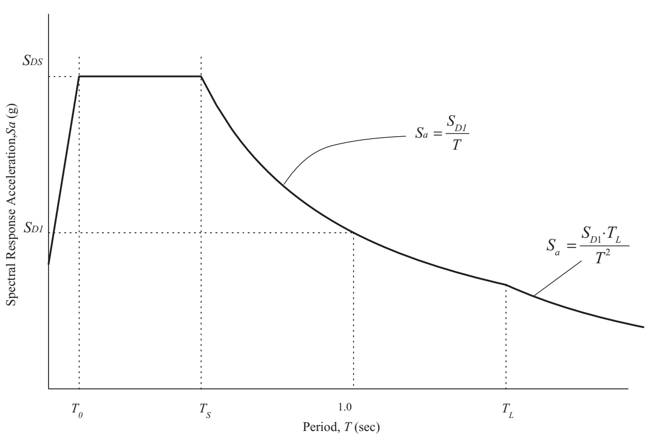


FIGURE 11.4-1 Design Response Spectrum.

11.4.5 Design Response Spectrum

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

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

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

2. For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response acceleration, S_a, shall be taken equal to S_{DS}.
3. For periods greater than T_S, and less than or equal to T_L, the design spectral response acceleration, S_a, shall be taken as given by Eq. 11.4-6:

$$S_a = \frac{S_{D1}}{T} \tag{11.4-6}$$

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

$$S_a = \frac{S_{D1}T_L}{T^2} \tag{11.4-7}$$

where

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

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

T = the fundamental period of the structure, s

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

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

T_L = long-period transition period (s) shown in Figs. 22-12 through 22-16.

11.4.6 Risk-Targeted Maximum Considered (MCE_R) Response Spectrum

Where an MCE_R response spectrum is required, it shall be determined by multiplying the design response spectrum by 1.5.

11.4.7 Site-Specific Ground Motion Procedures

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

11.5 IMPORTANCE FACTOR AND RISK CATEGORY

11.5.1 Importance Factor

An importance factor, I_C, shall be assigned to each structure in accordance with Table 1.5-2.

11.5.2 Protected Access for Risk Category IV

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

11.6 SEISMIC DESIGN CATEGORY

Structures shall be assigned a Seismic Design Category in accordance with this section.

Risk Category I, II, or III structures located where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category E. Risk Category IV structures located where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category F. All other structures shall be assigned to a Seismic Design Category based on their Risk Category and the design spectral response acceleration parameters, S_{DS} and S_{D1}, determined in accordance with Section 11.4.4. Each building and structure shall be assigned to the more severe Seismic Design Category in accordance with Table 11.6-1 or 11.6-2, irrespective of the fundamental period of vibration of the structure, T.

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

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, T_a, determined in accordance with Section 12.8.2.1 is less than $0.8T_s$, where T_s is determined in accordance with Section 11.4.5.
2. In each of two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.
3. Eq. 12.8-2 is used to determine the seismic response coefficient C_s.

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

	Risk Category	
Value of S_{DS}	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	D	D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

	Risk Category	
Value of S_{D1}	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \le S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$	D	D

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

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

11.7 DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORY A

Buildings and other structures assigned to Seismic Design Category A need only comply with the requirements of Section 1.4. Nonstructural components in SDC A are exempt from seismic design requirements. In addition, tanks assigned to Risk Category IV shall satisfy the freeboard requirement in Section 15.7.6.1.2.

11.8 GEOLOGIC HAZARDS AND GEOTECHNICAL INVESTIGATION

11.8.1 Site Limitation for Seismic Design Categories E and F

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

11.8.2 Geotechnical Investigation Report Requirements for Seismic Design Categories C through F

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

a. Slope instability,
b. Liquefaction,
c. Total and differential settlement, and
d. Surface displacement due to faulting or seismically induced lateral spreading or lateral flow.

The report shall contain recommendations for foundation designs or other measures to mitigate the effects of the previously mentioned hazards.

EXCEPTION: Where approved by the authority having jurisdiction, a site-specific geotechnical report is not required where prior evaluations of nearby sites with similar soil conditions provide direction relative to the proposed construction.

11.8.3 Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

The geotechnical investigation report for a structure assigned to Seismic Design Category D, E, or F shall include all of the following, as applicable:

1. The determination of dynamic seismic lateral earth pressures on basement and retaining walls due to design earthquake ground motions.
2. The potential for liquefaction and soil strength loss evaluated for site peak ground acceleration, earthquake magnitude, and source characteristics consistent with the MCE_G peak ground acceleration. Peak ground acceleration shall be determined based on either (1) a site-specific study taking into account soil amplification effects as specified in

Table 11.8-1 Site Coefficient F_{PGA}

	Mapped Maximum Considered Geometric Mean (MCE_G) Peak Ground Acceleration, PGA				
Site Class	PGA ≤ 0.1	PGA = 0.2	PGA = 0.3	PGA = 0.4	PGA ≥ 0.5
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of PGA.

Section 11.4.7 or (2) the peak ground acceleration PGA_M, from Eq. 11.8-1.

$$PGA_M = F_{PGA}\ PGA \qquad \text{(Eq. 11.8-1)}$$

where

PGA_M = MCE_G peak ground acceleration adjusted for Site Class effects.

PGA = Mapped MCE_G peak ground acceleration shown in Figs. 22-6 through 22-10.

F_{PGA} = Site coefficient from Table 11.8-1.

3. Assessment of potential consequences of liquefaction and soil strength loss, including, but not limited to, estimation of total and differential settlement, lateral soil movement, lateral soil loads on foundations, reduction in foundation soil-bearing capacity and lateral soil reaction, soil downdrag and reduction in axial and lateral soil reaction for pile foundations, increases in soil lateral pressures on retaining walls, and flotation of buried structures.
4. Discussion of mitigation measures such as, but not limited to, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements and forces, ground stabilization, or any combination of these measures and how they shall be considered in the design of the structure.

Chapter 12
SEISMIC DESIGN REQUIREMENTS FOR BUILDING STRUCTURES

12.1 STRUCTURAL DESIGN BASIS

12.1.1 Basic Requirements

The seismic analysis and design procedures to be used in the design of building structures and their members shall be as prescribed in this section. 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 12.6 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.

EXCEPTION: As an alternative, the simplified design procedures of Section 12.14 is permitted to be used in lieu of the requirements of Sections 12.1 through 12.12, subject to all of the limitations contained in Section 12.14.

12.1.2 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 12.1.1. The deformation of the structure shall not exceed the prescribed limits where the structure is subjected to the design seismic forces.

12.1.3 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 0.133 times the short period design spectral response acceleration parameter, S_{DS}, times the weight of the smaller portion or 5 percent of the portion's weight, whichever is greater. 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.

12.1.4 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.

12.1.5 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 12.13.

12.1.6 Material Design and Detailing Requirements

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

12.2 STRUCTURAL SYSTEM SELECTION

12.2.1 Selection and Limitations

The basic lateral and vertical seismic force-resisting system shall conform to one of the types indicated in Table 12.2-1 or a combination of systems as permitted in Sections 12.2.2, 12.2.3, and 12.2.4. 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 12.2-1. The appropriate response modification coefficient, R, overstrength factor, Ω_0, and the deflection amplification factor, C_d, indicated in Table 12.2-1 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 12.2-1 and the additional requirements set forth in Chapter 14.

Seismic force-resisting systems not contained in Table 12.2-1 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 12.2-1 for equivalent values of response modification coefficient, R, overstrength factor, Ω_0, and deflection amplification factor, C_d.

12.2.2 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 Ω_0 coefficients shall apply to each system, including the structural system limitations contained in Table 12.2-1.

12.2.3 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 12.2-1 shall apply and the design shall comply with the requirements of this section.

12.2.3.1 R, C_d, *and* Ω_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, Ω_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, Ω_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, Ω_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.
3. Detached one- and two-family dwellings of light-frame construction.

12.2.3.2 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 values of R and ρ.

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, $\Omega_0{}^g$	Deflection Amplification Factor, $C_d{}^b$	Structural System Limitations Including Structural Height, h_n (ft) Limits[c] — Seismic Design Category: B	C	D^d	E^d	F^e
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls[l, m]	14.2	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls[l]	14.2	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls[l]	14.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls[l]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls [l]	14.2	4	2½	4	NL	NL	40[k]	40[k]	40[k]
6. Ordinary precast shear walls[l]	14.2	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Ordinary reinforced AAC masonry shear walls	14.4	2	2½	2	NL	35	NP	NP	NP
14. Ordinary plain AAC masonry shear walls	14.4	1½	2½	1½	NL	NP	NP	NP	NP
15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1 and 14.5	6½	3	4	NL	NL	65	65	65
16. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6½	3	4	NL	NL	65	65	65
17. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	2½	2	NL	NL	35	NP	NP
18. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35[j]	35[j]	NP[j]

Continued

Table 12.2-1 (*Continued*)

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, $\Omega_0{}^g$	Deflection Amplification Factor, $C_d{}^b$	Structural System Limitations Including Structural Height, h_n (ft) Limits[c] — Seismic Design Category: B	C	D^d	E^d	F^e
4. Special reinforced concrete shear walls[l,m]	14.2	6	2½	5	NL	NL	160	160	100
5. Ordinary reinforced concrete shear walls[l]	14.2	5	2½	4½	NL	NL	NP	NP	NP
6. Detailed plain concrete shear walls[l]	14.2 and 14.2.2.8	2	2½	2	NL	NP	NP	NP	NP
7. Ordinary plain concrete shear walls[l]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
8. Intermediate precast shear walls[l]	14.2	5	2½	4½	NL	NL	40[k]	40[k]	40[k]
9. Ordinary precast shear walls[l]	14.2	4	2½	4	NL	NP	NP	NP	NP
10. Steel and concrete composite eccentrically braced frames	14.3	8	2 ½	4	NL	NL	160	160	100
11. Steel and concrete composite special concentrically braced frames	14.3	5	2	4½	NL	NL	160	160	100
12. Steel and concrete composite ordinary braced frames	14.3	3	2	3	NL	NL	NP	NP	NP
13. Steel and concrete composite plate shear walls	14.3	6½	2½	5½	NL	NL	160	160	100
14. Steel and concrete composite special shear walls	14.3	6	2½	5	NL	NL	160	160	100
15. Steel and concrete composite ordinary shear walls	14.3	5	2½	4½	NL	NL	NP	NP	NP
16. Special reinforced masonry shear walls	14.4	5½	2½	4	NL	NL	160	160	100
17. Intermediate reinforced masonry shear walls	14.4	4	2½	4	NL	NL	NP	NP	NP
18. Ordinary reinforced masonry shear walls	14.4	2	2½	2	NL	160	NP	NP	NP
19. Detailed plain masonry shear walls	14.4	2	2½	2	NL	NP	NP	NP	NP
20. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
21. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
22. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	7	2½	4½	NL	NL	65	65	65
23. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	7	2½	4½	NL	NL	65	65	65
24. Light-frame walls with shear panels of all other materials	14.1and 14.5	2½	2½	2½	NL	NL	35	NP	NP
25. Steel buckling-restrained braced frames	14.1	8	2½	5	NL	NL	160	160	100
26. Steel special plate shear walls	14.1	7	2	6	NL	NL	160	160	100

Table 12.2-1 (*Continued*)

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R[a]	Overstrength Factor, Ω_0[g]	Deflection Amplification Factor, C_d[b]	Structural System Limitations Including Structural Height, h_n (ft) Limits[c]				
					Seismic Design Category				
					B	C	D[d]	E[d]	F[e]
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Steel intermediate moment frames	12.2.5.7 and 14.1	4½	3	4	NL	NL	35[h]	NP[h]	NP[h]
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP[i]	NP[i]	NP[i]
5. Special reinforced concrete moment frames[n]	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Steel and concrete composite special moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Steel and concrete composite intermediate moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Steel and concrete composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Steel and concrete composite ordinary moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP
12. Cold-formed steel—special bolted moment frame[p]	14.1	3½	3[o]	3½	35	35	35	35	35
D. DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
1. Steel eccentrically braced frames	14.1	8	2½	4	NL	NL	NL	NL	NL
2. Steel special concentrically braced frames	14.1	7	2½	5½	NL	NL	NL	NL	NL
3. Special reinforced concrete shear walls[l]	14.2	7	2½	5½	NL	NL	NL	NL	NL
4. Ordinary reinforced concrete shear walls[l]	14.2	6	2½	5	NL	NL	NP	NP	NP
5. Steel and concrete composite eccentrically braced frames	14.3	8	2½	4	NL	NL	NL	NL	NL
6. Steel and concrete composite special concentrically braced frames	14.3	6	2½	5	NL	NL	NL	NL	NL

Continued

Table 12.2-1 (*Continued*)

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, $\Omega_0{}^g$	Deflection Amplification Factor, $C_d{}^b$	Structural System Limitations Including Structural Height, h_n (ft) Limits[c] — Seismic Design Category: B	C	D^d	E^d	F^e
7. Steel and concrete composite plate shear walls	14.3	7½	2½	6	NL	NL	NL	NL	NL
8. Steel and concrete composite special shear walls	14.3	7	2½	6	NL	NL	NL	NL	NL
9. Steel and concrete composite ordinary shear walls	14.3	6	2½	5	NL	NL	NP	NP	NP
10. Special reinforced masonry shear walls	14.4	5½	3	5	NL	NL	NL	NL	NL
11. Intermediate reinforced masonry shear walls	14.4	4	3	3½	NL	NL	NP	NP	NP
12. Steel buckling-restrained braced frames	14.1	8	2½	5	NL	NL	NL	NL	NL
13. Steel special plate shear walls	14.1	8	2½	6½	NL	NL	NL	NL	NL
E. DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
1. Steel special concentrically braced frames[f]	14.1	6	2½	5	NL	NL	35	NP	NP
2. Special reinforced concrete shear walls[l]	14.2	6½	2½	5	NL	NL	160	100	100
3. Ordinary reinforced masonry shear walls	14.4	3	3	2½	NL	160	NP	NP	NP
4. Intermediate reinforced masonry shear walls	14.4	3½	3	3	NL	NL	NP	NP	NP
5. Steel and concrete composite special concentrically braced frames	14.3	5½	2½	4½	NL	NL	160	100	NP
6. Steel and concrete composite ordinary braced frames	14.3	3½	2½	3	NL	NL	NP	NP	NP
7. Steel and concrete composite ordinary shear walls	14.3	5	3	4½	NL	NL	NP	NP	NP
8. Ordinary reinforced concrete shear walls[l]	14.2	5½	2½	4½	NL	NL	NP	NP	NP
F. SHEAR WALL-FRAME INTERACTIVE SYSTEM WITH ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS[l]	12.2.5.8 and 14.2	4½	2½	4	NL	NP	NP	NP	NP

Table 12.2-1 (*Continued*)

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, $\Omega_0{}^g$	Deflection Amplification Factor, $C_d{}^b$	Structural System Limitations Including Structural Height, h_n (ft) Limits[c] — Seismic Design Category: B	C	D^d	E^d	F^e
G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR:	12.2.5.2								
1. Steel special cantilever column systems	14.1	2½	1¼	2½	35	35	35	35	35
2. Steel ordinary cantilever column systems	14.1	1¼	1¼	1¼	35	35	NP[i]	NP[i]	NP[i]
3. Special reinforced concrete moment frames[n]	12.2.5.5 and 14.2	2½	1¼	2½	35	35	35	35	35
4. Intermediate reinforced concrete moment frames	14.2	1½	1¼	1½	35	35	NP	NP	NP
5. Ordinary reinforced concrete moment frames	14.2	1	1¼	1	35	NP	NP	NP	NP
6. Timber frames	14.5	1½	1½	1½	35	35	35	NP	NP
H. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE, EXCLUDING CANTILEVER COLUMN SYSTEMS	14.1	3	3	3	NL	NL	NP	NP	NP

[a]Response modification coefficient, R, for use throughout the standard. Note R reduces forces to a strength level, not an allowable stress level.
[b]Deflection amplification factor, C_d, for use in Sections 12.8.6, 12.8.7, and 12.9.2.
[c]NL = Not Limited and NP = Not Permitted. For metric units use 30.5 m for 100 ft and use 48.8 m for 160 ft.
[d]See Section 12.2.5.4 for a description of seismic force-resisting systems limited to buildings with a structural height, h_n, of 240 ft (73.2 m) or less.
[e]See Section 12.2.5.4 for seismic force-resisting systems limited to buildings with a structural height, h_n, of 160 ft (48.8 m) or less.
[f]Ordinary moment frame is permitted to be used in lieu of intermediate moment frame for Seismic Design Categories B or C.
[g]Where the tabulated value of the overstrength factor, Ω_0, is greater than or equal to 2½, Ω_o is permitted to be reduced by subtracting the value of 1/2 for structures with flexible diaphragms.
[h]See Section 12.2.5.7 for limitations in structures assigned to Seismic Design Categories D, E, or F.
[i] See Section 12.2.5.6 for limitations in structures assigned to Seismic Design Categories D, E, or F.
[j]Steel ordinary concentrically braced frames are permitted in single-story buildings up to a structural height, h_n, of 60 ft (18.3 m) where the dead load of the roof does not exceed 20 psf
(0.96 kN/m^2) and in penthouse structures.
[k]An increase in structural height, h_n, to 45 ft (13.7 m) is permitted for single story storage warehouse facilities.
[l]In Section 2.2 of ACI 318. A shear wall is defined as a structural wall.
[m]In Section 2.2 of ACI 318. The definition of “special structural wall” includes precast and cast-in-place construction.
[n]In Section 2.2 of ACI 318. The definition of “special moment frame” includes precast and cast-in-place construction.
[o]Alternately, the seismic load effect with overstrength, E_{mh}, is permitted to be based on the expected strength determined in accordance with AISI S110.
[p]Cold-formed steel – special bolted moment frames shall be limited to one-story in height in accordance with AISI S110.

d. The lower portion shall be designed as a separate structure using the appropriate values of R and ρ. The reactions from the upper portion shall be those determined from the analysis of the upper portion amplified by the ratio of the R/ρ of the upper portion over R/ρ 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.

12.2.3.3* R, C_d, *and* Ω_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, Ω_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.

12.2.4 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 Chapter 12 required by the highest response modification coefficient, R, of the connected framing systems.

12.2.5 System Specific Requirements

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

12.2.5.1 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.

12.2.5.2 Cantilever Column Systems

Cantilever column systems are permitted as indicated in Table 12.2-1 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 12.4.3.

12.2.5.3 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 12.8 and varying uniformly to a moment at the top equal to one-half the calculated bending moment at the base.

12.2.5.4 Increased Structural Height Limit for Steel Eccentrically Braced Frames, Steel Special Concentrically Braced Frames, Steel Buckling-restrained Braced Frames, Steel Special Plate Shear Walls and Special Reinforced Concrete Shear Walls

The limits on structural height, h_n, in Table 12.2-1 are permitted to be increased from 160 ft (50 m) to 240 ft (75 m) for structures assigned to Seismic Design Categories D or E and from 100 ft (30 m) to 160 ft (50 m) for structures assigned to Seismic Design Category F provided the seismic force-resisting systems are limited to steel eccentrically braced frames, steel special concentrically braced frames, steel buckling-restrained braced frames, steel special plate shear walls, or special reinforced concrete cast-in-place shear walls and both of the following requirements are met:

1. The structure shall not have an extreme torsional irregularity as defined in Table 12.2-1 (horizontal structural irregularity Type 1b).
2. The steel eccentrically braced frames, steel special concentrically braced frames, steel buckling-restrained braced frames, steel special plate shear walls or special reinforced cast-in-place concrete shear walls in any one plane shall resist no more than 60 percent of the total seismic forces in each direction, neglecting accidental torsional effects.

12.2.5.5 Special Moment Frames in Structures Assigned to Seismic Design Categories D through F

For structures assigned to Seismic Design Categories D, E, or F, a special moment frame that is used but not required by Table 12.2-1 shall not be discontinued and supported by a more rigid system with a lower response modification coefficient, R, unless the requirements of Sections 12.3.3.2 and 12.3.3.4 are met. Where a special moment frame is required by Table 12.2-1, the frame shall be continuous to the base.

12.2.5.6 Steel Ordinary Moment Frames

12.2.5.6.1 Seismic Design Category D or E.

a. Single-story steel ordinary moment frames in structures assigned to Seismic Design Category D or E are permitted up to a structural height, h_n, of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior walls more than 35 ft (10.6 m) above the base tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).

   EXCEPTION: Single-story structures with steel ordinary moment frames whose purpose is to enclose equipment or machinery and whose occupants are engaged in maintenance or monitoring of that equipment, machinery, or their associated processes shall be permitted to be of unlimited height where the sum of the dead and equipment loads supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior wall system including exterior columns more than 35 ft (10.6 m) above the base shall not exceed 20 psf (0.96 kN/m^2). For determining compliance with the exterior wall or roof load limits, the weight of equipment or machinery, including cranes, not self-supporting for all loads shall be assumed fully tributary to the area of the adjacent exterior wall or roof not to exceed 600 ft^2 (55.8 m^2) regardless of their height above the base of the structure.

b. Steel ordinary moment frames in structures assigned to Seismic Design Category D or E not meeting the limitations set forth in Section 12.2.5.6.1.a are permitted within light-frame construction up to a structural height, h_n, of 35 ft (10.6 m) where neither the roof dead load nor the dead load of any floor above the base supported by and tributary to the moment frames exceeds 35 psf (1.68 kN/m^2). In addition, the dead load of the exterior walls tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).

12.2.5.6.2 Seismic Design Category F. Single-story steel ordinary moment frames in structures assigned to Seismic Design Category F are permitted up to a structural height, h_n, of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior walls tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).

12.2.5.7 Steel Intermediate Moment Frames

12.2.5.7.1 Seismic Design Category D

a. Single-story steel intermediate moment frames in structures assigned to Seismic Design Category D are permitted up to a structural height, h_n, of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior walls more than 35 ft (10.6 m) above the base tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).

   EXCEPTION: Single-story structures with steel intermediate moment frames whose purpose is to enclose equipment or machinery and whose occupants are engaged in maintenance or monitoring of that equipment, machinery, or their associated processes shall be permitted to be of unlimited height where the sum of the dead and equipment loads supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior wall system including exterior columns more than 35 ft (10.6 m) above the base shall not exceed 20 psf (0.96 kN/m^2). For determining compliance with the exterior wall or roof load limits, the weight of equipment or machinery, including cranes, not self-supporting for all loads shall be assumed fully tributary to the area of the adjacent exterior wall or roof not to exceed 600 ft^2 (55.8 m^2) regardless of their height above the base of the structure.

b. Steel intermediate moment frames in structures assigned to Seismic Design Category D not meeting the limitations set forth in Section 12.2.5.7.1.a are permitted up to a structural height, h_n, of 35 ft (10.6 m).

12.2.5.7.2 Seismic Design Category E.

a. Single-story steel intermediate moment frames in structures assigned to Seismic Design Category E are permitted up to a structural height, h_n, of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior walls more than 35 ft (10.6 m) above the base tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).

   EXCEPTION: Single-story structures with steel intermediate moment frames whose purpose is to enclose equipment or machinery and whose occupants are engaged in maintenance or monitoring of that equipment, machinery, or their associated processes shall be permitted to be of unlimited height where the sum of the dead and equipment loads supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior wall system including exterior columns more than 35 ft (10.6 m) above the base shall not exceed 20 psf (0.96 kN/m^2). For determining compliance with the exterior wall or roof load limits, the weight of equipment or machinery, including cranes, not self-supporting for all loads shall be assumed fully tributary to the area of the adjacent exterior wall or roof not to exceed 600 ft^2 (55.8 m^2) regardless of their height above the base of the structure.

b. Steel intermediate moment frames in structures assigned to Seismic Design Category E not meeting the limitations set forth in Section 12.2.5.7.2.a are permitted up to a structural height, h_n, of 35 ft (10.6 m) where neither the roof dead load nor the dead load of any floor above the base supported by and tributary to the moment frames exceeds 35 psf (1.68 kN/m^2). In addition, the dead load of the exterior walls tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).

12.2.5.7.3 Seismic Design Category F.

a. Single-story steel intermediate moment frames in structures assigned to Seismic Design Category F are permitted up to a structural height, h_n, of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior walls tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).
b. Steel intermediate moment frames in structures assigned to Seismic Design Category F not meeting the limitations set forth in Section 12.2.5.7.3.a are permitted within light-frame construction up to a structural height, h_n, of 35 ft (10.6 m) where neither the roof dead load nor the dead load of any floor above the base supported by and tributary to the moment frames exceeds 35 psf (1.68 kN/m^2). In addition, the dead load of the exterior walls tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).

12.2.5.8 Shear Wall-Frame Interactive Systems

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

12.3 DIAPHRAGM FLEXIBILITY, CONFIGURATION IRREGULARITIES, AND REDUNDANCY

12.3.1 Diaphragm Flexibility

The structural analysis shall consider the relative 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 12.3.1.1, 12.3.1.2, or 12.3.1.3, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semirigid modeling assumption).

12.3.1.1 Flexible Diaphragm Condition

Diaphragms constructed of untopped steel decking or wood structural panels are permitted to be idealized as flexible 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 12.12-1.

12.3.1.2 Rigid Diaphragm Condition

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

12.3.1.3 Calculated Flexible Diaphragm Condition

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

12.3.2 Irregular and Regular Classification

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

12.3.2.1 Horizontal Irregularity

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

12.3.2.2 Vertical Irregularity

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

EXCEPTIONS:

1. Vertical structural irregularities of Types 1a, 1b, and 2 in Table 12.3-2 do not apply where no story drift ratio under design lateral seismic force is greater than 130 percent of the story drift ratio of the next story above. Torsional effects need not be considered in the calculation of story drifts. The story drift ratio relationship for the top two stories of the structure are not required to be evaluated.
2. Vertical structural irregularities of Types 1a, 1b, and 2 in Table 12.3-2 are not required to be considered for one-story buildings in any seismic design category or for two-story buildings assigned to Seismic Design Categories B, C, or D.

12.3.3 Limitations and Additional Requirements for Systems with Structural Irregularities

12.3.3.1 Prohibited Horizontal and Vertical Irregularities for Seismic Design Categories D through F

Structures assigned to Seismic Design Category E or F having horizontal irregularity Type 1b of Table 12.3-1 or vertical irregularities Type 1b, 5a, or 5b of Table 12.3-2 shall not be permitted. Structures assigned to Seismic Design Category D having vertical irregularity Type 5b of Table 12.3-2 shall not be permitted.

12.3.3.2 Extreme Weak Stories

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

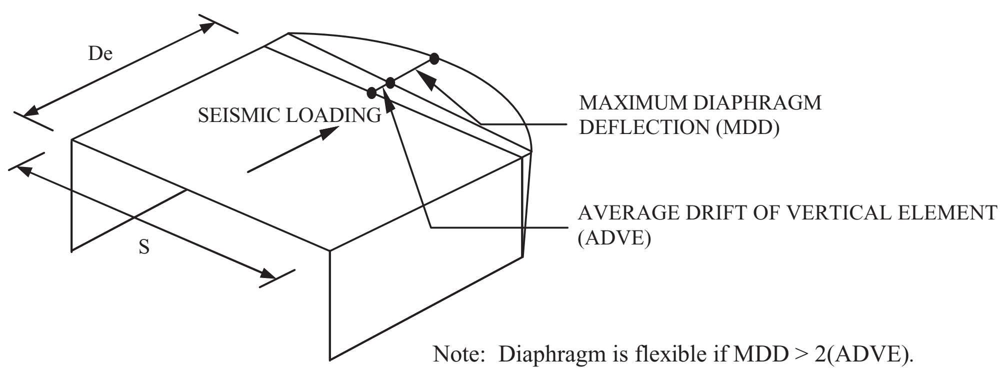


FIGURE 12.3-1 Flexible Diaphragm

Table 12.3-1 Horizontal Structural Irregularities

Type	Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity:** 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.	12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F B, C, D, E, and F C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity:** 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.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D
2.	**Reentrant Corner Irregularity:** Reentrant corner irregularity is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
3.	**Diaphragm Discontinuity Irregularity:** Diaphragm discontinuity irregularity is defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one having a cutout or open area greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
4.	**Out-of-Plane Offset Irregularity:** 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.	12.3.3.3 12.3.3.4 12.7.3 Table 12.6-1 Section 16.2.2	B, C, D, E, and F D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
5.	**Nonparallel System Irregularity:** 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.	12.5.3 12.7.3 Table 12.6-1 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

EXCEPTION: The limit does not apply where the "weak" story is capable of resisting a total seismic force equal to Ω_0 times the design force prescribed in Section 12.8.

12.3.3.3 Elements Supporting Discontinuous Walls or Frames

Columns, beams, trusses, or slabs supporting discontinuous walls or frames of structures having horizontal irregularity Type 4 of Table 12.3-1 or vertical irregularity Type 4 of Table 12.3-2 shall be designed to resist the seismic load effects including overstrength factor of Section 12.4.3. 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.

12.3.3.4 Increase in Forces Due to Irregularities for Seismic Design Categories D through F

For structures assigned to Seismic Design Category D, E, or F and having a horizontal structural irregularity of Type 1a, 1b, 2, 3, or 4 in Table 12.3-1 or a vertical structural irregularity of Type 4 in Table 12.3-2, the design forces determined from Section 12.10.1.1 shall be increased 25 percent for the following elements of the seismic force-resisting system:

1. Connections of diaphragms to vertical elements and to collectors.
2. Collectors and their connections, including connections to vertical elements, of the seismic force-resisting system.

Table 12.3-2 Vertical Structural Irregularities

Type	Description	Reference Section	Seismic Design Category Application
1a.	**Stiffness-Soft Story Irregularity:** Stiffness-soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	Table 12.6-1	D, E, and F
1b.	**Stiffness-Extreme Soft Story Irregularity:** Stiffness-extreme soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	12.3.3.1 Table 12.6-1	E and F D, E, and F
2.	**Weight (Mass) Irregularity:** Weight (mass) irregularity is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	Table 12.6-1	D, E, and F
3.	**Vertical Geometric Irregularity:** Vertical geometric irregularity is defined to exist where the horizontal dimension of the seismic force-resisting system in any story is more than 130% of that in an adjacent story.	Table 12.6-1	D, E, and F
4.	**In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity:** In-plane discontinuity in vertical lateral force-resisting elements irregularity is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on a supporting beam, column, truss, or slab.	12.3.3.3 12.3.3.4 Table 12.6-1	B, C, D, E, and F D, E, and F D, E, and F
5a.	**Discontinuity in Lateral Strength–Weak Story Irregularity:** Discontinuity in lateral strength–weak story irregularity is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 Table 12.6-1	E and F D, E, and F
5b.	**Discontinuity in Lateral Strength–Extreme Weak Story Irregularity:** Discontinuity in lateral strength–extreme weak story irregularity is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 12.3.3.2 Table 12.6-1	D, E, and F B and C D, E, and F

EXCEPTION:

Forces calculated using the seismic load effects including overstrength factor of Section 12.4.3 need not be increased.

12.3.4 Redundancy

A redundancy factor, ρ, shall be assigned to the seismic force-resisting system in each of two orthogonal directions for all structures in accordance with this section.

12.3.4.1 Conditions Where Value of ρ is 1.0

The value of ρ is permitted to equal 1.0 for the following:

1. Structures assigned to Seismic Design Category B or C.
2. Drift calculation and P-delta effects.
3. Design of nonstructural components.
4. Design of nonbuilding structures that are not similar to buildings.
5. Design of collector elements, splices, and their connections for which the seismic load effects including overstrength factor of Section 12.4.3 are used.
6. Design of members or connections where the seismic load effects including overstrength factor of Section 12.4.3 are required for design.
7. Diaphragm loads determined using Eq. 12.10-1.
8. Structures with damping systems designed in accordance with Chapter 18.
9. Design of structural walls for out-of-plane forces, including their anchorage.

Table 12.3-3 Requirements for Each Story Resisting More than 35% of the Base Shear

Lateral Force-Resisting Element	Requirement
Braced frames	Removal of an individual brace, or connection thereto, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).
Moment frames	Loss of moment resistance at the beam-to-column connections at both ends of a single beam would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).
Shear walls or wall piers with a height-to-length ratio greater than 1.0	Removal of a shear wall or wall pier with a height-to-length ratio greater than 1.0 within any story, or collector connections thereto, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b). The shear wall and wall pier height-to-length ratios are determined as shown in Figure 12.3-2.
Cantilever columns	Loss of moment resistance at the base connections of any single cantilever column would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).
Other	No requirements

12.3.4.2 Redundancy Factor, ρ, for Seismic Design Categories D through F

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

a. Each story resisting more than 35 percent of the base shear in the direction of interest shall comply with Table 12.3-3.
b. Structures that are regular in plan at all levels provided that the seismic force-resisting systems consist of at least two bays of seismic force-resisting perimeter framing on each side of the structure in each orthogonal direction at each story resisting more than 35 percent of the base shear. The number of bays for a shear wall shall be calculated as the length of shear wall divided by the story height or two times the length of shear wall divided by the story height, h_{sx}, for light-frame construction.

12.4 SEISMIC LOAD EFFECTS AND COMBINATIONS

12.4.1 Applicability

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

12.4.2 Seismic Load Effect

The seismic load effect, E, shall be determined in accordance with the following:

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

$$E = E_h + E_v \tag{12.4-1}$$

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

$$E = E_h - E_v \tag{12.4-2}$$

where

E = seismic load effect
E_h = effect of horizontal seismic forces as defined in Section 12.4.2.1
E_v = effect of vertical seismic forces as defined in Section 12.4.2.2

12.4.2.1 Horizontal Seismic Load Effect

The horizontal seismic load effect, E_h, shall be determined in accordance with Eq. 12.4-3 as follows:

$$E_h = \rho Q_E \tag{12.4-3}$$

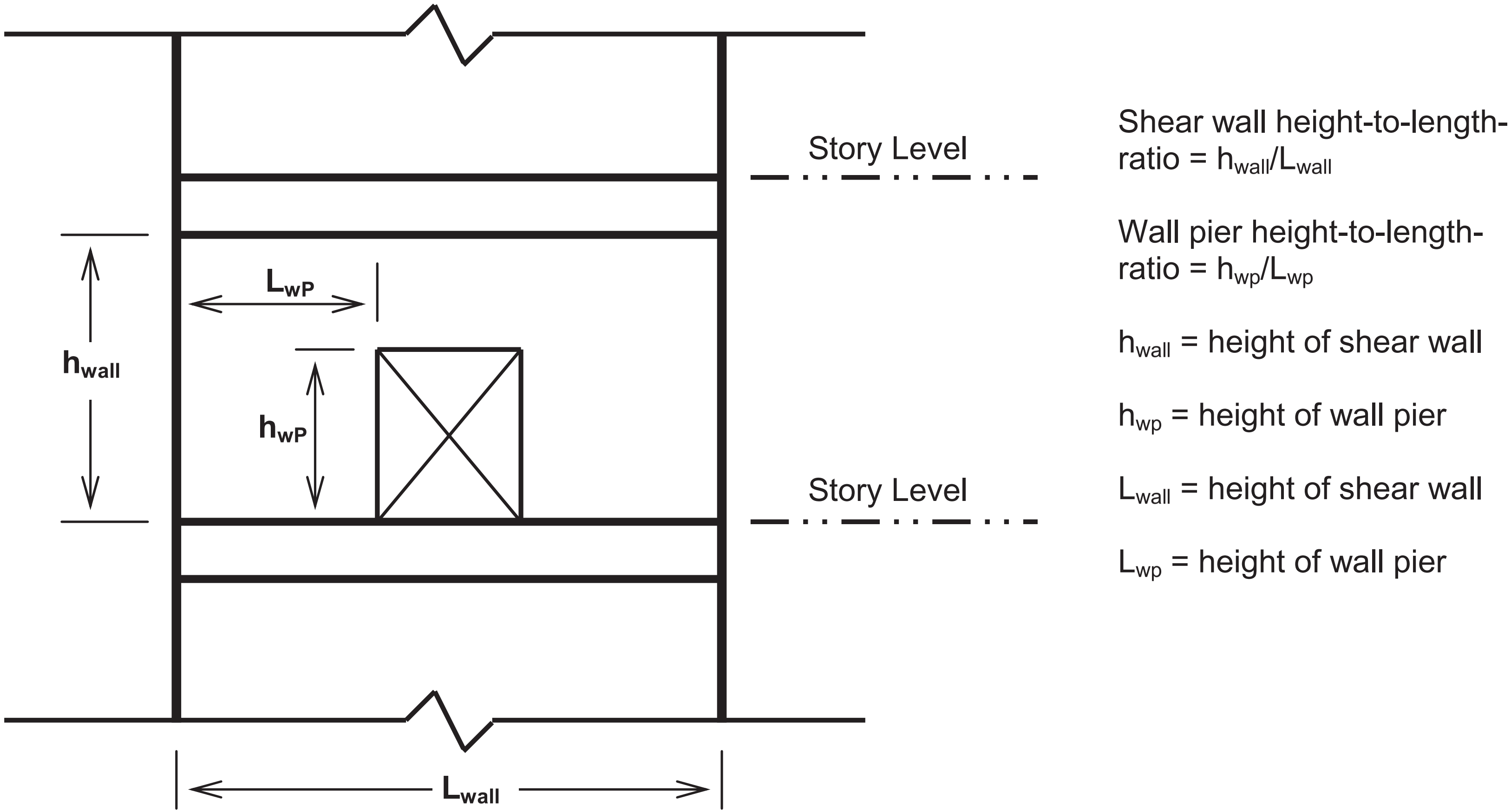


FIGURE 12.3-2 Shear Wall and Wall Pier Height-To-Length Ratio Determination

where

Q_E = effects of horizontal seismic forces from V or F_p. Where required by Section 12.5.3 or 12.5.4, such effects shall result from application of horizontal forces simultaneously in two directions at right angles to each other

ρ = redundancy factor, as defined in Section 12.3.4

12.4.2.2 Vertical Seismic Load Effect

The vertical seismic load effect, E_v, shall be determined in accordance with Eq. 12.4-4 as follows:

$$E_v = 0.2S_{DS}D \qquad (12.4\text{-}4)$$

where

S_{DS} = design spectral response acceleration parameter at short periods obtained from Section 11.4.4

D = effect of dead load

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

1. In Eqs. 12.4-1, 12.4-2, 12.4-5, and 12.4-6 where S_{DS} is equal to or less than 0.125.
2. In Eq. 12.4-2 where determining demands on the soil–structure interface of foundations.

12.4.2.3 Seismic Load Combinations

Where the prescribed seismic load effect, E, defined in Section 12.4.2 is combined with the effects of other loads as set forth in Chapter 2, the following seismic load combinations for structures not subject to flood or atmospheric ice loads shall be used in lieu of the seismic load combinations in either Section 2.3.2 or 2.4.1:

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

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

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/m^2), 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. $(1.0 + 0.14S_{DS})D + H + F + 0.7\rho Q_E$
6. $(1.0 + 0.10S_{DS})D + H + F + 0.525\rho Q_E + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
8. $(0.6 - 0.14S_{DS})D + 0.7\rho Q_E + H$

12.4.3 Seismic Load Effect Including Overstrength Factor

Where specifically required, conditions requiring overstrength factor applications shall be determined in accordance with the following:

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

$$E_m = E_{mh} + E_v \qquad (12.4\text{-}5)$$

2. For use in load combination 7 in Section 2.3.2 or load combination 8 in Section 2.4.1, E shall be taken equal to E_m as determined in accordance with Eq. 12.4-6 as follows:

$$E_m = E_{mh} - E_v \qquad (12.4\text{-}6)$$

where

E_m = seismic load effect including overstrength factor

E_{mh} = effect of horizontal seismic forces including overstrength factor as defined in Section 12.4.3.1

E_v = vertical seismic load effect as defined in Section 12.4.2.2

12.4.3.1 Horizontal Seismic Load Effect with Overstrength Factor

The horizontal seismic load effect with overstrength factor, E_{mh}, shall be determined in accordance with Eq. 12.4-7 as follows:

$$E_{mh} = \Omega_o Q_E \qquad (12.4\text{-}7)$$

where

Q_E = effects of horizontal seismic forces from V, F_{px}, or F_p as specified in Sections 12.8.1, 12.10, or 13.3.1. Where required by Section 12.5.3 or 12.5.4, such effects shall result from application of horizontal forces simultaneously in two directions at right angles to each other.

Ω_o = overstrength factor

EXCEPTION: The value of E_{mh} need not exceed the maximum force that can develop in the element as determined by a rational, plastic mechanism analysis or nonlinear response analysis utilizing realistic expected values of material strengths.

12.4.3.2 Load Combinations with Overstrength Factor

Where the seismic load effect with overstrength factor, E_m, defined in Section 12.4.3, is combined with the effects of other loads as set forth in Chapter 2, the following seismic load combination for structures not subject to flood or atmospheric ice loads shall be used in lieu of the seismic load combinations in either Section 2.3.2 or 2.4.1:

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

5. $(1.2 + 0.2S_{DS})D + \Omega_o Q_E + L + 0.2S$
7. $(0.9 - 0.2S_{DS})D + \Omega_o Q_E + 1.6H$

NOTES:

1. The load factor on L in combination 5 is permitted to equal 0.5 for all occupancies in which L_o in Table 4-1 is less than or equal to 100 psf (4.79 kN/m^2), with the exception of garages or areas occupied as places of public assembly.
2. The load factor on H shall be set equal to zero in combination 7 if the structural action due to H counteracts that due to E. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.

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

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

12.4.3.3 Allowable Stress Increase for Load Combinations with Overstrength

Where allowable stress design methodologies are used with the seismic load effect defined in Section 12.4.3 applied in load combinations 5, 6, or 8 of Section 2.4.1, allowable stresses are permitted to be determined using an allowable stress increase of 1.2. This increase shall not be combined with increases in allowable stresses or load combination reductions otherwise permitted by this standard or the material reference document except for increases due to adjustment factors in accordance with AF&PA NDS.

12.4.4 Minimum Upward Force for Horizontal Cantilevers for Seismic Design Categories D through F

In structures assigned to Seismic Design Category D, E, or F, horizontal cantilever structural members shall be designed for a minimum net upward force of 0.2 times the dead load in addition to the applicable load combinations of Section 12.4.

12.5 DIRECTION OF LOADING

12.5.1 Direction of Loading Criteria

The directions of application of seismic forces used in the design shall be those which will produce the most critical load effects. It is permitted to satisfy this requirement using the procedures of Section 12.5.2 for Seismic Design Category B, Section 12.5.3 for Seismic Design Category C, and Section 12.5.4 for Seismic Design Categories D, E, and F.

12.5.2 Seismic Design Category B

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

12.5.3 Seismic Design Category C

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

a. **Orthogonal Combination Procedure.** The structure shall be analyzed using the equivalent lateral force analysis procedure of Section 12.8, the modal response spectrum analysis procedure of Section 12.9, or the linear response history procedure of Section 16.1, as permitted under Section 12.6, with the loading applied independently in any two orthogonal directions. The requirement of Section 12.5.1 is deemed satisfied if members and their foundations are designed for 100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction. The combination requiring the maximum component strength shall be used.
b. **Simultaneous Application of Orthogonal Ground Motion.** The structure shall be analyzed

using the linear response history procedure of Section 16.1 or the nonlinear response history procedure of Section 16.2, as permitted by Section 12.6, with orthogonal pairs of ground motion acceleration histories applied simultaneously.

12.5.4 Seismic Design Categories D through F

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

12.6 ANALYSIS PROCEDURE SELECTION

The structural analysis required by Chapter 12 shall consist of one of the types permitted in Table 12.6-1, based on the structure's seismic design category, structural system, dynamic properties, and regularity, or with the approval of the authority having jurisdiction, an alternative generally accepted procedure is permitted to be used. The analysis procedure selected shall be completed in accordance with the requirements of the corresponding section referenced in Table 12.6-1.

12.7 MODELING CRITERIA

12.7.1 Foundation Modeling

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

12.7.2 Effective Seismic Weight

The effective seismic weight, W, 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.

Table 12.6-1 Permitted Analytical Procedures

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Analysis, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9[a]	Seismic Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding 2 stories above the base	P	P	P
	Structures of light frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft in structural height	P	P	P
	Structures exceeding 160 ft in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12.3-1 or vertical irregularities of Type 4, 5a, or 5b in Table 12.3-2	P	P	P
	All other structures	NP	P	P

[a]P: Permitted; NP: Not Permitted; $T_s = S_{D1}/S_{DS}$.

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/m^2) 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/m^2), 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.

12.7.3 Structural Modeling

A mathematical model of the structure shall be constructed for the purpose of determining member forces and structure displacements resulting from applied loads and any imposed displacements or 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 12.3-1 shall be analyzed using a 3-D representation. Where a 3-D model is used, a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and rotation about the vertical axis shall be included at each level of the structure. Where the diaphragms have not been classified as rigid or flexible in accordance with Section 12.3.1, the model shall include representation of the diaphragm's stiffness characteristics and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response.

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

12.7.4 Interaction Effects

Moment-resisting frames that are enclosed or adjoined by elements that are more rigid and not considered to be part of the seismic force-resisting system shall be designed so that the action or failure of those elements will not impair the vertical load and seismic force-resisting capability of the frame. The design shall provide for the effect of these rigid elements on the structural system at structural deformations corresponding to the design story drift (Δ) as determined in Section 12.8.6. In addition, the effects of these elements shall be considered where determining whether a structure has one or more of the irregularities defined in Section 12.3.2.

12.8 EQUIVALENT LATERAL FORCE PROCEDURE

12.8.1 Seismic Base Shear

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

$$V = C_s W \qquad (12.8\text{-}1)$$

where

C_s = the seismic response coefficient determined in accordance with Section 12.8.1.1
W = the effective seismic weight per Section 12.7.2

12.8.1.1 Calculation of Seismic Response Coefficient

The seismic response coefficient, C_s, shall be determined in accordance with Eq. 12.8-2.

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \qquad (12.8\text{-}2)$$

where

S_{DS} = 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 12.2-1
I_e = the importance factor determined in accordance with Section 11.5.1

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

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \quad \text{for} \quad T \le T_L \qquad (12.8\text{-}3)$$

$$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)} \quad \text{for} \quad T > T_L \qquad (12.8\text{-}4)$$

C_s shall not be less than

$$C_s = 0.044S_{DS}I_e \geq 0.01 \quad (12.8\text{-}5)$$

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

$$C_s = 0.5S_1/(R/I_e) \quad (12.8\text{-}6)$$

where I_e and R are as defined in Section 12.8.1.1 and

S_{D1} = 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 12.8.2

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

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

12.8.1.2 Soil Structure Interaction Reduction

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

***12.8.1.3 Maximum* S**

For regular structures five stories or less above the base as defined in Section 11.2 and with a period, T, of 0.5 s or less, C_s is permitted to be calculated using a value of 1.5 for S_S.

12.8.2 Period Determination

The fundamental period of the structure, T, in the direction under consideration shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The fundamental period, T, shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 12.8-1 and the approximate fundamental period, T_a, determined in accordance with Section 12.8.2.1. As an alternative to performing an analysis to determine the fundamental period, T, it is permitted to use the approximate building period, T_a, calculated in accordance with Section 12.8.2.1, directly.

12.8.2.1 Approximate Fundamental Period

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

$$T_a = C_t h_n^x \quad (12.8\text{-}7)$$

where h_n is the structural height as defined in Section 11.2 and the coefficients C_t and x are determined from Table 12.8-2.

Alternatively, it is permitted to determine the approximate fundamental period (T_a), in s, from the following equation for structures not exceeding 12 stories above the base as defined in Section 11.2 where the seismic force-resisting system consists

Table 12.8-1 Coefficient for Upper Limit on Calculated Period

Design Spectral Response Acceleration Parameter at 1 s, S_{D1}	Coefficient C_u
≥ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤ 0.1	1.7

Table 12.8-2 Values of Approximate Period Parameters C_t and x

Structure Type	C_t	x
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:		
Steel moment-resisting frames	0.028 (0.0724)[a]	0.8
Concrete moment-resisting frames	0.016 (0.0466)[a]	0.9
Steel eccentrically braced frames in accordance with Table 12.2-1 lines B1 or D1	0.03 (0.0731)[a]	0.75
Steel buckling-restrained braced frames	0.03 (0.0731)[a]	0.75
All other structural systems	0.02 (0.0488)[a]	0.75

[a]Metric equivalents are shown in parentheses.

entirely of concrete or steel moment resisting frames and the average story height is at least 10 ft (3 m):

$$T_a = 0.1N \qquad (12.8\text{-}8)$$

where N = number of stories above the base.

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

$$T_a = \frac{0.0019}{\sqrt{C_w}} h_n \qquad (12.8\text{-}9)$$

where C_w is calculated from Eq. 12.8-10 as follows:

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

where

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

12.8.3 Vertical Distribution of Seismic Forces

The lateral seismic force (F_x) (kip or kN) induced at any level shall be determined from the following equations:

$$F_x = C_{vx} V \qquad (12.8\text{-}11)$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad (12.8\text{-}12)$$

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

12.8.4 Horizontal Distribution of Forces

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

$$V_x = \sum_{i=x}^{n} F_i \qquad (12.8\text{-}13)$$

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

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

12.8.4.1 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.

12.8.4.2 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_{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.

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

12.8.4.3 Amplification of Accidental Torsional Moment

Structures assigned to Seismic Design Category C, D, E, or F, where Type 1a or 1b torsional irregularity exists as defined in Table 12.3-1 shall have the effects accounted for by multiplying M_{ta} at each level by a torsional amplification factor (A_x) as illustrated in

Fig. 12.8-1 and determined from the following equation:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}} \right)^2 \qquad (12.8\text{-}14)$$

where

δ_{max} = the maximum displacement at Level x computed assuming $A_x = 1$ (in. or mm)

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x computed assuming $A_x = 1$ (in. or mm)

The torsional amplification factor (A_x) shall not be less than 1 and is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

12.8.5 Overturning

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

12.8.6 Story Drift Determination

The design story drift (Δ) shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration. See Fig. 12.8-2. Where 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, Δ shall be computed using the strength level seismic forces specified in Section 12.8 without reduction for allowable stress design.

For structures assigned to Seismic Design Category C, D, E, or F having horizontal irregularity Type 1a or 1b of Table 12.3-1, the design story drift, Δ, shall be computed as the largest difference of the deflections of vertically aligned points at the top and bottom of the story under consideration along any of the edges of the structure.

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

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \qquad (12.8\text{-}15)$$

where

C_d = the deflection amplification factor in Table 12.2-1

δ_{xe} = 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

12.8.6.1 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 12.8.

EXCEPTION: Eq. 12.8-5 need not be considered for computing drift.

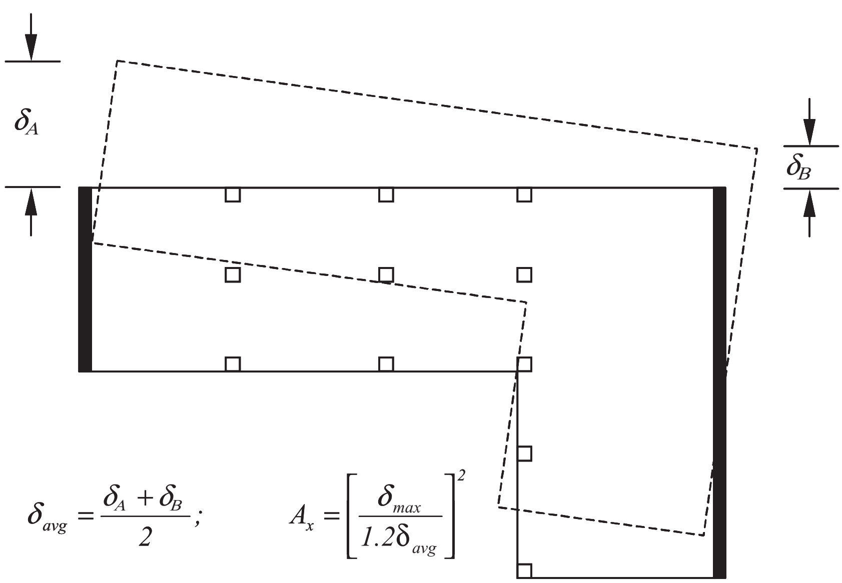


FIGURE 12.8-1 Torsional Amplification Factor, A_x

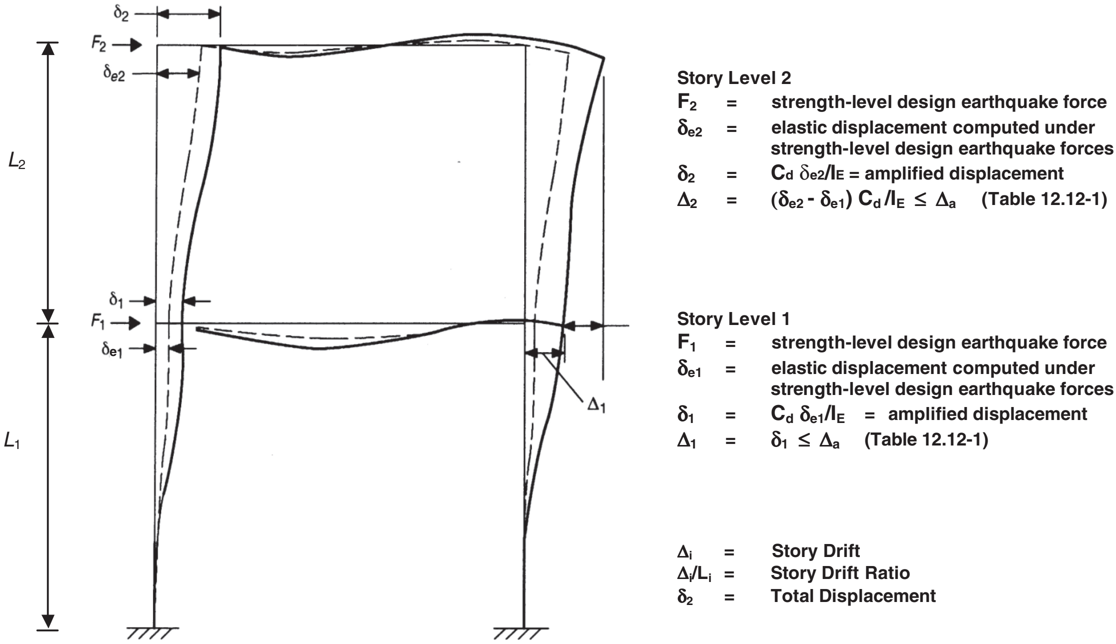


FIGURE 12.8-2 Story Drift Determination

12.8.6.2 Period for Computing Drift

For determining compliance with the story drift limits of Section 12.12.1, it is permitted to determine the elastic drifts, (δ_{xe}), using seismic design forces based on the computed fundamental period of the structure without the upper limit (C_uT_a) specified in Section 12.8.2.

12.8.7 P-Delta Effects

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

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} \qquad (12.8\text{-}16)$$

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

Δ = the design story drift as defined in Section 12.8.6 occurring simultaneously with V_x (in. or mm)

I_e = the importance factor determined in accordance with Section 11.5.1

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

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

C_d = the deflection amplification factor in Table 12.2-1

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

$$\theta_{max} = \frac{0.5}{\beta C_d} \le 0.25 \qquad (12.8\text{-}17)$$

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

Where the stability coefficient (θ) is greater than 0.10 but less than or equal to θ_{max}, the incremental factor related to P-delta effects 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 θ is greater than θ_{max}, the structure is potentially unstable and shall be redesigned.

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

12.9 MODAL RESPONSE SPECTRUM ANALYSIS

12.9.1 Number of Modes

An analysis shall be conducted to determine the natural modes of vibration for the structure. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual mass in each of the orthogonal horizontal directions of response considered by the model.

12.9.2 Modal Response Parameters

The value for each force-related design parameter of interest, including story drifts, support forces, and individual member forces for each mode of response shall be computed using the properties of each mode and the response spectra defined in either Section 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.

12.9.3 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.

12.9.4 Scaling Design Values of Combined Response

A base shear (V) shall be calculated in each of the two orthogonal horizontal directions using the calculated fundamental period of the structure T in each direction and the procedures of Section 12.8.

12.9.4.1 Scaling of Forces

Where the calculated fundamental period exceeds C_uT_a in a given direction, C_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 $0.85\frac{V}{V_t}$:

where

V = the equivalent lateral force procedure base shear, calculated in accordance with this section and Section 12.8

V_t = the base shear from the required modal combination

12.9.4.2 Scaling of Drifts

Where the combined response for the modal base shear (V_t) is less than $0.85C_sW$, and where C_s is determined in accordance with Eq. 12.8-6, drifts shall be multiplied by $0.85\dfrac{C_sW}{V_t}$

12.9.5 Horizontal Shear Distribution

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

12.9.6 P-Delta Effects

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

12.9.7 Soil Structure Interaction Reduction

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

12.10 DIAPHRAGMS, CHORDS, AND COLLECTORS

12.10.1 Diaphragm Design

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

12.10.1.1 Diaphragm Design Forces

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

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad (12.10\text{-}1)$$

where

F_{px} = the diaphragm design force
F_i = the design force applied to Level i
w_i = the weight tributary to Level i
w_{px} = the weight tributary to the diaphragm at Level x

The force determined from Eq. 12.10-1 shall not be less than

$$F_{px} = 0.2S_{DS}I_e w_{px} \quad (12.10\text{-}2)$$

The force determined from Eq. 12.10-1 need not exceed

$$F_{px} = 0.4S_{DS}I_e w_{px} \quad (12.10\text{-}3)$$

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. 12.10-1. The redundancy factor, ρ, applies to the design of diaphragms in structures assigned to Seismic Design Category D, E, or F. For inertial forces calculated in accordance with Eq. 12.10-1, the redundancy factor shall equal 1.0. For transfer forces, the redundancy factor, ρ, shall be the same as that used for the structure. For structures having horizontal or vertical structural irregularities of the types indicated in Section 12.3.3.4, the requirements of that section shall also apply.

12.10.2 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.

12.10.2.1 Collector Elements Requiring Load Combinations with Overstrength Factor for Seismic Design Categories C through F

In structures assigned to Seismic Design Category C, D, E, or F, collector elements (see Fig. 12.10-1) and their connections including connections to vertical elements shall be designed to resist the maximum of the following:

1. Forces calculated using the seismic load effects including overstrength factor of Section 12.4.3 with seismic forces determined by the Equivalent Lateral Force procedure of Section 12.8 or the Modal Response Spectrum Analysis procedure of Section 12.9.
2. Forces calculated using the seismic load effects including overstrength factor of Section 12.4.3 with seismic forces determined by Equation 12.10-1.
3. Forces calculated using the load combinations of Section 12.4.2.3 with seismic forces determined by Equation 12.10-2.

Transfer forces as described in Section 12.10.1.1 shall be considered.

EXCEPTIONS:
1. The forces calculated above need not exceed those calculated using the load combinations of Section 12.4.2.3 with seismic forces determined by Equation 12.10-3.
2. In structures or portions thereof braced entirely by light-frame shear walls, collector elements and their connections including connections to vertical elements need only be designed to resist forces using the load combinations of Section 12.4.2.3 with seismic forces determined in accordance with Section 12.10.1.1.

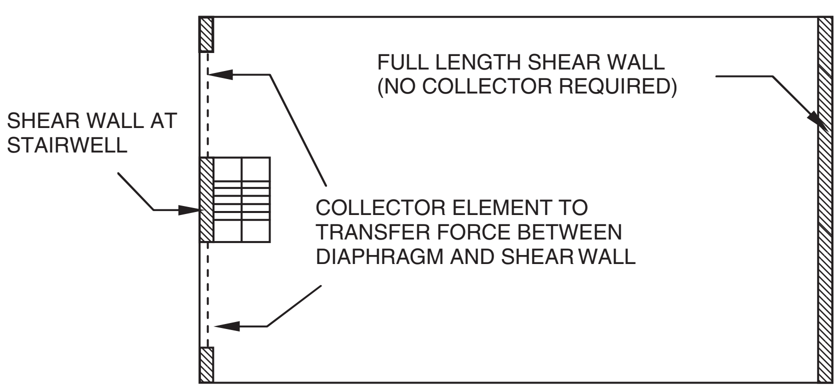


FIGURE 12.10-1 Collectors

12.11 STRUCTURAL WALLS AND THEIR ANCHORAGE

12.11.1 Design for Out-of-Plane Forces

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

12.11.2 Anchorage of Structural Walls and Transfer of Design Forces into Diaphragms.

12.11.2.1 Wall Anchorage Forces

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

$$F_p = 0.4S_{DS}k_aI_eW_p \qquad (12.11\text{-}1)$$

F_p shall not be taken less than $0.2k_aI_eW_p$.

$$k_a = 1.0 + \frac{L_f}{100} \qquad (12.11\text{-}2)$$

k_a need not be taken larger than 2.0.

where

F_p = the design force in the individual anchors
S_{DS} = the design spectral response acceleration parameter at short periods per Section 11.4.4
I_e = the importance factor determined in accordance with Section 11.5.1
k_a = amplification factor for diaphragm flexibility
L_f = the span, in feet, of a flexible diaphragm that provides the lateral support for the wall; the span is measured between vertical elements that provide lateral support to the diaphragm in the direction considered; use zero for rigid diaphragms
W_p = the weight of the wall tributary to the anchor

Where the anchorage is not located at the roof and all diaphragms are not flexible, the value from Eq. 12.11-1 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).

12.11.2.2 Additional Requirements for Diaphragms in Structures Assigned to Seismic Design Categories C through F

12.11.2.2.1 Transfer of Anchorage Forces into Diaphragm Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragms. Diaphragm connections shall be positive, mechanical, or welded. Added chords are permitted to be used to form subdiaphragms to transmit the anchorage forces to the main continuous cross-ties. The maximum length-to-width ratio of the structural subdiaphragm shall be 2.5 to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components. Connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm.

12.11.2.2.2 Steel Elements of Structural Wall Anchorage System The strength design forces for steel elements of the structural wall anchorage system, with the exception of anchor bolts and reinforcing steel, shall be increased by 1.4 times the forces otherwise required by this section.

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

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

12.11.2.2.5 Embedded Straps Diaphragm to structural wall anchorage using embedded straps shall be attached to, or hooked around, the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

12.11.2.2.6 Eccentrically Loaded Anchorage System Where elements of the wall anchorage system are loaded eccentrically or are not perpendicular to the wall, the system shall be designed to resist all components of the forces induced by the eccentricity.

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

12.12 DRIFT AND DEFORMATION

12.12.1 Story Drift Limit

The design story drift (Δ) as determined in Sections 12.8.6, 12.9.2, or 16.1, shall not exceed the allowable story drift (Δ_a) as obtained from Table 12.12-1 for any story.

12.12.1.1 Moment Frames in Structures Assigned to Seismic Design Categories D through F

For seismic force-resisting systems comprised solely of moment frames in structures assigned to Seismic Design Categories D, E, or F, the design story drift (Δ) shall not exceed Δ_a/ρ for any story. ρ shall be determined in accordance with Section 12.3.4.2.

12.12.2 Diaphragm Deflection

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

12.12.3 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 (δ_M). δ_M shall be determined at critical locations with consideration for translational and torsional displacements of the structure including torsional amplifications, where applicable, using the following equation:

$$\delta_M = \frac{C_d \delta_{max}}{I_e} \qquad (12.12\text{-}1)$$

Where δ_{max} = maximum elastic displacement at the critical location.

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

$$\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2} \qquad (12.12\text{-}2)$$

where δ_{M1} and δ_{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 δ_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.

Table 12.12-1 Allowable Story Drift, Δ_a[a,b]

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

[a] h_{sx} is the story height below Level x.
[b] For seismic force-resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

12.12.4 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. 12.8-15 multiplied by $1.5R/C_d$, and
2. Considering additional deflection due to diaphragm rotation including the torsional amplification factor calculated per Section 12.8.4.3 where either structure is torsionally irregular, and
3. Considering diaphragm deformations, and
4. Assuming the two structures are moving in opposite directions and using the absolute sum of the displacements.

12.12.5 Deformation Compatibility for Seismic Design Categories D through F

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

EXCEPTION: Reinforced concrete frame members not designed as part of the seismic force-resisting system shall comply with Section 21.11 of ACI 318.

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

12.13 FOUNDATION DESIGN

12.13.1 Design Basis

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

12.13.2 Materials of Construction

Materials used for the design and construction of foundations shall comply with the requirements of Chapter 14. Design and detailing of steel piles shall comply with Section 14.1.7 Design and detailing of concrete piles shall comply with Section 14.2.3.

12.13.3 Foundation Load-Deformation Characteristics

Where foundation flexibility is included for the linear analysis procedures in Chapters 12 and 16, 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.

12.13.4 Reduction of Foundation Overturning

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

a. The structure is designed in accordance with the Equivalent Lateral Force Analysis as set forth in Section 12.8.
b. The structure is not an inverted pendulum or cantilevered column type structure.

Overturning effects at the soil–foundation interface are permitted to be reduced by 10 percent for foundations of structures designed in accordance with the modal analysis requirements of Section 12.9.

12.13.5 Requirements for Structures Assigned to Seismic Design Category C

In addition to the requirements of Section 11.8.2, the following foundation design requirements shall apply to structures assigned to Seismic Design Category C.

12.13.5.1 Pole-Type Structures

Where construction employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth is used to resist lateral loads, the depth of embedment required for posts or poles to resist seismic forces shall be determined by means of the design criteria established in the foundation investigation report.

12.13.5.2 Foundation Ties

Individual pile caps, drilled piers, or caissons shall be interconnected by ties. All ties shall have a design strength in tension or compression at least equal to a force equal to 10 percent of S_{DS} times the larger pile cap or column factored dead plus factored live load unless it is demonstrated that equivalent restraint will be provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils, very dense granular soils, or other approved means.

12.13.5.3 Pile Anchorage Requirements

In addition to the requirements of Section 14.2.3.1, anchorage of piles shall comply with this section. Where required for resistance to uplift forces, anchorage of steel pipe (round HSS sections), concrete-filled steel pipe or H piles to the pile cap shall be made by means other than concrete bond to the bare steel section.

EXCEPTION: Anchorage of concrete-filled steel pipe piles is permitted to be accomplished using deformed bars developed into the concrete portion of the pile.

12.13.6 Requirements for Structures Assigned to Seismic Design Categories D through F

In addition to the requirements of Sections 11.8.2, 11.8.3, 14.1.8, and 14.2.3.2, the following foundation design requirements shall apply to structures assigned to Seismic Design Category D, E, or F. Design and construction of concrete foundation elements shall conform to the requirements of ACI 318, Section 21.8, except as modified by the requirements of this section.

EXCEPTION: Detached one- and two-family dwellings of light-frame construction not exceeding two stories above grade plane need only comply with the requirements for Sections 11.8.2, 11.8.3 (Items 2 through 4), 12.13.2, and 12.13.5.

12.13.6.1 Pole-Type Structures

Where construction employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth is used to resist lateral loads, the depth of embedment required for posts or poles to resist seismic forces shall be determined by means of the design criteria established in the foundation investigation report.

12.13.6.2 Foundation Ties

Individual pile caps, drilled piers, or caissons shall be interconnected by ties. In addition, individual spread footings founded on soil defined in Chapter 20 as Site Class E or F shall be interconnected by ties. All ties shall have a design strength in tension or compression at least equal to a force equal to 10 percent of S_{DS} times the larger pile cap or column factored dead plus factored live load unless it is demonstrated that equivalent restraint will be provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils, very dense granular soils, or other approved means.

12.13.6.3 General Pile Design Requirement

Piling shall be designed and constructed to withstand deformations from earthquake ground motions and structure response. Deformations shall include both free-field soil strains (without the structure) and deformations induced by lateral pile resistance to structure seismic forces, all as modified by soil–pile interaction.

12.13.6.4 Batter Piles

Batter piles and their connections shall be capable of resisting forces and moments from the load combinations with overstrength factor of Section 12.4.3.2 or 12.14.3.2.2. Where vertical and batter piles act jointly to resist foundation forces as a group, these forces shall be distributed to the individual piles in accordance with their relative horizontal and vertical rigidities and the geometric distribution of the piles within the group.

12.13.6.5 Pile Anchorage Requirements

In addition to the requirements of Section 12.13.5.3, anchorage of piles shall comply with this section. Design of anchorage of piles into the pile cap shall consider the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. For piles required to resist uplift forces or provide rotational restraint, anchorage into the pile cap shall comply with the following:

1. In the case of uplift, the anchorage shall be capable of developing the least of the nominal tensile strength of the longitudinal reinforcement in a concrete pile, the nominal tensile strength of a steel pile, and 1.3 times the pile pullout resistance, or shall be designed to resist the axial tension force resulting from the seismic load effects including overstrength factor of Section 12.4.3 or 12.14.3.2.

The pile pullout resistance shall be taken as the ultimate frictional or adhesive force that can be developed between the soil and the pile plus the pile and pile cap weight.

2. In the case of rotational restraint, the anchorage shall be designed to resist the axial and shear forces and moments resulting from the seismic load effects including overstrength factor of Section 12.4.3 or 12.14.3.2 or shall be capable of developing the full axial, bending, and shear nominal strength of the pile.

12.13.6.6 Splices of Pile Segments

Splices of pile segments shall develop the nominal strength of the pile section.

EXCEPTION: Splices designed to resist the axial and shear forces and moments from the seismic load effects including overstrength factor of Section 12.4.3 or 12.14.3.2.

12.13.6.7 Pile Soil Interaction

Pile moments, shears, and lateral deflections used for design shall be established considering the interaction of the shaft and soil. Where the ratio of the depth of embedment of the pile to the pile diameter or width is less than or equal to 6, the pile is permitted to be assumed to be flexurally rigid with respect to the soil.

12.13.6.8 Pile Group Effects

Pile group effects from soil on lateral pile nominal strength shall be included where pile center-to-center spacing in the direction of lateral force is less than eight pile diameters or widths. Pile group effects on vertical nominal strength shall be included where pile center-to-center spacing is less than three pile diameters or widths.

12.14 SIMPLIFIED ALTERNATIVE STRUCTURAL DESIGN CRITERIA FOR SIMPLE BEARING WALL OR BUILDING FRAME SYSTEMS

12.14.1 General

12.14.1.1 Simplified Design Procedure

The procedures of this section are permitted to be used in lieu of other analytical procedures in Chapter 12 for the analysis and design of simple buildings with bearing wall or building frame systems, subject to all of the limitations listed in this section. Where these procedures are used, the seismic design category shall be determined from Table 11.6-1 using the value of S_{DS} from Section 12.14.8.1. The simplified design procedure is permitted to be used if the following limitations are met:

1. The structure shall qualify for Risk Category I or II in accordance with Table 1.5-1.
2. The site class, defined in Chapter 20, shall not be class E or F.
3. The structure shall not exceed three stories above grade plane.
4. The seismic force-resisting system shall be either a bearing wall system or building frame system, as indicated in Table 12.14-1.
5. The structure shall have at least two lines of lateral resistance in each of two major axis directions.
6. At least one line of resistance shall be provided on each side of the center of mass in each direction.
7. For structures with flexible diaphragms, overhangs beyond the outside line of shear walls or braced frames shall satisfy the following:

$$a \le d/5 \qquad (12.14\text{-}1)$$

where

a = the distance perpendicular to the forces being considered from the extreme edge of the diaphragm to the line of vertical resistance closest to that edge

d = the depth of the diaphragm parallel to the forces being considered at the line of vertical resistance closest to the edge

8. For buildings with a diaphragm that is not flexible, the distance between the center of rigidity and the center of mass parallel to each major axis shall not exceed 15 percent of the greatest width of the diaphragm parallel to that axis. In addition, the following two equations shall be satisfied:

$$\sum_{i=1}^{m} k_{1i}d_{1i}^2 + \sum_{j=1}^{n} k_{2j}d_{2j}^2 \ge 2.5\left(0.05 + \frac{e_1}{b_1}\right)b_1^2 \sum_{i=1}^{m} k_{1i} \qquad \text{(Eq. 12.14-2A)}$$

$$\sum_{i=1}^{m} k_{1i}d_{1i}^2 + \sum_{j=1}^{n} k_{2j}d_{2j}^2 \ge 2.5\left(0.05 + \frac{e_2}{b_2}\right)b_2^2 \sum_{j=1}^{m} k_{1j} \qquad \text{(Eq. 12.14-2B)}$$

where (see Fig. 12.14-1)

k_{1i} = the lateral load stiffness of wall i or braced frame i parallel to major axis 1

k_{2j} = the lateral load stiffness of wall j or braced frame j parallel to major axis 2

Table 12.14-1 Design Coefficients and Factors for Seismic Force-Resisting Systems for Simplified Design Procedure

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R[a]	Limitations[b] Seismic Design Category B	C	D, E
A. BEARING WALL SYSTEMS					
1. Special reinforced concrete shear walls	14.2	5	P	P	P
2. Ordinary reinforced concrete shear walls	14.2	4	P	P	NP
3. Detailed plain concrete shear walls	14.2	2	P	NP	NP
4. Ordinary plain concrete shear walls	14.2	1½	P	NP	NP
5. Intermediate precast shear walls	14.2	4	P	P	40[c]
6. Ordinary precast shear walls	14.2	3	P	NP	NP
7. Special reinforced masonry shear walls	14.4	5	P	P	P
8. Intermediate reinforced masonry shear walls	14.4	3½	P	P	NP
9. Ordinary reinforced masonry shear walls	14.4	2	P	NP	NP
10. Detailed plain masonry shear walls	14.4	2	P	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	P	NP	NP
12. Prestressed masonry shear walls	14.4	1½	P	NP	NP
13. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	6½	P	P	P
14. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6½	P	P	P
15. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	P	P	NP[d]
16. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1 and 14.5	4	P	P	P
B. BUILDING FRAME SYSTEMS					
1. Steel eccentrically braced frames	14.1	8	P	P	P
2. Steel special concentrically braced frames	14.1	6	P	P	P
3. Steel ordinary concentrically braced frames	14.1	3¼	P	P	P
4. Special reinforced concrete shear walls	14.2	6	P	P	P
5. Ordinary reinforced concrete shear walls	14.2	5	P	P	NP
6. Detailed plain concrete shear walls	14.2 and 14.2.2.8	2	P	NP	NP
7. Ordinary plain concrete shear walls	14.2	1½	P	NP	NP
8. Intermediate precast shear walls	14.2	5	P	P	40[c]
9. Ordinary precast shear walls	14.2	4	P	NP	NP
10. Steel and concrete composite eccentrically braced frames	14.3	8	P	P	P
11. Steel and concrete composite special concentrically braced frames	14.3	5	P	P	P
12. Steel and concrete composite ordinary braced frames	14.3	3	P	P	NP
13. Steel and concrete composite plate shear walls	14.3	6½	P	P	P
14. Steel and concrete composite special shear walls	14.3	6	P	P	P
15. Steel and concrete composite ordinary shear walls	14.3	5	P	P	NP
16. Special reinforced masonry shear walls	14.4	5½	P	P	P

Continued

Table 12.14-1 (*Continued*)

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Limitations[b] Seismic Design Category B	C	D, E
17. Intermediate reinforced masonry shear walls	14.4	4	P	P	NP
18. Ordinary reinforced masonry shear walls	14.4	2	P	NP	NP
19. Detailed plain masonry shear walls	14.4	2	P	NP	NP
20. Ordinary plain masonry shear walls	14.4	1½	P	NP	NP
21. Prestressed masonry shear walls	14.4	1½	P	NP	NP
22. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.5	7	P	P	P
23. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	7	P	P	P
24. Light-frame walls with shear panels of all other materials	14.1and 14.5	2½	P	P	NP[d]
25. Steel buckling-restrained braced frames	14.1	8	P	P	P
26. Steel special plate shear walls	14.1	7	P	P	P

[a]Response modification coefficient, R, for use throughout the standard.
[b]P = permitted; NP = not permitted.
[c]Light-frame walls with shear panels of all other materials are not permitted in Seismic Design Category E.
[d]Light-frame walls with shear panels of all other materials are permitted up to 35 ft (10.6 m) in structural height, h_n, in Seismic Design Category D and are not permitted in Seismic Design Category E.

d_{1i} = the distance from the wall i or braced frame i to the center of rigidity, perpendicular to major axis 1
d_{2j} = the distance from the wall j or braced frame j to the center of rigidity, perpendicular to major axis 2
e_1 = the distance perpendicular to major axis 1 between the center of rigidity and the center of mass
b_1 = the width of the diaphragm perpendicular to major axis 1
e_2 = the distance perpendicular to major axis 2 between the center of rigidity and the center of mass
b_2 = the width of the diaphragm perpendicular to major axis 2
m = the number of walls and braced frames resisting lateral force in direction 1
n = the number of walls and braced frames resisting lateral force in direction 2

Eq. 12.14-2 A and B need not be checked where a structure fulfills all the following limitations:

1. The arrangement of walls or braced frames is symmetric about each major axis direction.
2. The distance between the two most separated lines of walls or braced frames is at least 90 percent of the dimension of the structure perpendicular to that axis direction.
3. The stiffness along each of the lines considered for item 2 above is at least 33 percent of the total stiffness in that axis direction.

9. Lines of resistance of the seismic force-resisting system shall be oriented at angles of no more than 15° from alignment with the major orthogonal horizontal axes of the building.
10. The simplified design procedure shall be used for each major orthogonal horizontal axis direction of the building.
11. System irregularities caused by in-plane or out-of-plane offsets of lateral force-resisting elements shall not be permitted.

    EXCEPTION: Out-of-plane and in-plane offsets of shear walls are permitted in two-story buildings of light-frame construction provided that the framing supporting the upper wall is designed for seismic force effects from overturning of the wall amplified by a factor of 2.5.

12. The lateral load resistance of any story shall not be less than 80 percent of the story above.

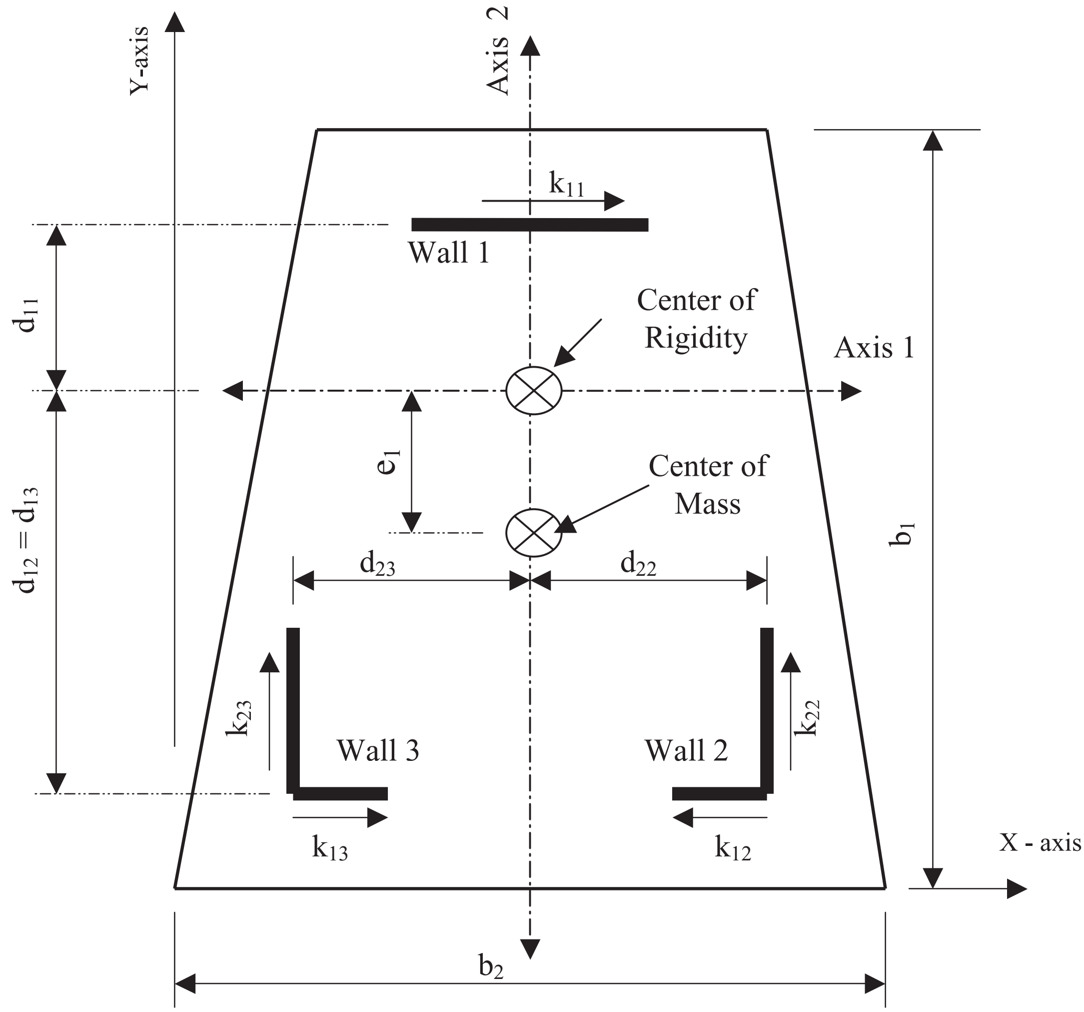


FIGURE 12.14-1 Notation Used in Torsion Check for Nonflexible Diaphragms

12.14.1.2 Reference Documents

The reference documents listed in Chapter 23 shall be used as indicated in Section 12.14.

12.14.1.3 Definitions

The definitions listed in Section 11.2 shall be used in addition to the following:

PRINCIPAL ORTHOGONAL HORIZONTAL DIRECTIONS: The orthogonal directions that overlay the majority of lateral force-resisting elements.

12.14.1.4 Notation

D = The effect of dead load

E = The effect of horizontal and vertical earthquake-induced forces

F_a = Acceleration-based site coefficient, see Section 12.14.8.1

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

F_p = The seismic design force applicable to a particular structural component

F_x = See Section 12.14.8.2

h_i = The height above the base to Level i

h_x = The height above the base to Level x

Level i = The building level referred to by the subscript i; i = 1 designates the first level above the base

Level n = The level that is uppermost in the main portion of the building

Level x = See "Level i"

Q_E = The effect of horizontal seismic forces

R = The response modification coefficient as given in Table 12.14-1

S_{DS} = See Section 12.14.8.1

S_S = See Section 11.4.1

V = The total design shear at the base of the structure in the direction of interest, as determined using the procedure of 12.14.8.1
V_x = The seismic design shear in Story x. See Section 12.14.8.3
W = See Section 12.14.8.1
W_c = Weight of wall
W_p = Weight of structural component
w_i = The portion of the effective seismic weight, W, located at or assigned to Level i
w_x = See Section 12.14.8.2

12.14.2 Design Basis

The structure shall include complete lateral and vertical force-resisting systems with adequate strength to resist the design seismic forces, specified in this section, in combination with other loads. Design seismic forces shall be distributed to the various elements of the structure and their connections using a linear elastic analysis in accordance with the procedures of Section 12.14.8. The members of the seismic force-resisting system and their connections shall be detailed to conform with the applicable requirements for the selected structural system as indicated in Section 12.14.4.1. A continuous load path, or paths, with adequate strength and stiffness shall be provided to transfer all forces from the point of application to the final point of resistance. The foundation shall be designed to accommodate the forces developed.

12.14.3 Seismic Load Effects and Combinations

All members of the structure, including those not part of the seismic force-resisting system, shall be designed using the seismic load effects of Section 12.14.3 unless otherwise exempted by this standard. 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 12.14.3.1. Where specifically required, seismic load effects shall be modified to account for overstrength, as set forth in Section 12.14.3.2.

12.14.3.1 Seismic Load Effect

The seismic load effect, E, shall be determined in accordance with the following:

1. For use in load combination 5 in Section 2.3.2 or load combinations 5 and 6 in Section 2.4.1, E shall be determined in accordance with Eq. 12.14-3 as follows:

$$E = E_h + E_v \tag{12.14-3}$$

2. For use in load combination 7 in Section 2.3.2 or load combination 8 in Section 2.4.1, E shall be determined in accordance with Eq. 12.14-4 as follows:

$$E = E_h - E_v \tag{12.14-4}$$

where

E = seismic load effect
E_h = effect of horizontal seismic forces as defined in Section 12.14.3.1.1
E_v = effect of vertical seismic forces as defined in Section 12.14.3.1.2

12.14.3.1.1 Horizontal Seismic Load Effect The horizontal seismic load effect, E_h, shall be determined in accordance with Eq. 12.14-5 as follows:

$$E_h = Q_E \tag{12.14-5}$$

where

Q_E = effects of horizontal seismic forces from V or F_p as specified in Sections 12.14.7.5, 12.14.8.1, and 13.3.1.

12.14.3.1.2 Vertical Seismic Load Effect The vertical seismic load effect, E_v, shall be determined in accordance with Eq. 12.14-6 as follows:

$$E_v = 0.2S_{DS}D \tag{12.14-6}$$

where

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

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

1. In Eqs. 12.4-3, 12.4-4, 12.4-7, and 12.14-8 where S_{DS} is equal to or less than 0.125.
2. In Eq. 12.14-4 where determining demands on the soil–structure interface of foundations.

12.14.3.1.3 Seismic Load Combinations Where the prescribed seismic load effect, E, defined in Section 12.14.3.1 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 Sections 2.3.2 or 2.4.1:

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

5. $(1.2 + 0.2S_{DS})D + Q_E + L + 0.2S$
7. $(0.9 - 0.2S_{DS})D + Q_E + 1.6H$

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/m^2), 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. $(1.0 + 0.14S_{DS})D + H + F + 0.7Q_E$
6. $(1.0 + 0.105S_{DS})D + H + F + 0.525Q_E + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
8. $(0.6 - 0.14S_{DS})D + 0.7Q_E + H$

12.14.3.2 Seismic Load Effect Including a 2.5 Overstrength Factor

Where specifically required, conditions requiring overstrength factor applications shall be determined in accordance with the following:

1. For use in load combination 5 in Section 2.3.2 or load combinations 5 and 6 in Section 2.4.1, E shall be taken equal to E_m as determined in accordance with Eq. 12.14-7 as follows:

$$E_m = E_{mh} + E_v \tag{12.14-7}$$

2. For use in load combination 7 in Section 2.3.2 or load combination 8 in Section 2.4.1, E shall be taken equal to E_m as determined in accordance with Eq. 12.14-8 as follows:

$$E_m = E_{mh} - E_v \tag{12.14-8}$$

where

E_m = seismic load effect including overstrength factor
E_{mh} = effect of horizontal seismic forces including overstrength factor as defined in Section 12.14.3.2.1
E_v = vertical seismic load effect as defined in Section 12.14.3.1.2

12.14.3.2.1 Horizontal Seismic Load Effect with a 2.5 Overstrength Factor The horizontal seismic load effect with overstrength factor, E_{mh}, shall be determined in accordance with Eq. 12.14-9 as follows:

$$E_{mh} = 2.5Q_E \tag{12.14-9}$$

where

Q_E = effects of horizontal seismic forces from V or F_p as specified in Sections 12.14.7.5, 12.14.8.1, and 13.3.1

EXCEPTION: The value of E_{mh} need not exceed the maximum force that can develop in the element as determined by a rational, plastic mechanism analysis or nonlinear response analysis utilizing realistic expected values of material strengths.

12.14.3.2.2 Load Combinations with Overstrength Factor Where the seismic load effect with overstrength factor, E_m, defined in Section 12.14.3.2, 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 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. $(1.2 + 0.2S_{DS})D + 2.5Q_E + L + 0.2S$
7. $(0.9 - 0.2S_{DS})D + 2.5Q_E + 1.6H$

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/m^2), with the exception of garages or areas occupied as places of public assembly.
2. The load factor on H shall be set equal to zero in combination 7 if the structural action due to H counteracts that due to E. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H, but shall be included in the design resistance.

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

5. $(1.0 + 0.14S_{DS})D + H + F + 1.75Q_E$
6. $(1.0 + 0.105S_{DS})D + H + F + 1.313Q_E + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
8. $(0.6 - 0.14S_{DS})D + 1.75Q_E + H$

12.14.3.2.3 Allowable Stress Increase for Load Combinations with Overstrength Where allowable stress design methodologies are used with the seismic load effect defined in Section 12.14.3.2 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 that combination with the duration of load increases permitted in AF&PA NDS is permitted.

12.14.4 Seismic Force-Resisting System

12.14.4.1 Selection and Limitations

The basic lateral and vertical seismic force-resisting system shall conform to one of the types indicated in Table 12.14-1 and shall conform to all of the detailing requirements referenced in the table. The appropriate response modification coefficient, R, indicated in Table 12.14-1 shall be used in determining the base shear and element design forces as set forth in the seismic requirements of this standard.

Special framing and detailing requirements are indicated in Section 12.14.7 and in Sections 14.1, 14.2, 14.3, 14.4, and 14.5 for structures assigned to the various seismic design categories.

12.14.4.2 Combinations of Framing Systems

12.14.4.2.1 Horizontal Combinations Different seismic force-resisting systems are permitted to be used in each of the two principal orthogonal building directions. Where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of R used for design in that direction shall not be greater than the least value of R for any of the systems utilized in that direction.

EXCEPTION: For buildings of light-frame construction or having flexible diaphragms and that are two stories or less above grade plane, resisting elements are permitted to be designed using the least value of R of the different seismic force-resisting systems found in each independent line of framing. The value of R used for design of diaphragms in such structures shall not be greater than the least value for any of the systems utilized in that same direction.

12.14.4.2.2 Vertical Combinations Different seismic force-resisting systems are permitted to be used in different stories. The value of R used in a given direction shall not be greater than the least value of any of the systems used in that direction.

12.14.4.2.3 Combination Framing Detailing Requirements The detailing requirements of Section 12.14.7 required by the higher response modification coefficient, R, shall be used for structural members common to systems having different response modification coefficients.

12.14.5 Diaphragm Flexibility

Diaphragms constructed of steel decking (untopped), wood structural panels, or similar panelized construction are permitted to be considered flexible.

12.14.6 Application of Loading

The effects of the combination of loads shall be considered as prescribed in Section 12.14.3. The design seismic forces are permitted to be applied separately in each orthogonal direction and the combination of effects from the two directions need not be considered. Reversal of load shall be considered.

12.14.7 Design and Detailing Requirements

The design and detailing of the members of the seismic force-resisting system shall comply with the requirements of this section. 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 12.13. Structural elements including foundation elements shall conform to the material design and detailing requirements set forth in Chapter 14.

12.14.7.1 Connections

All parts of the structure between separation joints shall be interconnected, and the connection shall be capable of transmitting the seismic force, F_p, induced by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure with elements having a strength of 0.20 times the short period design spectral response acceleration coefficient, S_{DS}, times the weight of the smaller portion or 5 percent of the portion's weight, whichever is greater.

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

12.14.7.2 Openings or Reentrant Building Corners

Except where as otherwise specifically provided for in this standard, openings in shear walls, diaphragms, or other plate-type elements, shall be provided with reinforcement at the edges of the openings or reentrant corners designed to transfer the stresses into the structure. The edge reinforcement shall extend into the body of the wall or diaphragm a distance sufficient to develop the force in the reinforcement.

EXCEPTION: Shear walls of wood structural panels are permitted where designed in accordance with AF&PA SDPWS for perforated shear walls or AISI S213 for Type II shear walls.

12.14.7.3 Collector Elements

Collector elements shall be provided with adequate strength to transfer the seismic forces originating in other portions of the structure to the element providing the resistance to those forces (see Fig. 12.10-1). Collector elements, splices, and their connections to resisting elements shall be designed to resist the forces defined in Section 12.14.3.2.

EXCEPTION: In structures, or portions thereof, braced entirely by light-frame shear walls, collector elements, splices, and connections to resisting elements are permitted to be designed to resist forces in accordance with Section 12.14.7.4.

12.14.7.4 Diaphragms

Floor and roof diaphragms shall be designed to resist the design seismic forces at each level, F_x, calculated in accordance with Section 12.14.8.2. Where the diaphragm is required to transfer design seismic forces from the vertical-resisting elements above the diaphragm to other vertical-resisting elements below the diaphragm due to changes in relative lateral stiffness in the vertical elements, the transferred portion of the seismic shear force at that level, V_x, shall be added to the diaphragm design force. Diaphragms shall provide for both the shear and bending stresses resulting from these forces. Diaphragms shall have ties or struts to distribute the wall anchorage forces into the diaphragm. Diaphragm connections shall be positive, mechanical, or welded type connections.

12.14.7.5 Anchorage of Structural Walls

Structural walls shall be anchored to all floors, roofs, and members that provide out-of-plane lateral support for the wall or that are supported by the wall. The anchorage shall provide a positive direct connection between the wall and floor, roof, or supporting member with the strength to resist the out-of-plane force given by Eq. 12.14-10:

$$F_p = 0.4 k_a S_{DS} W_p \qquad (12.14\text{-}10)$$

F_p shall not be taken less than $0.2k_a\ W_p$.

$$k_a = 1.0 + \frac{L_f}{100} \qquad (12.14\text{-}11)$$

k_a need not be taken larger than 2.0 where

F_p = the design force in the individual anchors
k_a = amplification factor for diaphragm flexibility
L_f = the span, in feet, of a flexible diaphragm that provides the lateral support for the wall; the span is measured between vertical elements that provide lateral support to the diaphragm in the direction considered; use zero for rigid diaphragms
S_{DS} = the design spectral response acceleration at short periods per Section 12.14.8.1
W_p = the weight of the wall tributary to the anchor

12.14.7.5.1 Transfer of Anchorage Forces into Diaphragms Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragms. Added chords are permitted to be used to form subdiaphragms to transmit the anchorage forces to the main continuous cross-ties. The maximum length-to-width ratio of the structural subdiaphragm shall be 2.5 to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components. Connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm.

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

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

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

12.14.7.6 Bearing Walls and Shear Walls

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

12.14.7.7 Anchorage of Nonstructural Systems

Where required by Chapter 13, all portions or components of the structure shall be anchored for the seismic force, F_p, prescribed therein.

12.14.8 Simplified Lateral Force Analysis Procedure

An equivalent lateral force analysis shall consist of the application of equivalent static lateral forces to a linear mathematical model of the structure. The lateral forces applied in each direction shall sum to a total seismic base shear given by Section 12.14.8.1 and shall be distributed vertically in accordance with Section 12.14.8.2. For purposes of analysis, the structure shall be considered fixed at the base.

12.14.8.1 Seismic Base Shear

The seismic base shear, V, in a given direction shall be determined in accordance with Eq. 12.14-11:

$$V = \frac{FS_{DS}}{R}W \qquad (12.14\text{-}11)$$

where

$$S_{DS} = \frac{2}{3}F_aS_s$$

where F_a is permitted to be taken as 1.0 for rock sites, 1.4 for soil sites, or determined in accordance with Section 11.4.3. For the purpose of this section, sites are permitted to be considered to be rock if there is no more than 10 ft (3 m) of soil between the rock surface and the bottom of spread footing or mat foundation. In calculating S_{DS}, S_s shall be in accordance with Section 11.4.1, but need not be taken larger than 1.5.

F = 1.0 for buildings that are one story above grade plane
F = 1.1 for buildings that are two stories above grade plane
F = 1.2 for buildings that are three stories above grade plane
R = the response modification factor from Table 12.14-1
W = effective seismic weight of the structure that includes the dead load, as defined in Section 3.1, above grade plane and other loads above grade plane as listed in the following text:

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/m^2) 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/m^2), 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.

12.14.8.2 Vertical Distribution

The forces at each level shall be calculated using the following equation:

$$F_x = \frac{w_x}{W}V \qquad (12.14\text{-}12)$$

where w_x = the portion of the effective seismic weight of the structure, W, at level x.

12.14.8.3 Horizontal Shear Distribution

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

$$V_x = \sum_{i=x}^{n} F_i \qquad (12.14\text{-}13)$$

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

12.14.8.3.1 Flexible Diaphragm Structures The seismic design story shear in stories of structures with flexible diaphragms, as defined in Section 12.14.5, shall be distributed to the vertical elements of the seismic force-resisting system using tributary area rules. Two-dimensional analysis is permitted where diaphragms are flexible.

12.14.8.3.2 Structures with Diaphragms That Are Not Flexible For structures with diaphragms that are not flexible, as defined in Section 12.14.5, the seismic design story shear, V_x (kip or kN), shall be distributed to the various vertical elements of the seismic force-resisting system in the story under consideration based on the relative lateral stiffnesses of the vertical elements and the diaphragm.

12.14.8.3.2.1 Torsion The design of structures with diaphragms that are not flexible shall include the torsional moment, M_t (kip-ft or KN-m) resulting from eccentricity between the locations of center of mass and the center of rigidity.

12.14.8.4 Overturning

The structure shall be designed to resist overturning effects caused by the seismic forces determined in Section 12.14.8.2. The foundations of structures shall be designed for not less than 75 percent of the foundation overturning design moment, M_f (kip-ft or kN-m) at the foundation–soil interface.

12.14.8.5 Drift Limits and Building Separation

Structural drift need not be calculated. Where a drift value is needed for use in material standards, to determine structural separations between buildings or from property lines, for design of cladding, or for other design requirements, it shall be taken as 1 percent of structural height, h_n, unless computed to be less. All portions of the structure shall be designed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact under the total deflection.

Chapter 13

SEISMIC DESIGN REQUIREMENTS FOR NONSTRUCTURAL COMPONENTS

13.1 GENERAL

13.1.1 Scope

This chapter establishes minimum design criteria for nonstructural components that are permanently attached to structures and for their supports and attachments. 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 shall be designed in accordance with Section 15.3.2.

13.1.2 Seismic Design Category

For the purposes of this chapter, nonstructural components shall be assigned to the same seismic design category as the structure that they occupy or to which they are attached.

13.1.3 Component Importance Factor

All components shall be assigned a component importance factor as indicated in this section. The component importance factor, I_p, shall be taken as 1.5 if any of the following conditions apply:

1. The component is required to function for life-safety purposes after an earthquake, including fire protection sprinkler systems and egress stairways.
2. The component conveys, supports, or otherwise contains toxic, highly toxic, or explosive substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.
3. The component is in or attached to a Risk Category IV structure and it is needed for continued operation of the facility or its failure could impair the continued operation of the facility.
4. The component conveys, supports, or otherwise contains hazardous substances and is attached to a structure or portion thereof classified by the authority having jurisdiction as a hazardous occupancy.

All other components shall be assigned a component importance factor, I_p, equal to 1.0.

13.1.4 Exemptions

The following nonstructural components are exempt from the requirements of this section:

1. Furniture (except storage cabinets as noted in Table 13.5-1).
2. Temporary or movable equipment.
3. Architectural components in Seismic Design Category B other than parapets supported by bearing walls or shear walls provided that the component importance factor, I_p, is equal to 1.0.
4. Mechanical and electrical components in Seismic Design Category B.
5. Mechanical and electrical components in Seismic Design Category C provided that the component importance factor, I_p, is equal to 1.0.
6. Mechanical and electrical components in Seismic Design Categories D, E, or F where all of the following apply:
   a. The component importance factor, I_p, is equal to 1.0;
   b. The component is positively attached to the structure;
   c. Flexible connections are provided between the component and associated ductwork, piping, and conduit; and either
      i. The component weighs 400 lb (1,780 N) or less and has a center of mass located 4 ft (1.22 m) or less above the adjacent floor level; or
      ii. The component weighs 20 lb (89 N) or less or, in the case of a distributed system, 5 lb/ft (73 N/m) or less.

13.1.5 Application of Nonstructural Component Requirements to Nonbuilding Structures

Nonbuilding structures (including storage racks and tanks) that are supported by other structures shall be designed in accordance with Chapter 15. Where Section 15.3 requires that seismic forces be determined in accordance with Chapter 13 and values for R_p are not provided in Table 13.5-1 or 13.6-1, R_p shall be taken as equal to the value of *R* listed in Section 15. The value of a_p shall be determined in accordance with footnote *a* of Table 13.5-1 or 13.6-1.

13.1.6 Reference Documents

Where a reference document provides a basis for the earthquake-resistant design of a particular type of nonstructural component, that document is permitted to be used, subject to the approval of the authority having jurisdiction and the following conditions:

a. The design earthquake forces shall not be less than those determined in accordance with Section 13.3.1.
b. Each nonstructural component's seismic interactions with all other connected components and with the supporting structure shall be accounted for in the design. The component shall accommodate drifts, deflections, and relative displacements determined in accordance with the applicable seismic requirements of this standard.
c. Nonstructural component anchorage requirements shall not be less than those specified in Section 13.4.

13.1.7 Reference Documents Using Allowable Stress Design

Where a reference document provides a basis for the earthquake-resistant design of a particular type of component, and the same reference document defines acceptance criteria in terms of allowable stresses rather than strengths, that reference document is permitted to be used. The allowable stress load combination shall consider dead, live, operating, and earthquake loads in addition to those in the reference document. The earthquake loads determined in accordance with Section 13.3.1 shall be multiplied by a factor of 0.7. The allowable stress design load combinations of Section 2.4 need not be used. The component shall also accommodate the relative displacements specified in Section 13.3.2.

13.2 GENERAL DESIGN REQUIREMENTS

13.2.1 Applicable Requirements for Architectural, Mechanical, and Electrical Components, Supports, and Attachments

Architectural, mechanical, and electrical components, supports, and attachments shall comply with the sections referenced in Table 13.2-1. These requirements shall be satisfied by one of the following methods:

1. Project-specific design and documentation submitted for approval to the authority having jurisdiction after review and acceptance by a registered design professional.
2. Submittal of the manufacturer's certification that the component is seismically qualified by at least one of the following:
   a. Analysis, or
   b. Testing in accordance with the alternative set forth in Section 13.2.5, or
   c. Experience data in accordance with the alternative set forth in Section 13.2.6.

13.2.2 Special Certification Requirements for Designated Seismic Systems

Certifications shall be provided for designated seismic systems assigned to Seismic Design Categories C through F as follows:

1. Active mechanical and electrical equipment that must remain operable following the design earthquake ground motion shall be certified by the manufacturer as operable whereby active parts or energized components shall be certified exclusively on the basis of approved shake table testing in accordance with Section 13.2.5 or experience data in accordance with Section 13.2.6 unless it can be

Table 13.2-1 Applicable Requirements for Architectural, Mechanical, and Electrical Components: Supports and Attachments

Nonstructural Element (i.e., Component, Support, Attachment)	General Design Requirements (Section 13.2)	Force and Displacement Requirements (Section 13.3)	Attachment Requirements (Section 13.4)	Architectural Component Requirements (Section 13.5)	Mechanical and Electrical Component Requirements (Section 13.6)
Architectural components and supports and attachments for architectural components	X	X	X	X	
Mechanical and electrical components with $I_p > 1$	X	X	X		X
Supports and attachments for mechanical and electrical components	X	X	X		X

shown that the component is inherently rugged by comparison with similar seismically qualified components. Evidence demonstrating compliance with this requirement shall be submitted for approval to the authority having jurisdiction after review and acceptance by a registered design professional.

2. Components with hazardous substances and assigned a component importance factor, I_p, of 1.5 in accordance with Section 13.1.3 shall be certified by the manufacturer as maintaining containment following the design earthquake ground motion by (1) analysis, (2) approved shake table testing in accordance with Section 13.2.5, or (3) experience data in accordance with Section 13.2.6. Evidence demonstrating compliance with this requirement shall be submitted for approval to the authority having jurisdiction after review and acceptance by a registered design professional.

13.2.3 Consequential Damage

The functional and physical interrelationship of components, their supports, and their effect on each other shall be considered so that the failure of an essential or nonessential architectural, mechanical, or electrical component shall not cause the failure of an essential architectural, mechanical, or electrical component.

13.2.4 Flexibility

The design and evaluation of components, their supports, and their attachments shall consider their flexibility as well as their strength.

13.2.5 Testing Alternative for Seismic Capacity Determination

As an alternative to the analytical requirements of Sections 13.2 through 13.6, testing shall be deemed as an acceptable method to determine the seismic capacity of components and their supports and attachments. Seismic qualification by testing based upon a nationally recognized testing standard procedure, such as ICC-ES AC 156, acceptable to the authority having jurisdiction shall be deemed to satisfy the design and evaluation requirements provided that the substantiated seismic capacities equal or exceed the seismic demands determined in accordance with Sections 13.3.1 and 13.3.2.

13.2.6 Experience Data Alternative for Seismic Capacity Determination

As an alternative to the analytical requirements of Sections 13.2 through 13.6, use of experience data shall be deemed as an acceptable method to determine the seismic capacity of components and their supports and attachments. Seismic qualification by experience data based upon nationally recognized procedures acceptable to the authority having jurisdiction shall be deemed to satisfy the design and evaluation requirements provided that the substantiated seismic capacities equal or exceed the seismic demands determined in accordance with Sections 13.3.1 and 13.3.2.

13.2.7 Construction Documents

Where design of nonstructural components or their supports and attachments is required by Table 13.2-1, such design shall be shown in construction documents prepared by a registered design professional for use by the owner, authorities having jurisdiction, contractors, and inspectors. Such documents shall include a quality assurance plan if required by Appendix 11A.

13.3 SEISMIC DEMANDS ON NONSTRUCTURAL COMPONENTS

13.3.1 Seismic Design Force

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. 13.3-1:

$$F_P = \frac{0.4 a_p S_{DS} W_p}{\left(\frac{R_p}{I_p}\right)}\left(1 + 2\frac{z}{h}\right) \quad (13.3\text{-}1)$$

F_p is not required to be taken as greater than

$$F_p = 1.6 S_{DS} I_p W_p \quad (13.3\text{-}2)$$

and F_p shall not be taken as less than

$$F_p = 0.3 S_{DS} I_p W_p \quad (13.3\text{-}3)$$

where

F_p = seismic design force

S_{DS} = spectral acceleration, short period, as determined from Section 11.4.4

a_p = component amplification factor that varies from 1.00 to 2.50 (select appropriate value from Table 13.5-1 or 13.6-1)

I_p = component importance factor that varies from 1.00 to 1.50 (see Section 13.1.3)

W_p = component operating weight

R_p = component response modification factor that varies from 1.00 to 12 (select appropriate value from Table 13.5-1 or 13.6-1)

z = height in structure of point of attachment of component with respect to the base. For items at or below the base, z shall be taken as 0. The value of z/h need not exceed 1.0

h = average roof height of structure with respect to the base

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. In addition, the component shall be designed for a concurrent vertical force $\pm 0.2S_{DS}W_p$. The redundancy factor, ρ, is permitted to be taken equal to 1 and the overstrength factor, Ω_0, does not apply.

EXCEPTION: The concurrent vertical seismic force need not be considered for lay-in access floor panels and lay-in ceiling panels.

Where nonseismic loads on nonstructural components exceed F_p, such loads shall govern the strength design, but the detailing requirements and limitations prescribed in this chapter shall apply.

In lieu of the forces determined in accordance with Eq. 13.3-1, accelerations at any level are permitted to be determined by the modal analysis procedures of Section 12.9 with R = 1.0. Seismic forces shall be in accordance with Eq. 13.3-4:

$$F_p = \frac{a_i a_p W_p}{\left(\frac{R_p}{I_p}\right)} A_x \qquad (13.3\text{-}4)$$

where a_i is the acceleration at level i obtained from the modal analysis and where A_x is the torsional amplification factor determined by Eq.12.8-14. Upper and lower limits of F_p determined by Eqs. 13.3-2 and 13.3-3 shall apply.

13.3.2 Seismic Relative Displacements

The effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate. Seismic relative displacements, D_{pI}, shall be determined in accordance with with Eq. 13.3-5 as:

$$D_{pI} = D_p I_e \qquad (13.3\text{-}5)$$

where

I_e = the importance factor in Section 11.5.1

D_p = displacement determined in accordance with the equations set forth in Sections 13.3.2.1 and 13.3.2.2.

13.3.2.1 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

$$D_p = \Delta_{xA} - \Delta_{yA} \qquad (13.3\text{-}6)$$

Alternatively, D_p is permitted to be determined using modal procedures described in Section 12.9, using the difference in story deflections calculated for each mode and then combined using appropriate modal combination procedures. D_p is not required to be taken as greater than

$$D_p = \frac{(h_x - h_y)\Delta_{aA}}{h_{sx}} \qquad (13.3\text{-}7)$$

13.3.2.2 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

$$D_p = |\delta_{xA}| + |\delta_{yB}| \qquad (13.3\text{-}8)$$

D_p is not required to be taken as greater than

$$D_p = \frac{h_x \Delta_{aA}}{h_{sx}} + \frac{h_y \Delta_{aB}}{h_{sx}} \qquad (13.3\text{-}9)$$

where

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

δ_{xA} = deflection at building Level x of Structure A, determined in accordance with Eq. (12.8-15)

δ_{yA} = deflection at building Level y of Structure A, determined in accordance with Eq. (12.8-15).

δ_{yB} = deflection at building Level y of Structure B, determined in accordance with Eq. (12.8-15).

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

Δ_{aA} = allowable story drift for Structure A as defined in Table 12.12-1

Δ_{aB} = allowable story drift for Structure B as defined in Table 12.12-1

h_{sx} = story height used in the definition of the allowable drift Δ_a in Table12.12-1. Note that Δ_a/h_{sx} = the drift index.

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

13.4 NONSTRUCTURAL COMPONENT ANCHORAGE

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

Component attachments 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 between the component and the supporting structure shall be provided. Local elements of the structure including connections shall be designed and constructed for the component forces where they control the design of the elements or their connections. The component forces shall be those determined in Section 13.3.1, except that modifications to F_p and R_p due to anchorage conditions need not be considered. The design documents shall include sufficient information relating to the attachments to verify compliance with the requirements of this section.

13.4.1 Design Force in the Attachment

The force in the attachment shall be determined based on the prescribed forces and displacements for the component as determined in Sections 13.3.1 and 13.3.2, except that R_p shall not be taken as larger than 6.

13.4.2 Anchors in Concrete or Masonry.

13.4.2.1 Anchors in Concrete

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

13.4.2.2 Anchors in Masonry

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 shall be permitted to be designed so that the attachment 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 component.

13.4.2.3 Post-Installed Anchors in Concrete and Masonry

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

13.4.3 Installation Conditions

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

13.4.4 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.

13.4.5 Power Actuated Fasteners

Power actuated fasteners in concrete or steel shall not be used for sustained tension loads or for brace applications in Seismic Design Categories D, E, or F unless approved for seismic loading. Power actuated fasteners in masonry are not permitted unless approved for seismic loading.

EXCEPTION: 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).

13.4.6 Friction Clips

Friction clips in Seismic Design Categories D, E, or F shall not be used for supporting sustained loads in addition to resisting seismic forces. C-type beam and large flange clamps are permitted for hangers provided they are equipped with restraining straps equivalent to those specified in NFPA 13, Section 9.3.7. Lock nuts or equivalent shall be provided to prevent loosening of threaded connections.

13.5 ARCHITECTURAL COMPONENTS

13.5.1 General

Architectural components, and their supports and attachments, shall satisfy the requirements of this section. Appropriate coefficients shall be selected from Table 13.5-1.

EXCEPTION: Components supported by chains or otherwise suspended from the structure are not required to satisfy the seismic force and relative displacement requirements provided they meet all of the following criteria:

1. The design load for such items shall be equal to 1.4 times the operating weight acting down with a simultaneous horizontal load equal to 1.4 times the operating weight. The horizontal load shall be applied in the direction that results in the most critical loading for design.
2. Seismic interaction effects shall be considered in accordance with Section 13.2.3.
3. The connection to the structure shall allow a 360° range of motion in the horizontal plane.

13.5.2 Forces and Displacements

All architectural components, and their supports and attachments, shall be designed for the seismic forces defined in Section 13.3.1.

Architectural components that could pose a life-safety hazard shall be designed to accommodate the seismic relative displacement requirements of Section 13.3.2. Architectural components shall be designed considering vertical deflection due to joint rotation of cantilever structural members.

13.5.3 Exterior Nonstructural Wall Elements and Connections

Exterior nonstructural wall panels or elements that are attached to or enclose the structure shall be designed to accommodate the seismic relative displacements defined in Section 13.3.2 and movements due to temperature changes. Such elements shall be supported by means of positive and direct structural supports or by mechanical connections and fasteners in accordance with the following requirements:

a. Connections and panel joints shall allow for the story drift caused by relative seismic displacements (D_p) determined in Section 13.3.2, or 0.5 in. (13 mm), whichever is greatest.
b. Connections to permit movement in the plane of the panel for story drift shall be sliding connections using slotted or oversize holes, connections that permit movement by bending of steel, or other connections that provide equivalent sliding or ductile capacity.
c. The connecting member itself shall have sufficient ductility and rotation capacity to preclude fracture of the concrete or brittle failures at or near welds.
d. All fasteners in the connecting system such as bolts, inserts, welds, and dowels and the body of the connectors shall be designed for the force (F_p) determined by Section 13.3.1 with values of R_p and a_p taken from Table 13.5-1 applied at the center of mass of the panel.
e. Where anchorage is achieved using flat straps embedded in concrete or masonry, such straps shall be attached to or hooked around reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel or to assure that pullout of anchorage is not the initial failure mechanism.

13.5.4 Glass

Glass in glazed curtain walls and storefronts shall be designed and installed in accordance with Section 13.5.9.

13.5.5 Out-of-Plane Bending

Transverse or out-of-plane bending or deformation of a component or system that is subjected to forces as determined in Section 13.5.2 shall not exceed the deflection capability of the component or system.

13.5.6 Suspended Ceilings

Suspended ceilings shall be in accordance with this section.

EXCEPTIONS:

1. Suspended ceilings with areas less than or equal to 144 ft^2 (13.4 m^2) that are surrounded by walls or soffits that are laterally braced to the structure above are exempt from the requirements of this section.
2. Suspended ceilings constructed of screw- or nail-attached gypsum board on one level that are surrounded by and connected to walls or soffits that are laterally braced to the structure above are exempt from the requirements of this section.

13.5.6.1 Seismic Forces

The weight of the ceiling, W_p, shall include the ceiling grid; ceiling tiles or panels; light fixtures if attached to, clipped to, or laterally supported by the ceiling grid; and other components that are laterally supported by the ceiling. W_p shall be taken as not less than 4 psf (192 N/m^2).

Table 13.5-1 Coefficients for Architectural Components

Architectural Component	a_p[a]	R_p[b]
Interior nonstructural walls and partitions[b]		
Plain (unreinforced) masonry walls	1.0	1.5
All other walls and partitions	1.0	2.5
Cantilever elements (Unbraced or braced to structural frame below its center of mass)		
Parapets and cantilever interior nonstructural walls	2.5	2.5
Chimneys where laterally braced or supported by the structural frame	2.5	2.5
Cantilever elements (Braced to structural frame above its center of mass)		
Parapets	1.0	2.5
Chimneys	1.0	2.5
Exterior nonstructural walls[b]	1.0[b]	2.5
Exterior nonstructural wall elements and connections[b]		
Wall element	1.0	2.5
Body of wall panel connections	1.0	2.5
Fasteners of the connecting system	1.25	1.0
Veneer		
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
Penthouses (except where framed by an extension of the building frame)	2.5	3.5
Ceilings		
All	1.0	2.5
Cabinets		
Permanent floor-supported storage cabinets over 6 ft (1,829 mm) tall, including contents	1.0	2.5
Permanent floor-supported library shelving, book stacks, and bookshelves over 6 ft (1,829 mm) tall, including contents	1.0	2.5
Laboratory equipment	1.0	2.5
Access floors		
Special access floors (designed in accordance with Section 13.5.7.2)	1.0	2.5
All other	1.0	1.5
Appendages and ornamentations	2.5	2.5
Signs and billboards	2.5	3.0
Other rigid components		
High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability materials and attachments	1.0	1.5
Other flexible components		
High deformability elements and attachments	2.5	3.5
Limited deformability elements and attachments	2.5	2.5
Low deformability materials and attachments	2.5	1.5
Egress stairways not part of the building structure	1.0	2.5

[a]A lower value for a_p shall not be used unless justified by detailed dynamic analysis. The value for a_p shall not be less than 1.00. The value of $a_p = 1$ is for rigid components and rigidly attached components. The value of $a_p = 2.5$ is for flexible components and flexibly attached components.
[b]Where flexible diaphragms provide lateral support for concrete or masonry walls and partitions, the design forces for anchorage to the diaphragm shall be as specified in Section 12.11.2.

The seismic force, F_p, shall be transmitted through the ceiling attachments to the building structural elements or the ceiling–structure boundary.

13.5.6.2 Industry Standard Construction for Acoustical Tile or Lay-in Panel Ceilings

Unless designed in accordance with Section 13.5.6.3, or seismically qualified in accordance with

Section 13.2.5 or 13.2.6, acoustical tile or lay-in panel ceilings shall be designed and constructed in accordance with this section.

13.5.6.2.1 Seismic Design Category C Acoustical tile or lay-in panel ceilings in structures assigned to Seismic Design Category C shall be designed and installed in accordance with ASTM C635, ASTM C636, and ASTM E580, Section 4—Seismic Design Category C.

13.5.6.2.2 Seismic Design Categories D through F Acoustical tile or lay-in panel ceilings in Seismic Design Categories D, E, and F shall be designed and installed in accordance with ASTM C635, ASTM C636, and ASTM E580, Section 5—Seismic Design Categories D, E, and F as modified by this section.

Acoustical tile or lay-in panel ceilings shall also comply with the following:

a. The width of the perimeter supporting closure angle or channel shall be not less than 2.0 in. (50 mm). Where perimeter supporting clips are used, they shall be qualified in accordance with approved test criteria. In each orthogonal horizontal direction, one end of the ceiling grid shall be attached to the closure angle or channel. The other end in each horizontal direction shall have a 0.75 in. (19 mm) clearance from the wall and shall rest upon and be free to slide on a closure angle or channel.
b. For ceiling areas exceeding 2,500 ft^2 (232 m^2), a seismic separation joint or full height partition that breaks the ceiling up into areas not exceeding 2,500 ft^2 (232 m^2), each with a ratio of the long to short dimension less than or equal to 4, shall be provided unless structural analyses are performed of the ceiling bracing system for the prescribed seismic forces that demonstrate ceiling penetrations and closure angles or channels provide sufficient clearance to accommodate the anticipated lateral displacement. Each area shall be provided with closure angles or channels in accordance with Section 13.5.6.2.2.a and horizontal restraints or bracing.

13.5.6.3 Integral Construction

As an alternate to providing large clearances around sprinkler system penetrations through ceilings, the sprinkler system and ceiling grid are permitted to be designed and tied together as an integral unit. Such a design shall consider the mass and flexibility of all elements involved, including the ceiling, sprinkler system, light fixtures, and mechanical (HVAC) appurtenances. Such design shall be performed by a registered design professional.

13.5.7 Access Floors

13.5.7.1 General

The weight of the access floor, W_p, shall include the weight of the floor system, 100 percent of the weight of all equipment fastened to the floor, and 25 percent of the weight of all equipment supported by but not fastened to the floor. The seismic force, F_p, shall be transmitted from the top surface of the access floor to the supporting structure.

Overturning effects of equipment fastened to the access floor panels also shall be considered. The ability of "slip on" heads for pedestals shall be evaluated for suitability to transfer overturning effects of equipment.

Where checking individual pedestals for overturning effects, the maximum concurrent axial load shall not exceed the portion of W_p assigned to the pedestal under consideration.

13.5.7.2 Special Access Floors

Access floors shall be considered to be "special access floors" if they are designed to comply with the following considerations:

1. Connections transmitting seismic loads consist of mechanical fasteners, anchors satisfying the requirements of Appendix D of ACI 318, welding, or bearing. Design load capacities comply with recognized design codes and/or certified test results.
2. Seismic loads are not transmitted by friction, power actuated fasteners, adhesives, or by friction produced solely by the effects of gravity.
3. The design analysis of the bracing system includes the destabilizing effects of individual members buckling in compression.
4. Bracing and pedestals are of structural or mechanical shapes produced to ASTM specifications that specify minimum mechanical properties. Electrical tubing shall not be used.
5. Floor stringers that are designed to carry axial seismic loads and that are mechanically fastened to the supporting pedestals are used.

13.5.8 Partitions

13.5.8.1 General

Partitions that are tied to the ceiling and all partitions greater than 6 ft (1.8 m) in height shall be laterally braced to the building structure. Such bracing shall be independent of any ceiling lateral force bracing. Bracing shall be spaced to limit horizontal deflection at the partition head to be compatible with ceiling deflection requirements as determined in Section 13.5.6 for suspended ceilings and elsewhere in this section for other systems.

EXCEPTION: Partitions that meet all of the following conditions:

1. The partition height does not exceed 9 ft (2,740 mm).
2. The linear weight of the partition does not exceed the product of 10 lb (0.479 kN) times the height (ft or m) of the partition.
3. The partition horizontal seismic load does not exceed 5 psf (0.24 kN/m^2).

13.5.8.2 Glass

Glass in glazed partitions shall be designed and installed in accordance with Section 13.5.9.

13.5.9 Glass in Glazed Curtain Walls, Glazed Storefronts, and Glazed Partitions

13.5.9.1 General

Glass in glazed curtain walls, glazed storefronts, and glazed partitions shall meet the relative displacement requirement of Eq. 13.5-1:

$$\Delta_{\text{fallout}} \geq 1.25 I_e D_p \qquad (13.5\text{-}1)$$

or 0.5 in. (13 mm), whichever is greater where:

Δ_{fallout} = the relative seismic displacement (drift) at which glass fallout from the curtain wall, storefront wall, or partition occurs (Section 13.5.9.2)

D_p = the relative seismic displacement that the component must be designed to accommodate (Section 13.3.2.1). D_p shall be applied over the height of the glass component under consideration

I_e = the importance factor determined in accordance with Section 11.5.1

EXCEPTION:

1. Glass with sufficient clearances from its frame such that physical contact between the glass and frame will not occur at the design drift, as demonstrated by Eq. 13.5-2, need not comply with this requirement:

$$D_{\text{clear}} \geq 1.25 D_p \qquad (13.5\text{-}2)$$

where

D_{clear} = relative horizontal (drift) displacement, measured over the height of the glass panel under consideration, which causes initial glass-to-frame contact. For rectangular glass panels within a rectangular wall frame

$$D_{\text{clear}} = 2c_1\left(1+\frac{h_p c_2}{b_p c_1}\right) \text{ where}$$

h_p = the height of the rectangular glass panel

b_p = the width of the rectangular glass panel

c_1 = the average of the clearances (gaps) on both sides between the vertical glass edges and the frame

c_2 = the average of the clearances (gaps) top and bottom between the horizontal glass edges and the frame

2. Fully tempered monolithic glass in Risk Categories I, II, and III located no more than 10 ft (3 m) above a walking surface need not comply with this requirement.
3. Annealed or heat-strengthened laminated glass in single thickness with interlayer no less than 0.030 in. (0.76 mm) that is captured mechanically in a wall system glazing pocket, and whose perimeter is secured to the frame by a wet glazed gunable curing elastomeric sealant perimeter bead of 0.5 in. (13 mm) minimum glass contact width, or other approved anchorage system need not comply with this requirement.

13.5.9.2 Seismic Drift Limits for Glass Components

Δ_{fallout}, the drift causing glass fallout from the curtain wall, storefront, or partition shall be determined in accordance with AAMA 501.6 or by engineering analysis.

13.6 MECHANICAL AND ELECTRICAL COMPONENTS

13.6.1 General

Mechanical and electrical components and their supports shall satisfy the requirements of this section. The attachment of mechanical and electrical components and their supports to the structure shall meet the requirements of Section 13.4. Appropriate coefficients shall be selected from Table 13.6-1.

EXCEPTION: Light fixtures, lighted signs, and ceiling fans not connected to ducts or piping, which are supported by chains or otherwise suspended from the structure, are not required to satisfy the seismic

Table 13.6-1 Seismic Coefficients for Mechanical and Electrical Components

Mechanical and Electrical Components	a_p[a]	R_p[b]
Air-side HVAC, fans, air handlers, air conditioning units, cabinet heaters, air distribution boxes, and other mechanical components constructed of sheet metal framing	2.5	6.0
Wet-side HVAC, boilers, furnaces, atmospheric tanks and bins, chillers, water heaters, heat exchangers, evaporators, air separators, manufacturing or process equipment, and other mechanical components constructed of high-deformability materials	1.0	2.5
Engines, turbines, pumps, compressors, and pressure vessels not supported on skirts and not within the scope of Chapter 15	1.0	2.5
Skirt-supported pressure vessels not within the scope of Chapter 15	2.5	2.5
Elevator and escalator components	1.0	2.5
Generators, batteries, inverters, motors, transformers, and other electrical components constructed of high deformability materials	1.0	2.5
Motor control centers, panel boards, switch gear, instrumentation cabinets, and other components constructed of sheet metal framing	2.5	6.0
Communication equipment, computers, instrumentation, and controls	1.0	2.5
Roof-mounted stacks, cooling and electrical towers laterally braced below their center of mass	2.5	3.0
Roof-mounted stacks, cooling and electrical towers laterally braced above their center of mass	1.0	2.5
Lighting fixtures	1.0	1.5
Other mechanical or electrical components	1.0	1.5
Vibration Isolated Components and Systems[b]		
Components and systems isolated using neoprene elements and neoprene isolated floors with built-in or separate elastomeric snubbing devices or resilient perimeter stops	2.5	2.5
Spring isolated components and systems and vibration isolated floors closely restrained using built-in or separate elastomeric snubbing devices or resilient perimeter stops	2.5	2.0
Internally isolated components and systems	2.5	2.0
Suspended vibration isolated equipment including in-line duct devices and suspended internally isolated components	2.5	2.5
Distribution Systems		
Piping in accordance with ASME B31, including in-line components with joints made by welding or brazing	2.5	12.0
Piping in accordance with ASME B31, including in-line components, constructed of high or limited deformability materials, with joints made by threading, bonding, compression couplings, or grooved couplings	2.5	6.0
Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing	2.5	9.0
Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high- or limited-deformability materials, with joints made by threading, bonding, compression couplings, or grooved couplings	2.5	4.5
Piping and tubing constructed of low-deformability materials, such as cast iron, glass, and nonductile plastics	2.5	3.0
Ductwork, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing	2.5	9.0
Ductwork, including in-line components, constructed of high- or limited-deformability materials with joints made by means other than welding or brazing	2.5	6.0
Ductwork, including in-line components, constructed of low-deformability materials, such as cast iron, glass, and nonductile plastics	2.5	3.0

Table 13.6-1 (*Continued*)

Distribution Systems		
Electrical conduit and cable trays	2.5	6.0
Bus ducts	1.0	2.5
Plumbing	1.0	2.5
Manufacturing or process conveyors (nonpersonnel)	2.5	3.0

[a]A lower value for a_p is permitted where justified by detailed dynamic analyses. The value for a_p shall not be less than 1.0. The value of a_p equal to 1.0 is for rigid components and rigidly attached components. The value of a_p equal to 2.5 is for flexible components and flexibly attached components.
[b]Components mounted on vibration isolators shall have a bumper restraint or snubber in each horizontal direction. The design force shall be taken as $2F_p$ if the nominal clearance (air gap) between the equipment support frame and restraint is greater than 0.25 in. (6 mm). If the nominal clearance specified on the construction documents is not greater than 0.25 in. (6 mm), the design force is permitted to be taken as F_p.

force and relative displacement requirements provided they meet all of the following criteria:

1. The design load for such items shall be equal to 1.4 times the operating weight acting down with a simultaneous horizontal load equal to 1.4 times the operating weight. The horizontal load shall be applied in the direction that results in the most critical loading for the design.
2. Seismic interaction effects shall be considered in accordance with Section 13.2.3.
3. The connection to the structure shall allow a 360° range of motion in the horizontal plane.

Where design of mechanical and electrical components for seismic effects is required, consideration shall be given to the dynamic effects of the components, their contents, and where appropriate, their supports and attachments. In such cases, the interaction between the components and the supporting structures, including other mechanical and electrical components, shall also be considered.

13.6.2 Component Period

The fundamental period of the nonstructural component (including its supports and attachment to the structure), T_p, shall be determined by the following equation provided that the component, supports, and attachment can be reasonably represented analytically by a simple spring and mass single degree-of-freedom system:

$$T_P = 2\pi\sqrt{\frac{W_p}{K_p g}} \qquad (13.6\text{-}1)$$

where

T_p = component fundamental period
W_p = component operating weight
g = gravitational acceleration
K_p = combined stiffness of the component, supports and attachments, determined in terms of load per unit deflection at the center of gravity of the component

Alternatively, the fundamental period of the component, T_p, in seconds is permitted to be determined from experimental test data or by a properly substantiated analysis.

13.6.3 Mechanical Components

HVAC ductwork shall meet the requirements of Section 13.6.7. Piping systems shall meet the requirements of Section 13.6.8. Boilers and vessels shall meet the requirements of Section 13.6.9. Elevators shall meet the requirements of Section 13.6.10. All other mechanical components shall meet the requirements of Section 13.6.11. Mechanical components with I_p greater than 1.0 shall be designed for the seismic forces and relative displacements defined in Sections 13.3.1 and 13.3.2 and shall satisfy the following additional requirements:

1. Provision shall be made to eliminate seismic impact for components vulnerable to impact, for components constructed of nonductile materials, and in cases where material ductility will be reduced due to service conditions (e.g., low temperature applications).
2. The possibility of loads imposed on components by attached utility or service lines, due to differential movement of support points on separate structures, shall be evaluated.
3. Where piping or HVAC ductwork components are attached to structures that could displace relative to one another and for isolated structures where such components cross the isolation interface, the

components shall be designed to accommodate the seismic relative displacements defined in Section 13.3.2.

13.6.4 Electrical Components

Electrical components with I_p greater than 1.0 shall be designed for the seismic forces and relative displacements defined in Sections 13.3.1 and 13.3.2 and shall satisfy the following additional requirements:

1. Provision shall be made to eliminate seismic impact between components.
2. Loads imposed on the components by attached utility or service lines that are attached to separate structures shall be evaluated.
3. Batteries on racks shall have wrap-around restraints to ensure that the batteries will not fall from the racks. Spacers shall be used between restraints and cells to prevent damage to cases. Racks shall be evaluated for sufficient lateral load capacity.
4. Internal coils of dry type transformers shall be positively attached to their supporting substructure within the transformer enclosure.
5. Electrical control panels, computer equipment, and other items with slide-out components shall have a latching mechanism to hold the components in place.
6. Electrical cabinet design shall comply with the applicable National Electrical Manufacturers Association (NEMA) standards. Cutouts in the lower shear panel that have not been made by the manufacturer and reduce significantly the strength of the cabinet shall be specifically evaluated.
7. The attachments for additional external items weighing more than 100 lb (445 N) shall be specifically evaluated if not provided by the manufacturer.
8. Where conduit, cable trays, or similar electrical distribution components are attached to structures that could displace relative to one another and for isolated structures where such components cross the isolation interface, the components shall be designed to accommodate the seismic relative displacements defined in Section 13.3.2.

13.6.5 Component Supports

Mechanical and electrical component supports (including those with $I_p = 1.0$) and the means by which they are attached to the component shall be designed for the forces and displacements determined in Sections 13.3.1 and 13.3.2. Such supports include structural members, braces, frames, skirts, legs, saddles, pedestals, cables, guys, stays, snubbers, and tethers, as well as elements forged or cast as a part of the mechanical or electrical component.

13.6.5.1 Design Basis

If standard supports, for example, ASME B31, NFPA 13, or MSS SP-58, or proprietary supports are used, they shall be designed by either load rating (i.e., testing) or for the calculated seismic forces. In addition, the stiffness of the support, where appropriate, shall be designed such that the seismic load path for the component performs its intended function.

13.6.5.2 Design for Relative Displacement

Component supports shall be designed to accommodate the seismic relative displacements between points of support determined in accordance with Section 13.3.2.

13.6.5.3 Support Attachment to Component

The means by which supports are attached to the component, except where integral (i.e., cast or forged), shall be designed to accommodate both the forces and displacements determined in accordance with Sections 13.3.1 and 13.3.2. If the value of I_p = 1.5 for the component, the local region of the support attachment point to the component shall be evaluated for the effect of the load transfer on the component wall.

13.6.5.4 Material Detailing Requirements

The materials comprising supports and the means of attachment to the component shall be constructed of materials suitable for the application, including the effects of service conditions, for example, low temperature applications. Materials shall be in conformance with a nationally recognized standard.

13.6.5.5 Additional Requirements

The following additional requirements shall apply to mechanical and electrical component supports:

1. Seismic supports shall be constructed so that support engagement is maintained.
2. Reinforcement (e.g., stiffeners or Belleville washers) shall be provided at bolted connections through sheet metal equipment housings as required to transfer the equipment seismic loads specified in this section from the equipment to the structure. Where equipment has been certified per Section 13.2.2, 13.2.5, or 13.2.6, anchor bolts or other fasteners and associated hardware as included in the certification shall be installed in conformance

with the manufacturer's instructions. For those cases where no certification exists or where instructions for such reinforcement are not provided, reinforcement methods shall be as specified by a registered design professional or as approved by the authority having jurisdiction.
3. Where weak-axis bending of cold-formed steel supports is relied on for the seismic load path, such supports shall be specifically evaluated.
4. Components mounted on vibration isolators shall have a bumper restraint or snubber in each horizontal direction, and vertical restraints shall be provided where required to resist overturning. Isolator housings and restraints shall be constructed of ductile materials. (See additional design force requirements in footnote *b* to Table 13.6-1.) A viscoelastic pad or similar material of appropriate thickness shall be used between the bumper and components to limit the impact load.
5. Where post-installed mechanical anchors are used for non-vibration isolated mechanical equipment rated over 10 hp (7.45 kW), they shall be qualified in accordance with ACI 355.2.
6. For piping, boilers, and pressure vessels, attachments to concrete shall be suitable for cyclic loads.
7. For mechanical equipment, drilled and grouted-in-place anchors for tensile load applications shall use either expansive cement or expansive epoxy grout.

13.6.5.6 Conduit, Cable Tray, and Other Electrical Distribution Systems (Raceways)

Raceways shall be designed for seismic forces and seismic relative displacements as required in Section 13.3. Conduit greater than 2.5 in. (64 mm) trade size and attached to panels, cabinets, or other equipment subject to seismic relative displacement, D_p, shall be provided with flexible connections or designed for seismic forces and seismic relative displacements as required in Section 13.3.

EXCEPTIONS:

1. Design for the seismic forces and relative displacements of Section 13.3 shall not be required for raceways where either:
   a. Trapeze assemblies are used to support raceways and the total weight of the raceway supported by trapeze assemblies is less than 10 lb/ft (146 N/m), or
   b. The raceway is supported by hangers and each hanger in the raceway run is 12 in. (305 mm) or less in length from the raceway support point to the supporting structure. Where rod hangers are used, they shall be equipped with swivels to prevent inelastic bending in the rod.
2. Design for the seismic forces and relative displacements of Section 13.3 shall not be required for conduit, regardless of the value of I_p, where the conduit is less than 2.5 in. (64 mm) trade size.

13.6.6 Utility and Service Lines

At the interface of adjacent structures or portions of the same structure that may move independently, utility lines shall be provided with adequate flexibility to accommodate the anticipated differential movement between the portions that move independently. Differential displacement calculations shall be determined in accordance with Section 13.3.2.

The possible interruption of utility service shall be considered in relation to designated seismic systems in Risk Category IV as defined in Table 1.5-1. Specific attention shall be given to the vulnerability of underground utilities and utility interfaces between the structure and the ground where Site Class E or F soil is present, and where the seismic coefficient S_{DS} at the underground utility or at the base of the structure is equal to or greater than 0.33.

13.6.7 Ductwork

HVAC and other ductwork shall be designed for seismic forces and seismic relative displacements as required in Section 13.3. Design for the displacements across seismic joints shall be required for ductwork with I_p = 1.5 without consideration of the exceptions below.

EXCEPTIONS: The following exceptions pertain to ductwork not designed to carry toxic, highly toxic, or flammable gases or used for smoke control:

1. Design for the seismic forces and relative displacements of Section 13.3 shall not be required for ductwork where either:
   a. Trapeze assemblies are used to support ductwork and the total weight of the ductwork supported by trapeze assemblies is less than 10 lb/ft (146 N/m); or
   b. The ductwork is supported by hangers and each hanger in the duct run is 12 in. (305 mm) or less in length from the duct support point to the supporting structure. Where rod hangers are used, they shall be equipped with swivels to prevent inelastic bending in the rod.
2. Design for the seismic forces and relative displacements of Section 13.3 shall not be required where provisions are made to avoid impact with larger

ducts or mechanical components or to protect the ducts in the event of such impact; and HVAC ducts have a cross-sectional area of less than 6 ft^2 (0.557 m^2), or weigh 17 lb/ft (248 N/m) or less.

HVAC duct systems fabricated and installed in accordance with standards approved by the authority having jurisdiction shall be deemed to meet the lateral bracing requirements of this section.

Components that are installed in-line with the duct system and have an operating weight greater than 75 lb (334 N), such as fans, heat exchangers, and humidifiers, shall be supported and laterally braced independent of the duct system and such braces shall meet the force requirements of Section 13.3.1. Appurtenances such as dampers, louvers, and diffusers shall be positively attached with mechanical fasteners. Unbraced piping attached to in-line equipment shall be provided with adequate flexibility to accommodate the seismic relative displacements of Section 13.3.2.

13.6.8 Piping Systems

Unless otherwise noted in this section, piping systems shall be designed for the seismic forces and seismic relative displacements of Section 13.3. ASME pressure piping systems shall satisfy the requirements of Section 13.6.8.1. Fire protection sprinkler piping shall satisfy the requirements of Section 13.6.8.2. Elevator system piping shall satisfy the requirements of Section 13.6.10.

Where other applicable material standards or recognized design bases are not used, piping design including consideration of service loads shall be based on the following allowable stresses:

a. For piping constructed with ductile materials (e.g., steel, aluminum, or copper), 90 percent of the minimum specified yield strength.
b. For threaded connections in piping constructed with ductile materials, 70 percent of the minimum specified yield strength.
c. For piping constructed with nonductile materials (e.g., cast iron or ceramics), 10 percent of the material minimum specified tensile strength.
d. For threaded connections in piping constructed with nonductile materials, 8 percent of the material minimum specified tensile strength.

Piping not detailed to accommodate the seismic relative displacements at connections to other components shall be provided with connections having sufficient flexibility to avoid failure of the connection between the components.

13.6.8.1 ASME Pressure Piping Systems

Pressure piping systems, including their supports, designed and constructed in accordance with ASME B31 shall be deemed to meet the force, displacement, and other requirements of this section. In lieu of specific force and displacement requirements provided in ASME B31, the force and displacement requirements of Section 13.3 shall be used. Materials meeting the toughness requirements of ASME B31 shall be considered high-deformability materials.

13.6.8.2 Fire Protection Sprinkler Piping Systems

Fire protection sprinkler piping, pipe hangers, and bracing designed and constructed in accordance with NFPA 13 shall be deemed to meet the force and displacement requirements of this section. The exceptions of Section 13.6.8.3 shall not apply.

13.6.8.3 Exceptions

Design of piping systems and attachments for the seismic forces and relative displacements of Section 13.3 shall not be required where one of the following conditions apply:

1. Trapeze assemblies are used to support piping whereby no single pipe exceeds the limits set forth in 3a, 3b, or 3c below and the total weight of the piping supported by the trapeze assemblies is less than 10 lb/ft (146 N/m).
2. The piping is supported by hangers and each hanger in the piping run is 12 in. (305 mm) or less in length from the top of the pipe to the supporting structure. Where pipes are supported on a trapeze, the trapeze shall be supported by hangers having a length of 12 in. (305 mm) or less. Where rod hangers are used, they shall be equipped with swivels, eye nuts, or other devices to prevent bending in the rod.
3. Piping having an R_p in Table 13.6-1 of 4.5 or greater is used and provisions are made to avoid impact with other structural or nonstructural components or to protect the piping in the event of such impact and where the following size requirements are satisfied:
   a. For Seismic Design Category C where I_p is greater than 1.0, the nominal pipe size shall be 2 in. (50 mm) or less.
   b. For Seismic Design Categories D, E, or F and values of I_p are greater than 1.0, the nominal pipe size shall be 1 in. (25 mm) or less.
   c. For Seismic Design Categories D, E, or F where I_p = 1.0, the nominal pipe size shall be 3 in. (80 mm) or less.

13.6.9 Boilers and Pressure Vessels

Boilers or pressure vessels designed and constructed in accordance with ASME BPVC shall be deemed to meet the force, displacement, and other requirements of this section. In lieu of the specific force and displacement requirements provided in the ASME BPVC, the force and displacement requirements of Sections 13.3.1 and 13.3.2 shall be used. Materials meeting the toughness requirements of ASME BPVC shall be considered high-deformability materials. Other boilers and pressure vessels designated as having an I_p = 1.5, but not designed and constructed in accordance with the requirements of ASME BPVC, shall comply with the requirements of Section 13.6.11.

13.6.10 Elevator and Escalator Design Requirements

Elevators and escalators designed in accordance with the seismic requirements of ASME A17.1 shall be deemed to meet the seismic force requirements of this section, except as modified in the following text. The exceptions of Section 13.6.8.3 shall not apply to elevator piping.

13.6.10.1 Escalators, Elevators, and Hoistway Structural System

Escalators, elevators, and hoistway structural systems shall be designed to meet the force and displacement requirements of Sections 13.3.1 and 13.3.2.

13.6.10.2 Elevator Equipment and Controller Supports and Attachments

Elevator equipment and controller supports and attachments shall be designed to meet the force and displacement requirements of Sections 13.3.1 and 13.3.2.

13.6.10.3 Seismic Controls for Elevators

Elevators operating with a speed of 150 ft/min (46 m/min) or greater shall be provided with seismic switches. Seismic switches shall provide an electric signal indicating that structural motions are of such a magnitude that the operation of the elevators may be impaired. Seismic switches in accordance with Section 8.4.10.1.2 of ASME A17.1 shall be deemed to meet the requirements of this section.

EXCEPTION: In cases where seismic switches cannot be located near a column in accordance with ASME A17.1, they shall have two horizontal axes of sensitivity and have a trigger level set to 20 percent of the acceleration of gravity where located at or near the base of the structure and 50 percent of the acceleration of gravity in all other locations.

Upon activation of the seismic switch, elevator operations shall conform to requirements of ASME A17.1, except as noted in the following text.

In facilities where the loss of the use of an elevator is a life-safety issue, the elevator shall only be used after the seismic switch has triggered provided that:

1. The elevator shall operate no faster than the service speed.
2. Before the elevator is occupied, it is operated from top to bottom and back to top to verify that it is operable.

13.6.10.4 Retainer Plates

Retainer plates are required at the top and bottom of the car and counterweight.

13.6.11 Other Mechanical and Electrical Components

Mechanical and electrical components, including conveyor systems, not designed and constructed in accordance with the reference documents in Chapter 23 shall meet the following:

1. Components, their supports and attachments shall comply with the requirements of Sections 13.4, 13.6.3, 13.6.4, and 13.6.5.
2. For mechanical components with hazardous substances and assigned a component importance factor, I_p, of 1.5 in accordance with Section 13.1.3, and for boilers and pressure vessels not designed in accordance with ASME BPVC, the design strength for seismic loads in combination with other service loads and appropriate environmental effects shall be based on the following material properties:
   a. For mechanical components constructed with ductile materials (e.g., steel, aluminum, or copper), 90 percent of the minimum specified yield strength.
   b. For threaded connections in components constructed with ductile materials, 70 percent of the minimum specified yield strength.
   c. For mechanical components constructed with nonductile materials (e.g., plastic, cast iron, or ceramics), 10 percent of the material minimum specified tensile strength.
   d. For threaded connections in components constructed with nonductile materials, 8 percent of the material minimum specified tensile strength.

Chapter 14

MATERIAL SPECIFIC SEISMIC DESIGN AND DETAILING REQUIREMENTS

14.0 SCOPE

Structural elements including foundation elements shall conform to the material design and detailing requirements set forth in this chapter or as otherwise specified for non-building structures in Tables 15.4-1 and 15.4-2.

14.1 STEEL

Structures, including foundations, constructed of steel to resist seismic loads shall be designed and detailed in accordance with this standard including the reference documents and additional requirements provided in this section.

14.1.1 Reference Documents

The design, construction, and quality of steel members that resist seismic forces shall conform to the applicable requirements, as amended herein, of the following:

1. AISC 360
2. AISC 341
3. AISI S100
4. AISI S110
5. AISI S230
6. AISI S213
7. ASCE 19
8. ASCE 8
9. SJI-K-1.1
10. SJI-LH/DLH-1.1
11. SJI-JG-1.1
12. SJI-CJ-1.0

14.1.2 Structural Steel

14.1.2.1 General

The design of structural steel for buildings and structures shall be in accordance with AISC 360. Where required, the seismic design of structural steel structures shall be in accordance with the additional provisions of Section 14.1.2.2.

14.1.2.2 Seismic Requirements for Structural Steel Structures

The design of structural steel structures to resist seismic forces shall be in accordance with the provisions of Section 14.1.2.2.1 or 14.1.2.2.2, as applicable.

14.1.2.2.1 Seismic Design Categories B and C
Structural steel structures assigned to Seismic Design Category B or C shall be of any construction permitted by the applicable reference documents in Section 14.1.1. Where a response modification coefficient, *R*, in accordance with Table 12.2-1 is used for the design of structural steel structures assigned to Seismic Design Category B or C, the structures shall be designed and detailed in accordance with the requirements of AISC 341.

EXCEPTION: The response modification coefficient, *R,* designated for "Steel systems not specifically detailed for seismic resistance, excluding cantilever column systems" in Table 12.2-1 shall be permitted for systems designed and detailed in accordance with AISC 360 and need not be designed and detailed in accordance with AISC 341.

14.1.2.2.2 Seismic Design Categories D through F
Structural steel structures assigned to Seismic Design Category D, E, or F shall be designed and detailed in accordance with AISC 341, except as permitted in Table 15.4-1.

14.1.3 Cold-Formed Steel

14.1.3.1 General

The design of cold-formed carbon or low-alloy steel structural members shall be in accordance with the requirements of AISI S100 and the design of cold-formed stainless steel structural members shall be in accordance with the requirements of ASCE 8. Where required, the seismic design of cold-formed steel structures shall be in accordance with the additional provisions of Section 14.1.3.2.

14.1.3.2 Seismic Requirements for Cold-Formed Steel Structures

Where a response modification coefficient, *R*, in accordance with Table 12.2-1 is used for the design of cold-formed steel structures, the structures shall be designed and detailed in accordance with the requirements of AISI S100, ASCE 8, and AISI S110 as modified in Section 14.1.3.3.

14.1.3.3 Modifications to AISI S110

The text of AISI S110 shall be modified as indicated in Sections 14.1.3.3.1 through 14.1.3.3.5. Italics are used for text within Sections 14.1.3.3.1 through 14.1.3.3.5 to indicate requirements that differ from AISI S110.

14.1.3.3.1 AISI S110, Section D1 Modify Section D1 to read as follows:

D1 Cold-Formed Steel Special Bolted Moment Frames (CFS-SBMF)

Cold-formed steel–special bolted moment frame (CFS-SBMF) systems shall withstand significant inelastic deformations through friction and bearing at their bolted connections. Beams, columns, and connections shall satisfy the requirements in this section. CFS-SBMF systems shall be limited to one-story structures, no greater than 35 feet in height, without column splices and satisfying the requirements in this section. *The CFS-SBMF shall engage all columns supporting the roof or floor above. The single size beam and single size column with the same bolted moment connection detail shall be used for each frame. The frame shall be supported on a level floor or foundation.*

14.1.3.3.2 AISI S110, Section D1.1.1 Modify Section D1.1.1 to read as follows:

D1.1.1 Connection Limitations

Beam-to-column connections in CFS-SBMF systems shall be bolted connections with snug-tight high-strength bolts. The bolt spacing and edge distance shall be in accordance with the limits of AISI S100, Section E3. *The 8-bolt configuration shown in Table D1-1 shall be used. The faying surfaces of the beam and column in the bolted moment connection region shall be free of lubricants or debris.*

14.1.3.3.3 AISI S110, Section D1.2.1 Modify Section D1.2.1 and add new Section D1.2.1.1 to read as follows:

D1.2.1 Beam Limitations

In addition to the requirements of Section D1.2.3, beams in CFS-SBMF systems shall be *ASTM A653 galvanized 55 ksi (374 MPa) yield stress* cold-formed *steel* C-section members with lips, and designed in accordance with Chapter C of AISI S100. *The beams shall have a minimum design thickness of 0.105 in. (2.67 mm). The beam depth shall be not less than 12 in. (305 mm) or greater than 20 in. (508 mm).* The flat depth-to-thickness ratio of the web shall not exceed $6.18\sqrt{E/F_y}$.

D1.2.1.1 Single-Channel Beam Limitations

When single-channel beams are used, torsional effects shall be accounted for in the design.

14.1.3.3.4 AISI S110, Section D1.2.2 Modify Section D1.2.2 to read as follows:

D1.2.2 Column Limitations

In addition to the requirements of D1.2.3, columns in CFS-SBMF systems shall be *ASTM A500 Grade B* cold-formed *steel* hollow structural section (HSS) members *painted with a standard industrial finished surface,* and designed in accordance with Chapter C of AISI S100. *The column depth shall be not less than 8 in. (203 mm) or greater than 12 in. (305 mm).* The flat depth-to-thickness ratio shall not exceed $1.40\sqrt{E/F_y}$.

14.1.3.3.5 AISI S110, Section D1.3 Delete text in Section D1.3 to read as follows:

D1.3 Design Story Drift

Where the applicable building code does not contain design coefficients for CSF-SBMF systems, the provisions of Appendix 1 shall apply.

For structures having a period less than T_S, as defined in the applicable building code, alternate methods of computing Δ shall be permitted, provided such alternate methods are acceptable to the authority having jurisdiction.

14.1.4 Cold-Formed Steel Light-Frame Construction

14.1.4.1 General

Cold-formed steel light-frame construction shall be designed in accordance with AISI S100, Section D4. Where required, the seismic design of cold-formed steel light-frame construction shall be in accordance with the additional provisions of Section 14.1.4.2.

14.1.4.2 Seismic Requirements for Cold-Formed Steel Light-Frame Construction

Where a response modification coefficient, R, in accordance with Table 12.2-1 is used for the design of cold-formed steel light-frame construction, the structures shall be designed and detailed in accordance with the requirements of AISI S213.

14.1.4.3 Prescriptive Cold-Formed Steel Light-Frame Construction

Cold-formed steel light-frame construction for one- and two-family dwellings is permitted to be designed and constructed in accordance with the requirements of AISI S230 subject to the limitations therein.

14.1.5 Steel Deck Diaphragms

Steel deck diaphragms shall be made from materials conforming to the requirements of AISI S100 or ASCE 8. Nominal strengths shall be determined in accordance with approved analytical procedures or with test procedures prepared by a registered design professional experienced in testing of cold-formed steel assemblies and approved by the authority having jurisdiction. The required strength of diaphragms, including bracing members that form part of the diaphragm, shall be determined in accordance with Section 12.10.1. The steel deck installation for the building, including fasteners, shall comply with the test assembly arrangement. Quality standards established for the nominal strength test shall be the minimum standards required for the steel deck installation, including fasteners.

14.1.6 Steel Cables

The design strength of steel cables shall be determined by the requirements of ASCE 19 except as modified by this chapter. ASCE 19, Section 3.1.2(d), shall be modified by substituting $1.5(T_4)$ where T_4 is the net tension in cable due to dead load, prestress, live load, and seismic load. A load factor of 1.1 shall be applied to the prestress force to be added to the load combination of Section 3.1.2 of ASCE 19.

14.1.7 Additional Detailing Requirements for Steel Piles in Seismic Design Categories D through F

In addition to the foundation requirements set forth in Sections 12.1.5 and 12.13, design and detailing of H-piles shall conform to the requirements of AISC 341, and the connection between the pile cap and steel piles or unfilled steel pipe piles in structures assigned to Seismic Design Category D, E, or F shall be designed for a tensile force not less than 10 percent of the pile compression capacity.

EXCEPTION: Connection tensile capacity need not exceed the strength required to resist seismic load effects including overstrength factor of Section 12.4.3.2 or Section 12.14.2.2.2. Connections need not be provided where the foundation or supported structure does not rely on the tensile capacity of the piles for stability under the design seismic forces.

14.2 CONCRETE

Structures, including foundations, constructed of concrete to resist seismic loads shall be designed and detailed in accordance with this standard including the reference documents and additional requirements provided in this section.

14.2.1 Reference Documents

The quality and testing of concrete materials and the design and construction of structural concrete members that resist seismic forces shall conform to the requirements of ACI 318, except as modified in Section 14.2.2.

14.2.2 Modifications to ACI 318

The text of ACI 318 shall be modified as indicated in Sections 14.2.2.1 through 14.2.2.9. Italics are used for text within Sections 14.2.2.1 through 14.2.2.9 to indicate requirements that differ from ACI 318.

14.2.2.1 Definitions

Add the following definitions to Section 2.2.

DETAILED PLAIN CONCRETE STRUCTURAL WALL: *A wall complying with the requirements of Chapter 22.*

ORDINARY PRECAST STRUCTURAL WALL: *A precast wall complying with the requirements of Chapters 1 through 18.*

WALL PIER: *A wall segment with a horizontal length-to-thickness ratio of at least 2.5, but not exceeding 6, whose clear height is at least two times its horizontal length.*

14.2.2.2 ACI 318, Section 7.10

Modify Section 7.10 by revising Section 7.10.5.6 to read as follows:

7.10.5.6 Where anchor bolts are placed in the top of columns or pedestals, the bolts shall be enclosed by lateral reinforcement that also surrounds at least four vertical bars of the column or pedestal. The lateral reinforcement shall be distributed within 5 in. of the top of the column or pedestal, and shall consist of at least two No. 4 or three No. 3 bars. *In structures assigned to Seismic Design Categories C, D, E, or F, the ties shall have a hook on each free end that complies with 7.1.4.*

14.2.2.3 Scope

Modify Section 21.1.1.3 to read as follows:

21.1.1.3 All members shall satisfy requirements of Chapters 1 to 19 and 22. Structures assigned to SDC B, C, D, E, or F also shall satisfy 21.1.1.4 through 21.1.1.8, as applicable*, except as modified by the requirements of Chapters 14 and 15 of this standard*.

14.2.2.4 Intermediate Precast Structural Walls

Modify Section 21.4 by renumbering Section 21.4.3 to Section 21.4.4 and adding new Sections 21.4.3, 21.4.5, and 21.4.6 to read as follows:

21.4.3 *Connections that are designed to yield shall be capable of maintaining 80 percent of their design strength at the deformation induced by design displacement, or shall use type 2 mechanical splices.*

21.4.4 *Elements of the connection that are not designed to yield shall develop at least 1.5* S_y.

21.4.5 *Wall piers in structures assigned to SDC D, E, or F shall comply with Section 14.2.2.4 of this standard.*

21.4.6 *Wall piers not designed as part of a moment frame in SDC C shall have transverse reinforcement designed to resist the shear forces determined from Section 21.3.3. Spacing of transverse reinforcement shall not exceed 8 in. Transverse reinforcement shall be extended beyond the pier clear height for at least 12 in.*

EXCEPTIONS: *The preceding requirement need not apply in the following situations:*

1. *Wall piers that satisfy Section 21,13.*
2. *Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers and such segments have a total stiffness of at least six times the sum of the stiffnesses of all the wall piers.*

Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.

14.2.2.5 Wall Piers and Wall Segments

Modify Section 21.9 by adding a new Section 21.9.10 to read as follows:

21.9.10 Wall Piers and Wall Segments.

21.9.10.1 *Wall piers not designed as a part of a special moment-resisting frame shall have transverse reinforcement designed to satisfy the requirements in Section 21.9.10.2.*

EXCEPTIONS:

1. *Wall piers that satisfy Section 21.13.*
2. *Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers, and such segments have a total stiffness of at least six times the sum of the in-plane stiffnesses of all the wall piers.*

21.9.10.2 *Transverse reinforcement with seismic hooks at both ends shall be designed to resist the shear forces determined from Section 21.6.5.1. Spacing of transverse reinforcement shall not exceed 6 in. (152 mm). Transverse reinforcement shall be extended beyond the pier clear height for at least 12 in. (304 mm).*

21.9.10.3 *Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.*

14.2.2.6 Special Precast Structural Walls

Modify Section 21.10.2 to read as follows:

21.10.2 Special structural walls constructed using precast concrete shall satisfy all requirements of *Section 21.9* in addition *to Section 21.4 as modified by Section 14.2.2.*

14.2.2.7 Foundations

Modify Section 21.12.1.1 to read as follows:

21.12.1.1 Foundations resisting earthquake-induced forces or transferring earthquake-induced forces between structure and ground in structures assigned to SDC D, E, or F shall comply with *requirements of Section* 21.12 and other applicable code provisions *unless modified by Sections 12.1.5, 12.13, or 14.2 of ASCE 7.*

14.2.2.8 Detailed Plain Concrete Shear Walls

Modify Section 22.6 by adding a new Section 22.6.7 to read

22.6.7 Detailed Plain Concrete Shear Walls.

22.6.7.1 *Detailed plain concrete shear walls are walls conforming to the requirements for ordinary plain concrete shear walls and Section 22.6.7.2.*

22.6.7.2 *Reinforcement shall be provided as follows:*

a. *Vertical reinforcement of at least 0.20 in.*^{2} *(129 mm*^{2}*) in cross-sectional area shall be provided continuously from support to support at each corner, at each side of each opening, and at the ends of walls. The continuous vertical bar required beside an opening is permitted to substitute for the No. 5 bar required by Section 22.6.6.5.*
b. *Horizontal reinforcement at least 0.20 in.*^{2} *(129 mm*^{2}*) in cross-sectional area shall be provided:*
   1. *Continuously at structurally connected roof and floor levels and at the top of walls.*

2. *At the bottom of load-bearing walls or in the top of foundations where doweled to the wall.*
3. *At a maximum spacing of 120 in. (3,048 mm).*

Reinforcement at the top and bottom of openings, where used in determining the maximum spacing specified in Item 3 in the preceding text, shall be continuous in the wall.

14.2.2.9 Strength Requirements for Anchors

Modify Section D.4 by adding a new exception at the end of Section D.4.2.2 to read as follows:

EXCEPTION: *If* N_b *is determined using Eq. D-7, the concrete breakout strength of Section D.4.2 shall be considered satisfied by the design procedure of Sections D.5.2 and D.6.2 without the need for testing regardless of anchor bolt diameter and tensile embedment.*

14.2.3 Additional Detailing Requirements for Concrete Piles

In addition to the foundation requirements set forth in Sections 12.1.5 and 12.13 of this standard and in Section 21.12 of ACI 318, design, detailing, and construction of concrete piles shall conform to the requirements of this section.

14.2.3.1 Concrete Pile Requirements for Seismic Design Category C

Concrete piles in structures assigned to Seismic Design Category C shall comply with the requirements of this section.

14.2.3.1.1 Anchorage of Piles All concrete piles and concrete-filled pipe piles shall be connected to the pile cap by embedding the pile reinforcement in the pile cap for a distance equal to the development length as specified in ACI 318 as modified by Section 14.2.2 of this standard or by the use of field-placed dowels anchored in the concrete pile. For deformed bars, the development length is the full development length for compression or tension, in the case of uplift, without reduction in length for excess area.

Hoops, spirals, and ties shall be terminated with seismic hooks as defined in Section 2.2 of ACI 318.

Where a minimum length for reinforcement or the extent of closely spaced confinement reinforcement is specified at the top of the pile, provisions shall be made so that those specified lengths or extents are maintained after pile cutoff.

14.2.3.1.2 Reinforcement for Uncased Concrete Piles (SDC C) Reinforcement shall be provided where required by analysis. For uncased cast-in-place drilled or augered concrete piles, a minimum of four longitudinal bars, with a minimum longitudinal reinforcement ratio of 0.0025, and transverse reinforcement, as defined below, shall be provided throughout the minimum reinforced length of the pile as defined below starting at the top of the pile. The longitudinal reinforcement shall extend beyond the minimum reinforced length of the pile by the tension development length. Transverse reinforcement shall consist of closed ties (or equivalent spirals) with a minimum 3/8 in. (9 mm) diameter. Spacing of transverse reinforcing shall not exceed 6 in. (150 mm) or 8 longitudinal-bar diameters within a distance of three times the pile diameter from the bottom of the pile cap. Spacing of transverse reinforcing shall not exceed 16 longitudinal-bar diameters throughout the remainder of the minimum reinforced length.

The minimum reinforced length of the pile shall be taken as the greater of

1. One-third of the pile length.
2. A distance of 10 ft (3 m).
3. Three times the pile diameter.
4. The flexural length of the pile, which shall be taken as the length from the bottom of the pile cap to a point where the concrete section cracking moment multiplied by a resistance factor 0.4 exceeds the required factored moment at that point.

14.2.3.1.3 Reinforcement for Metal-Cased Concrete Piles (SDC C) Reinforcement requirements are the same as for uncased concrete piles.

EXCEPTION: Spiral-welded metal casing of a thickness not less than No. 14 gauge can be considered as providing concrete confinement equivalent to the closed ties or equivalent spirals required in an uncased concrete pile, provided that the metal casing is adequately protected against possible deleterious action due to soil constituents, changing water levels, or other factors indicated by boring records of site conditions.

14.2.3.1.4 Reinforcement for Concrete-Filled Pipe Piles (SDC C) Minimum reinforcement 0.01 times the cross-sectional area of the pile concrete shall be provided in the top of the pile with a length equal to two times the required cap embedment anchorage into the pile cap.

14.2.3.1.5 Reinforcement for Precast Nonprestressed Concrete Piles (SDC C) A minimum longitudinal steel reinforcement ratio of 0.01 shall be provided for precast nonprestressed concrete piles. The longitudinal reinforcing shall be confined with closed ties or equivalent spirals of a minimum 3/8 in. (10 mm) diameter. Transverse confinement reinforcing shall be provided at a maximum spacing of eight times the diameter of the smallest longitudinal bar, but not to exceed 6 in. (152 mm), within three pile diameters of the bottom of the pile cap. Outside of the confinement region, closed ties or equivalent spirals shall be provided at a 16 longitudinal-bar-diameter maximum spacing, but not greater than 8 in. (200 mm). Reinforcement shall be full length.

14.2.3.1.6 Reinforcement for Precast Prestressed Piles (SDC C) For the upper 20 ft (6 m) of precast prestressed piles, the minimum volumetric ratio of spiral reinforcement shall not be less than 0.007 or the amount required by the following equation:

$$\rho_s = \frac{0.12 f_c'}{f_{yh}} \tag{14.2-1}$$

where

ρ_s = volumetric ratio (vol. spiral/vol. core)
f_c' = specified compressive strength of concrete, psi (MPa)
f_{yh} = specified yield strength of spiral reinforcement, which shall not be taken greater than 85,000 psi (586 MPa)

A minimum of one-half of the volumetric ratio of spiral reinforcement required by Eq. 14.2-1 shall be provided for the remaining length of the pile.

14.2.3.2 Concrete Pile Requirements for Seismic Design Categories D through F

Concrete piles in structures assigned to Seismic Design Category D, E, or F shall comply with the requirements of this section.

14.2.3.2.1 Site Class E or F Soil Where concrete piles are used in Site Class E or F, they shall have transverse reinforcement in accordance with Sections 21.6.4.2 through 21.6.4.4 of ACI 318 within seven pile diameters of the pile cap and of the interfaces between strata that are hard or stiff and strata that are liquefiable or are composed of soft to medium stiff clay.

14.2.3.2.2 Nonapplicable ACI 318 Sections for Grade Beam and Piles Section 21.12.3.3 of ACI 318 need not apply to grade beams designed to resist the seismic load effects including overstrength factor of Section 12.4.3 or 12.14.3.2. Section 21.12.4.4(a) of ACI 318 need not apply to concrete piles. Section 21.12.4.4(b) of ACI 318 need not apply to precast, prestressed concrete piles.

14.2.3.2.3 Reinforcement for Uncased Concrete Piles (SDC D through F) Reinforcement shall be provided where required by analysis. For uncased cast-in-place drilled or augered concrete piles, a minimum of four longitudinal bars with a minimum longitudinal reinforcement ratio of 0.005 and transverse confinement reinforcement in accordance with Sections 21.6.4.2 through 21.6.4.4 of ACI 318 shall be provided throughout the minimum reinforced length of the pile as defined below starting at the top of the pile. The longitudinal reinforcement shall extend beyond the minimum reinforced length of the pile by the tension development length.

The minimum reinforced length of the pile shall be taken as the greater of

1. One-half of the pile length.
2. A distance of 10 ft (3 m).
3. Three times the pile diameter.
4. The flexural length of the pile, which shall be taken as the length from the bottom of the pile cap to a point where the concrete section cracking moment multiplied by a resistance factor 0.4 exceeds the required factored moment at that point.

In addition, for piles located in Site Classes E or F, longitudinal reinforcement and transverse confinement reinforcement, as described above, shall extend the full length of the pile.

Where transverse reinforcing is required, transverse reinforcing ties shall be a minimum of No. 3 bars for up to 20-in.-diameter (500 mm) piles and No. 4 bars for piles of larger diameter.

In Site Classes A through D, longitudinal reinforcement and transverse confinement reinforcement, as defined above, shall also extend a minimum of seven times the pile diameter above and below the interfaces of soft to medium stiff clay or liquefiable strata except that transverse reinforcing not located within the minimum reinforced length shall be permitted to use a transverse spiral reinforcement ratio of not less than one-half of that required in Section 21.6.4.4(a) of ACI 318. Spacing of transverse reinforcing not located within the minimum reinforced length is permitted to be increased, but shall not exceed the least of the following:

1. 12 longitudinal bar diameters.
2. One-half the pile diameter.
3. 12 in. (300 mm).

14.2.3.2.4 Reinforcement for Metal-Cased Concrete Piles (SDC D through F) Reinforcement requirements are the same as for uncased concrete piles.

EXCEPTION: Spiral-welded metal casing of a thickness not less than No. 14 gauge can be considered as providing concrete confinement equivalent to the closed ties or equivalent spirals required in an uncased concrete pile, provided that the metal casing is adequately protected against possible deleterious action due to soil constituents, changing water levels, or other factors indicated by boring records of site conditions.

14.2.3.2.5 Reinforcement for Precast Concrete Piles (SDC D through F) Transverse confinement reinforcement consisting of closed ties or equivalent spirals shall be provided in accordance with Sections 21.6.4.2 through 21.6.4.4 of ACI 318 for the full length of the pile.

EXCEPTION: In other than Site Classes E or F, the specified transverse confinement reinforcement shall be provided within three pile diameters below the bottom of the pile cap, but it is permitted to use a transverse reinforcing ratio of not less than one-half of that required in Section 21.6.4.4(a) of ACI 318 throughout the remainder of the pile length.

14.2.3.2.6 Reinforcement for Precast Prestressed Piles (SDC D through F) In addition to the requirements for Seismic Design Category C, the following requirements shall be met:

1. Requirements of ACI 318, Chapter 21, need not apply.
2. Where the total pile length in the soil is 35 ft (10,668 mm) or less, the ductile pile region shall be taken as the entire length of the pile. Where the pile length exceeds 35 ft (10,668 mm), the ductile pile region shall be taken as the greater of 35 ft (10,668 mm) or the distance from the underside of the pile cap to the point of zero curvature plus three times the least pile dimension.
3. In the ductile pile region, the center to center spacing of the spirals or hoop reinforcement shall not exceed one-fifth of the least pile dimension, six times the diameter of the longitudinal strand, or 8 in. (203 mm), whichever is smaller.
4. Spiral reinforcement shall be spliced by lapping one full turn, by welding, or by the use of a mechanical connector. Where spiral reinforcement is lap spliced, the ends of the spiral shall terminate in a seismic hook in accordance with ACI 318, except that the bend shall be not less than 135°. Welded splices and mechanical connectors shall comply with Section 12.14.3 of ACI 318.
5. Where the transverse reinforcement consists of spirals or circular hoops, the volumetric ratio of spiral transverse reinforcement in the ductile pile region shall comply with

$$\rho_s = 0.25\left(\frac{f'_c}{f_{yh}}\right)\left(\frac{A_g}{A_{ch}} - 1.0\right)\left[0.5 + \frac{1.4P}{f'_c A_g}\right]$$

but not less than

$$\rho_s = 0.12\left(\frac{f'_c}{f_{yh}}\right)\left[0.5 + \frac{1.4P}{f'_c A_g}\right]$$

and ρ_s need not exceed 0.021 where

ρ_s = volumetric ratio (vol. of spiral/vol. of core)
$f'_c \le 6{,}000$ psi (41.4 MPa)
f_{yh} = yield strength of spiral reinforcement ≤ 85 ksi (586 MPa)
A_g = pile cross-sectional area, in.^{2} (mm^2)
A_{ch} = core area defined by spiral outside diameter, in.^{2} (mm^2)
P = axial load on pile resulting from the load combination $1.2D + 0.5L + 1.0E$, lb (kN)

This required amount of spiral reinforcement is permitted to be obtained by providing an inner and outer spiral.
6. Where transverse reinforcement consists of rectangular hoops and cross ties, the total cross-sectional area of lateral transverse reinforcement in the ductile region with spacing, s, and perpendicular to dimension, h_c, shall conform to

$$A_{sh} = 0.3sh_c\left(\frac{f'_c}{f_{yh}}\right)\left(\frac{A_g}{A_{ch}} - 1.0\right)\left[0.5 + \frac{1.4P}{f'_c A_g}\right]$$

but not less than

$$A_{sh} = 0.12sh_c\left(\frac{f'_c}{f_{yh}}\right)\left[0.5 + \frac{1.4P}{f'_c A_g}\right]$$

where

s = spacing of transverse reinforcement measured along length of pile, in. (mm)
h_c = cross-sectional dimension of pile core measured center to center of hoop reinforcement, in. (mm)
$f_{yh} \le 70$ ksi (483 MPa)

The hoops and cross ties shall be equivalent to deformed bars not less than No. 3 in size. Rectangular hoop ends shall terminate at a corner with seismic hooks.

7. Outside of the ductile pile region, the spiral or hoop reinforcement with a volumetric ratio not less than one-half of that required for transverse confinement reinforcement shall be provided.

14.3 COMPOSITE STEEL AND CONCRETE STRUCTURES

Structures, including foundations, constructed of composite steel and concrete to resist seismic loads shall be designed and detailed in accordance with this standard, including the reference documents and additional requirements provided in this section.

14.3.1 Reference Documents

The design, construction, and quality of composite steel and concrete members that resist seismic forces shall conform to the applicable requirements of the following:

1. AISC 341
2. AISC 360
3. ACI 318, excluding Chapter 22

14.3.2 General

Systems of structural steel acting compositely with reinforced concrete shall be designed in accordance with AISC 360 and ACI 318, excluding Chapter 22. Where required, the seismic design of composite steel and concrete systems shall be in accordance with the additional provisions of Section 14.3.3.

14.3.3 Seismic Requirements for Composite Steel and Concrete Structures

Where a response modification coefficient, *R*, in accordance with Table 12.2-1 is used for the design of systems of structural steel acting compositely with reinforced concrete, the structures shall be designed and detailed in accordance with the requirements of AISC 341.

14.3.4 Metal-Cased Concrete Piles

Metal-cased concrete piles shall be designed and detailed in accordance with Section 14.2.3.2.4.

14.4 MASONRY

Structures, including foundations, constructed of masonry to resist seismic loads shall be designed and detailed in accordance with this standard, including the references and additional requirements provided in this section.

14.4.1 Reference Documents

The design, construction, and quality assurance of masonry members that resist seismic forces shall conform to the requirements of TMS 402/ACI 530/ASCE 5 and TMS 602/ACI 530.1/ASCE 6, except as modified by Section 14.4.

14.4.2 *R* factors

To qualify for the response modification coefficients, *R,* set forth in this standard, the requirements of TMS 402/ACI 530/ASCE 5 and TMS 602/ACI 530.1/ASCE 6, as amended in subsequent sections, shall be satisfied.

Intermediate and special reinforced masonry shear walls designed in accordance with Section 2.3 of TMS 402/ACI 530/ASCE 5 shall also comply with the additional requirements contained in Section 14.4.4.

14.4.3 Modifications to Chapter 1 of TMS 402/ACI 530/ASCE 5

14.4.3.1 Separation Joints

Add the following new Section 1.19.3 to TMS 402/ACI 530/ASCE 5:

1.19.3 Separation Joints. *Where concrete abuts structural masonry and the joint between the materials is not designed as a separation joint, the concrete shall be roughened so that the average height of aggregate exposure is 1/8 in. (3 mm) and shall be bonded to the masonry in accordance with these requirements as if it were masonry. Vertical joints not intended to act as separation joints shall be crossed by horizontal reinforcement as required by Section 1.9.4.2.*

14.4.4 Modifications to Chapter 2 of TMS 402/ACI 530/ASCE 5

14.4.4.1 Stress Increase

If the increase in stress given in Section 2.1.2.3 of TMS 402/ACI 530/ASCE 5 is used, the restriction on load reduction in Section 2.4.1 of this standard shall be observed.

14.4.4.2 Reinforcement Requirements and Details

14.4.4.2.1 Reinforcing Bar Size Limitations Reinforcing bars used in masonry shall not be larger than No. 9 (M#29). The nominal bar diameter shall not exceed one-eighth of the nominal member thickness and shall not exceed one-quarter of the least clear dimension of the cell, course, or collar joint in which it is placed. The area of reinforcing bars placed in a cell or in a course of hollow unit construction shall not exceed 4 percent of the cell area.

14.4.4.2.2 Splices Lap splices shall not be used in plastic hinge zones of special reinforced masonry shear walls. The length of the plastic hinge zone shall be taken as at least 0.15 times the distance between the point of zero moment and the point of maximum moment. Reinforcement splices shall comply with TMS 402/ACI 530/ASCE 5 except paragraphs 2.1.9.7.2 and 2.1.9.7.3 shall be modified as follows:

2.1.9.7.2 Welded Splices: A welded splice shall be capable of developing in tension at least 125 percent of the specified yield strength, f_y*, of the bar. Welded splices shall only be permitted for ASTM A706 steel reinforcement. Welded splices shall not be permitted in plastic hinge zones of intermediate or special reinforced walls of masonry.*

2.1.9.7.3 Mechanical Connections: Mechanical splices shall be classified as Type 1 or Type 2 according to Section 21.1.6.1 of ACI 318. Type 1 mechanical splices shall not be used within a plastic hinge zone or within a beam-wall joint of intermediate or special reinforced masonry shear wall system. Type 2 mechanical splices shall be permitted in any location within a member.

14.4.5 Modifications to Chapter 3 of TMS 402/ACI 530/ASCE 5

14.4.5.1 Anchoring to Masonry

Add the following as the first paragraph in Section 3.1.6 to TMS 402/ACI 530/ASCE 5:

3.1.6 Anchor Bolts Embedded in Grout. *Anchorage assemblies connecting masonry elements that are part of the seismic force-resisting system to diaphragms and chords shall be designed so that the strength of the anchor is governed by steel tensile or shear yielding. Alternatively, the anchorage assembly is permitted to be designed so that it is governed by masonry breakout or anchor pullout provided that the anchorage assembly is designed to resist not less than 2.5 times the factored forces transmitted by the assembly.*

14.4.5.2 Splices in Reinforcement

Replace Sections 3.3.3.4(b) and 3.3.3.4(c) of TMS 402/ACI 530/ASCE 5 with the following:

(b) A welded splice shall be capable of developing in tension at least 125 percent of the specified yield strength, f_y*, of the bar. Welded splices shall only be permitted for ASTM A706 steel reinforcement. Welded splices shall not be permitted in plastic hinge zones of intermediate or special reinforced walls of masonry.*

(c) Mechanical splices shall be classified as Type 1 or Type 2 according to Section 21.1.6.1 of ACI 318. Type 1 mechanical splices shall not be used within a plastic hinge zone or within a beam-column joint of intermediate or special reinforced masonry shear walls. Type 2 mechanical splices are permitted in any location within a member.

Add the following new Section 3.3.3.4.1 to TMS 402/ACI 530/ASCE 5:

3.3.3.4.1 *Lap splices shall not be used in plastic hinge zones of special reinforced masonry shear walls. The length of the plastic hinge zone shall be taken as at least 0.15 times the distance between the point of zero moment and the point of maximum moment.*

14.4.5.3 Coupling Beams

Add the following new Section 3.3.4.2.6 to TMS 402/ACI 530/ASCE 5:

3.3.4.2.6 Coupling Beams. *Structural members that provide coupling between shear walls shall be designed to reach their moment or shear nominal strength before either shear wall reaches its moment or shear nominal strength. Analysis of coupled shear walls shall comply with accepted principles of mechanics.*

The design shear strength, ϕV_n*, of the coupling beams shall satisfy the following criterion:*

$$\phi V_n \geq \frac{1.25(M_1 + M_2)}{L_c} + 1.4V_g$$

where

M_1 *and* M_2 = *nominal moment strength at the ends of the beam*

L_c = *length of the beam between the shear walls*

V_g = *unfactored shear force due to gravity loads*

The calculation of the nominal flexural moment shall include the reinforcement in reinforced concrete roof and floor systems. The width of the reinforced concrete used for calculations of reinforcement shall be six times the floor or roof slab thickness.

14.4.5.4 Deep Flexural Members

Add the following new Section 3.3.4.2.7 to TMS 402/ACI 530/ASCE 5:

3.3.4.2.7 Deep Flexural Member Detailing. *Flexural members with overall-depth-to-clear-span ratio greater than 2/5 for continuous spans or 4/5 for simple spans shall be detailed in accordance with this section.*

3.3.4.2.7.1 *Minimum flexural tension reinforcement shall conform to Section 3.3.4.3.2.*

3.3.4.2.7.2 *Uniformly distributed horizontal and vertical reinforcement shall be provided throughout the length and depth of deep flexural members such that the reinforcement ratios in both directions are at least 0.001. Distributed flexural reinforcement is to be included in the determination of the actual reinforcement ratios.*

14.4.5.5 Walls with Factored Axial Stress Greater Than 0.05 f'_m

Add the following exception following the second paragraph of Section 3.3.5.3 of TMS 402/ACI 530/ASCE 5:

EXCEPTION: *A nominal thickness of 4 in. (102 mm) is permitted where load-bearing reinforced hollow clay unit masonry walls satisfy all of the following conditions.*

1. *The maximum unsupported height-to-thickness or length-to-thickness ratios do not exceed 27.*
2. *The net area unit strength exceeds 8,000 psi (55 MPa).*
3. *Units are laid in running bond.*
4. *Bar sizes do not exceed No. 4 (13 mm).*
5. *There are no more than two bars or one splice in a cell.*
6. *Joints are not raked.*

14.4.5.6 Shear Keys

Add the following new Section 3.3.6.6 to TMS 402/ACI 530/ASCE 5:

3.3.6.11 Shear Keys. *The surface of concrete upon which a special reinforced masonry shear wall is constructed shall have a minimum surface roughness of 1/8 in. (3 mm). Shear keys are required where the calculated tensile strain in vertical reinforcement from in-plane loads exceeds the yield strain under load combinations that include seismic forces based on an* R *factor equal to 1.5. Shear keys that satisfy the following requirements shall be placed at the interface between the wall and the foundation.*

1. *The width of the keys shall be at least equal to the width of the grout space.*
2. *The depth of the keys shall be at least 1.5 in. (38 mm).*
3. *The length of the key shall be at least 6 in. (152 mm).*
4. *The spacing between keys shall be at least equal to the length of the key.*
5. *The cumulative length of all keys at each end of the shear wall shall be at least 10 percent of the length of the shear wall (20 percent total).*
6. *At least 6 in. (150 mm) of a shear key shall be placed within 16 in. (406 mm) of each end of the wall.*
7. *Each key and the grout space above each key in the first course of masonry shall be grouted solid.*

14.4.6 Modifications to Chapter 6 of TMS 402/ACI 530/ASCE 5

14.4.6.1 Corrugated Sheet Metal Anchors

Add Section 6.2.2.10.1 to TMS 402/ACI 530/ASCE 5 as follows:

6.2.2.10.1 *Provide continuous single wire joint reinforcement of wire size W1.7 (MW11) at a maximum spacing of 18 in. (457 mm) on center vertically. Mechanically attach anchors to the joint reinforcement with clips or hooks. Corrugated sheet metal anchors shall not be used.*

14.4.7 Modifications to TMS 602/ACI 530.1/ASCE 6

14.4.7.1 Construction Procedures

Add the following new Article 3.5 I to TMS 602/ACI 530.1/ASCE 6:

3.5 I. *Construction procedures or admixtures shall be used to facilitate placement and control shrinkage of grout.*

14.5 WOOD

Structures, including foundations, constructed of wood to resist seismic loads shall be designed and detailed in accordance with this standard including the references and additional requirements provided in this section.

14.5.1 Reference Documents

The quality, testing, design, and construction of members and their fastenings in wood systems that resist seismic forces shall conform to the requirements of the applicable following reference documents,:

1. AF&PA NDS
2. AF&PA SDPWS

14.5.2 Framing

All wood columns and posts shall be framed to provide full end bearing. Alternatively, column and post end connections shall be designed to resist the full compressive loads, neglecting all end-bearing capacity. Continuity of wall top plates or provision for transfer of induced axial load forces shall be provided. Where offsets occur in the wall line, portions of the shear wall on each side of the offset shall be considered as separate shear walls unless provisions for force transfer around the offset are provided.

Chapter 15

SEISMIC DESIGN REQUIREMENTS FOR NONBUILDING STRUCTURES

15.1 GENERAL

15.1.1 Nonbuilding Structures

Nonbuilding structures include all self-supporting structures that carry gravity loads and that may be required to resist the effects of earthquake, with the exception of building structures specifically excluded in Section 11.1.2, and other nonbuilding structures where specific seismic provisions have yet to be developed, and therefore, are not set forth in Chapter 15. Nonbuilding structures supported by the earth or supported by other structures shall be designed and detailed to resist the minimum lateral forces specified in this section. Design shall conform to the applicable requirements of other sections as modified by this section. Foundation design shall comply with the requirements of Sections 12.1.5, 12.13, and Chapter 14.

15.1.2 Design

The design of nonbuilding structures shall provide sufficient stiffness, strength, and ductility consistent with the requirements specified herein for buildings to resist the effects of seismic ground motions as represented by these design forces:

a. Applicable strength and other design criteria shall be obtained from other portions of the seismic requirements of this standard or its reference documents.
b. Where applicable strength and other design criteria are not contained in, or referenced by the seismic requirements of this standard, such criteria shall be obtained from reference documents. Where reference documents define acceptance criteria in terms of allowable stresses as opposed to strength, the design seismic forces shall be obtained from this section and used in combination with other loads as specified in Section 2.4 of this standard and used directly with allowable stresses specified in the reference documents. Detailing shall be in accordance with the reference documents.

15.1.3 Structural Analysis Procedure Selection

Structural analysis procedures for nonbuilding structures that are similar to buildings shall be selected in accordance with Section 12.6. Nonbuilding structures that are not similar to buildings shall be designed using either the equivalent lateral force procedure in accordance with Section 12.8, the modal analysis procedure in accordance with Section 12.9, the linear response history analysis procedure in accordance with Section 16.1, the nonlinear response history analysis procedure in accordance with Section 16.2, or the procedure prescribed in the specific reference document.

15.2 REFERENCE DOCUMENTS

Reference documents referred to in Chapter 15 are listed in Chapter 23 and have seismic requirements based on the same force and displacement levels used in this standard or have seismic requirements that are specifically modified by Chapter 15.

15.3 NONBUILDING STRUCTURES SUPPORTED BY OTHER STRUCTURES

Where nonbuilding structures identified in Table 15.4-2 are supported by other structures, and the nonbuilding structures are not part of the primary seismic force-resisting system, one of the following methods shall be used.

15.3.1 Less Than 25 percent Combined Weight Condition

For the condition where the weight of the nonbuilding structure is less than 25 percent of the combined effective seismic weights of the nonbuilding structure and supporting structure, the design seismic forces of the nonbuilding structure shall be determined in accordance with Chapter 13 where the values of R_p and a_p shall be determined in accordance to Section 13.1.5. The supporting structure shall be designed in accordance with the requirements of Chapter 12 or Section 15.5 as appropriate with the weight of the nonbuilding structure considered in the determination of the effective seismic weight, W.

15.3.2 Greater Than or Equal to 25 Percent Combined Weight Condition

For the condition where the weight of the nonbuilding structure is equal to or greater than 25 percent of the combined effective seismic weights of the nonbuilding structure and supporting structure, an analysis combining the structural characteristics of both the nonbuilding structure and the supporting structures shall be performed to determine the seismic design forces as follows:

1. Where the fundamental period, T, of the nonbuilding structure is less than 0.06 s, the nonbuilding structure shall be considered a rigid element with appropriate distribution of its effective seismic weight. The supporting structure shall be designed in accordance with the requirements of Chapter 12 or Section 15.5 as appropriate, and the R value of the combined system is permitted to be taken as the R value of the supporting structural system. The nonbuilding structure and attachments shall be designed for the forces using the procedures of Chapter 13 where the value of R_p shall be taken as equal to the R value of the nonbuilding structure as set forth in Table 15.4-2, and a_p shall be taken as 1.0.
2. Where the fundamental period, T, of the nonbuilding structure is 0.06 s or greater, the nonbuilding structure and supporting structure shall be modeled together in a combined model with appropriate stiffness and effective seismic weight distributions. The combined structure shall be designed in accordance with Section 15.5 with the R value of the combined system taken as the lesser R value of the nonbuilding structure or the supporting structure. The nonbuilding structure and attachments shall be designed for the forces determined for the nonbuilding structure in the combined analysis.

15.3.3 Architectural, Mechanical, and Electrical Components

Architectural, mechanical, and electrical components supported by nonbuilding structures shall be designed in accordance with Chapter 13 of this standard.

15.4 STRUCTURAL DESIGN REQUIREMENTS

15.4.1 Design Basis

Nonbuilding structures having specific seismic design criteria established in reference documents shall be designed using the standards as amended herein. Where reference documents are not cited herein, nonbuilding structures shall be designed in compliance with Sections 15.5 and 15.6 to resist minimum seismic lateral forces that are not less than the requirements of Section 12.8 with the following additions and exceptions:

1. The seismic force-resisting system shall be selected as follows:
   a. For nonbuilding structures similar to buildings, a system shall be selected from among the types indicated in Table 12.2-1 or Table 15.4-1 subject to the system limitations and limits on structural height, h_n, based on the seismic design category indicated in the table. The appropriate values of R, Ω_0, and C_d indicated in the selected table shall be used in determining the base shear, element design forces, and design story drift as indicated in this standard. Design and detailing requirements shall comply with the sections referenced in the selected table.
   b. For nonbuilding structures not similar to buildings, a system shall be selected from among the types indicated in Table 15.4-2 subject to the system limitations and limits on structural height, h_n, based on seismic design category indicated in the table. The appropriate values of R, Ω_o, and C_d indicated in Table 15.4-2 shall be used in determining the base shear, element design forces, and design story drift as indicated in this standard. Design and detailing requirements shall comply with the sections referenced in Table 15.4-2.
   c. Where neither Table 15.4-1 nor Table 15.4-2 contains an appropriate entry, applicable strength and other design criteria shall be obtained from a reference document that is applicable to the specific type of nonbuilding structure. Design and detailing requirements shall comply with the reference document.
2. For nonbuilding systems that have an R value provided in Table 15.4-2, the minimum specified value in Eq. 12.8-5 shall be replaced by

$$C_s = 0.044 S_{DS} I_e \qquad (15.4\text{-}1)$$

The value of C_s shall not be taken as less than 0.03.

And for nonbuilding structures located where $S_1 \geq 0.6g$, the minimum specified value in Eq. 12.8-6 shall be replaced by

$$C_s = 0.8 S_1 / (R / I_e) \qquad (15.4\text{-}2)$$

EXCEPTION: Tanks and vessels that are designed to AWWA D100, AWWA D103, API

Table 15.4-1 Seismic Coefficients for Nonbuilding Structures Similar to Buildings

Nonbuilding Structure Type	Detailing Requirements	R	Ω_0	C_d	Structural System and Structural Height, h_n, Limits (ft)[a] B	C	D	E	F
Steel storage racks	15.5.3	4	2	3.5	NL	NL	NL	NL	NL
Building frame systems:									
Steel special concentrically braced frames	AISC 341	6	2	5	NL	NL	160	160	100
Steel ordinary concentrically braced frame	AISC 341	3¼	2	3¼	NL	NL	35[b]	35[b]	NP[b]
With permitted height increase	AISC 341	2½	2	2½	NL	NL	160	160	100
With unlimited height	AISC 360	1.5	1	1.5	NL	NL	NL	NL	NL
Moment-resisting frame systems:									
Steel special moment frames	AISC 341	8	3	5.5	NL	NL	NL	NL	NL
Special reinforced concrete moment frames	14.2.2.6 & ACI 318, including Chapter 21	8	3	5.5	NL	NL	NL	NL	NL
Steel intermediate moment frames	AISC 341	4.5	3	4	NL	NL	35[c,d]	NP[c,d]	NP[c,d]
With permitted height increase	AISC 341	2.5	2	2.5	NL	NL	160	160	100
With unlimited height	AISC 341	1.5	1	1.5	NL	NL	NL	NL	NL
Intermediate reinforced concrete moment frames	ACI 318, including Chapter 21	5	3	4.5	NL	NL	NP	NP	NP
With permitted height increase	ACI 318, including Chapter 21	3	2	2.5	NL	NL	50	50	50
With unlimited height	ACI 318, including Chapter 21	0.8	1	1	NL	NL	NL	NL	NL
Steel ordinary moment frames	AISC 341	3.5	3	3	NL	NL	NP[c,d]	NP[c,d]	NP[c,d]
With permitted height increase	AISC 341	2.5	2	2.5	NL	NL	100	100	NP[c,d]
With unlimited height	AISC 360	1	1	1	NL	NL	NL	NL	NL
Ordinary reinforced concrete moment frames	ACI 318, excluding Chapter 21	3	3	2.5	NL	NP	NP	NP	NP
With permitted height increase	ACI 318, excluding Chapter 21	0.8	1	1	NL	NL	50	50	50

[a]NL = no limit and NP = not permitted.
[b]Steel ordinary braced frames are permitted in pipe racks up to 65 ft (20 m).
[c]Steel ordinary moment frames and intermediate moment frames are permitted in pipe racks up to a height of 65 ft (20 m) where the moment joints of field connections are constructed of bolted end plates.
[d]Steel ordinary moment frames and intermediate moment frames are permitted in pipe racks up to a height of 35 ft (11 m).

Table 15.4-2 Seismic Coefficients for Nonbuilding Structures not Similar to Buildings

Nonbuilding Structure Type	Detailing Requirements[c]	R	Ω_0	C_d	Structural Height, h_n, Limits (ft)[ad]				
					B	C	D	E	F
Elevated tanks, vessels, bins or hoppers									
On symmetrically braced legs (not similar to buildings)	15.7.10	3	2[b]	2.5	NL	NL	160	100	100
On unbraced legs or asymmetrically braced legs (not similar buildings)	15.7.10	2	2[b]	2.5	NL	NL	100	60	60
Horizontal, saddle supported welded steel vessels	15.7.14	3	2[b]	2.5	NL	NL	NL	NL	NL
Tanks or vessels supported on structural towers similar to buildings	15.5.5	Use values for the appropriate structure type in the categories for building frame systems and moment resisting frame systems listed in Table 12.2-1 or Table 15.4-1.							
Flat-bottom ground-supported tanks:	15.7								
Steel or fiber-reinforced plastic:									
Mechanically anchored		3	2[b]	2.5	NL	NL	NL	NL	NL
Self-anchored		2.5	2[b]	2	NL	NL	NL	NL	NL
Reinforced or prestressed concrete:									
Reinforced nonsliding base		2	2[b]	2	NL	NL	NL	NL	NL
Anchored flexible base		3.25	2[b]	2	NL	NL	NL	NL	NL
Unanchored and unconstrained flexible base		1.5	1.5[b]	1.5	NL	NL	NL	NL	NL
All other		1.5	1.5[b]	1.5	NL	NL	NL	NL	NL
Cast-in-place concrete silos having walls continuous to the foundation	15.6.2	3	1.75	3	NL	NL	NL	NL	NL
All other reinforced masonry structures not similar to buildings detailed as intermediate reinforced masonry shear walls	14.4.1[f]	3	2	2.5	NL	NL	50	50	50
All other reinforced masonry structures not similar to buildings detailed as ordinary reinforced masonry shear walls	14.4.1	2	2.5	1.75	NL	160	NP	NP	NP
All other nonreinforced masonry structures not similar to buildings	14.4.1	1.25	2	1.5	NL	NL	NP	NP	NP
Concrete chimneys and stacks	15.6.2 and ACI 307	2	1.5	2.0	NL	NL	NL	NL	NL

Table 15.4-2 (*Continued*)

Nonbuilding Structure Type	Detailing Requirements[c]	R	Ω_0	C_d	Structural Height, h_n, Limits (ft)[ad] B	C	D	E	F
All steel and reinforced concrete distributed mass cantilever structures not otherwise covered herein including stacks, chimneys, silos, skirt-supported vertical vessels and single pedestal or skirt supported	15.6.2								
Welded steel	15.7.10	2	2^b	2	NL	NL	NL	NL	NL
Welded steel with special detailing[e]	15.7.10 & 15.7.10.5 a and b	3	2^b	2	NL	NL	NL	NL	NL
Prestressed or reinforced concrete	15.7.10	2	2^b	2	NL	NL	NL	NL	NL
Prestressed or reinforced concrete with special detailing	15.7.10 and ACI 318 Chapter 21, Sections 21.2 and 21.7	3	2^b	2	NL	NL	NL	NL	NL
Trussed towers (freestanding or guyed), guyed stacks, and chimneys	15.6.2	3	2	2.5	NL	NL	NL	NL	NL
Cooling towers									
Concrete or steel		3.5	1.75	3	NL	NL	NL	NL	NL
Wood frames		3.5	3	3	NL	NL	NL	50	50
Telecommunication towers	15.6.6								
Truss: Steel		3	1.5	3	NL	NL	NL	NL	NL
Pole: Steel		1.5	1.5	1.5	NL	NL	NL	NL	NL
Wood		1.5	1.5	1.5	NL	NL	NL	NL	NL
Concrete		1.5	1.5	1.5	NL	NL	NL	NL	NL
Frame: Steel		3	1.5	1.5	NL	NL	NL	NL	NL
Wood		1.5	1.5	1.5	NL	NL	NL	NL	NL
Concrete		2	1.5	1.5	NL	NL	NL	NL	NL
Amusement structures and monuments	15.6.3	2	2	2	NL	NL	NL	NL	NL
Inverted pendulum type structures (except elevated tanks, vessels, bins, and hoppers)	12.2.5.3	2	2	2	NL	NL	NL	NL	NL
Signs and billboards		3.0	1.75	3	NL	NL	NL	NL	NL
All other self-supporting structures, tanks, or vessels not covered above or by reference standards that are similar to buildings		1.25	2	2.5	NL	NL	50	50	50

[a]NL = no limit and NP = not permitted.
[b]See Section 15.7.3a for the application of the overstrength factors, Ω_0, for tanks and vessels.
[c]If a section is not indicated in the Detailing Requirements column, no specific detailing requirements apply.
[d]For the purpose of height limit determination, the height of the structure shall be taken as the height to the top of the structural frame making up the primary seismic force-resisting system.
[e]Sections 15.7.10.5a and 15.7.10.5b shall be applied for any Risk Category.
[f]Detailed with an essentially complete vertical load carrying frame.

650 Appendix E, and API 620 Appendix L as modified by this standard, and stacks and chimneys that are designed to ACI 307 as modified by this standard, shall be subject to the larger of the minimum base shear value defined by the reference document or the value determined by replacing Eq. 12.8-5 with the following:

$$C_s = 0.044 S_{DS} I_e \tag{15.4-3}$$

The value of C_s shall not be taken as less than 0.01.

and for nonbuilding structures located where $S_1 \geq 0.6g$, Eq. 12.8-6 shall be replaced by

$$C_s = 0.5 S_1/(R/I_e) \tag{15.4-4}$$

Minimum base shear requirements need not apply to the convective (sloshing) component of liquid in tanks.

3. The importance factor, I_e, shall be as set forth in Section 15.4.1.1.
4. The vertical distribution of the lateral seismic forces in nonbuilding structures covered by this section shall be determined:
   a. Using the requirements of Section 12.8.3, or
   b. Using the procedures of Section 12.9, or
   c. In accordance with the reference document applicable to the specific nonbuilding structure.
5. For nonbuilding structural systems containing liquids, gases, and granular solids supported at the base as defined in Section 15.7.1, the minimum seismic design force shall not be less than that required by the reference document for the specific system.
6. Where a reference document provides a basis for the earthquake resistant design of a particular type of nonbuilding structure covered by Chapter 15, such a standard shall not be used unless the following limitations are met:
   a. The seismic ground accelerations, and seismic coefficients, shall be in conformance with the requirements of Section 11.4.
   b. The values for total lateral force and total base overturning moment used in design shall not be less than 80 percent of the base shear value and overturning moment, each adjusted for the effects of soil–structure interaction that is obtained using this standard.
7. The base shear is permitted to be reduced in accordance with Section 19.2.1 to account for the effects of soil–structure interaction. In no case shall the reduced base shear be less than $0.7V$.
8. Unless otherwise noted in Chapter 15, the effects on the nonbuilding structure due to gravity loads and seismic forces shall be combined in accordance with the factored load combinations as presented in Section 2.3.
9. Where specifically required by Chapter 15, the design seismic force on nonbuilding structures shall be as defined in Section 12.4.3.

15.4.1.1 Importance Factor

The importance factor, I_e, and risk category for nonbuilding structures are based on the relative hazard of the contents and the function. The value of I_e shall be the largest value determined by the following:

a. Applicable reference document listed in Chapter 23.
b. The largest value as selected from Table 1.5-2.
c. As specified elsewhere in Chapter 15.

15.4.2 Rigid Nonbuilding Structures

Nonbuilding structures that have a fundamental period, T, less than 0.06 s, including their anchorages, shall be designed for the lateral force obtained from the following:

$$V = 0.30 S_{DS} W I_e \tag{15.4-5}$$

where

V = the total design lateral seismic base shear force applied to a nonbuilding structure
S_{DS} = the site design response acceleration as determined from Section 11.4.4
W = nonbuilding structure operating weight
I_e = the importance factor determined in accordance with Section 15.4.1.1

The force shall be distributed with height in accordance with Section 12.8.3.

15.4.3 Loads

The seismic effective weight W for nonbuilding structures shall include the dead load and other loads as defined for structures in Section 12.7.2. For purposes of calculating design seismic forces in nonbuilding structures, W also shall include all normal operating contents for items such as tanks, vessels, bins, hoppers, and the contents of piping. W shall include snow and ice loads where these loads constitute 25 percent or more of W or where required by the authority having jurisdiction based on local environmental characteristics.

15.4.4 Fundamental Period

The fundamental period of the nonbuilding structure shall be determined using the structural properties and deformation characteristics of the resisting elements in a properly substantiated analysis as indicated in Section 12.8.2. Alternatively, the fundamental period T is permitted to be computed from the following equation:

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} f_i \delta_i^2}{g \sum_{i=1}^{n} f_i \delta_i}} \qquad (15.4\text{-}6)$$

The values of f_i represent any lateral force distribution in accordance with the principles of structural mechanics. The elastic deflections, δ_i, shall be calculated using the applied lateral forces, f_i. Equations 12.8-7, 12.8-8, 12.8-9, and 12.8-10 shall not be used for determining the period of a nonbuilding structure.

15.4.5 Drift Limitations

The drift limitations of Section 12.12.1 need not apply to nonbuilding structures if a rational analysis indicates they can be exceeded without adversely affecting structural stability or attached or interconnected components and elements such as walkways and piping. P-delta effects shall be considered where critical to the function or stability of the structure.

15.4.6 Materials Requirements

The requirements regarding specific materials in Chapter 14 shall be applicable unless specifically exempted in Chapter 15.

15.4.7 Deflection Limits and Structure Separation

Deflection limits and structure separation shall be determined in accordance with this standard unless specifically amended in Chapter 15.

15.4.8 Site-Specific Response Spectra

Where required by a reference document or the authority having jurisdiction, specific types of nonbuilding structures shall be designed for site-specific criteria that account for local seismicity and geology, expected recurrence intervals, and magnitudes of events from known seismic hazards (see Section 11.4.7 of this standard). If a longer recurrence interval is defined in the reference document for the nonbuilding structure, such as liquefied natural gas (LNG) tanks (NFPA 59A), the recurrence interval required in the reference document shall be used.

15.4.9 Anchors in Concrete or Masonry

15.4.9.1 Anchors in Concrete

Anchors in concrete used for nonbuilding structure anchorage shall be designed in accordance with Appendix D of ACI 318.

15.4.9.2 Anchors in Masonry

Anchors in masonry used for nonbuilding structure anchorage shall be designed in accordance with TMS402/ACI 530/ASCE 6. Anchors shall be designed to be governed by the tensile or shear strength of a ductile steel element.

EXCEPTION: Anchors shall be permitted to be designed so that the attachment 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 attachment.

15.4.9.3 Post-Installed Anchors in Concrete and Masonry

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

15.5 NONBUILDING STRUCTURES SIMILAR TO BUILDINGS

15.5.1 General

Nonbuilding structures similar to buildings as defined in Section 11.2 shall be designed in accordance with this standard as modified by this section and the specific reference documents. This general category of nonbuilding structures shall be designed in accordance with the seismic requirements of this standard and the applicable portions of Section 15.4. The combination of load effects, E, shall be determined in accordance with Section 12.4.

15.5.2 Pipe Racks

15.5.2.1 Design Basis

In addition to the requirements of Section 15.5.1, pipe racks supported at the base of the structure shall be designed to meet the force requirements of Section 12.8 or 12.9. Displacements of the pipe rack and potential for interaction effects (pounding of the piping system) shall be considered using the amplified deflections obtained from the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \tag{15.5-1}$$

where

C_d = deflection amplification factor in Table 15.4-1
δ_{xe} = deflections determined using the prescribed seismic design forces of this standard
I_e = importance factor determined in accordance with Section 15.4.1.1

See Section 13.6.3 for the design of piping systems and their attachments. Friction resulting from gravity loads shall not be considered to provide resistance to seismic forces.

15.5.3 Steel Storage Racks

Steel storage racks supported at or below grade shall be designed in accordance with ANSI/RMI MH 16.1 and its force and displacement requirements, except as follows:

15.5.3.1

Modify Section 2.6.2 of ANSI/RMI MH 16.1 as follows:

2.6.2 Minimum Seismic Forces

The storage rack shall be designed…

Above-Grade Elevation: *Storage rack installed at elevations above grade shall be designed, fabricated, and installed in accordance with the following requirements:*

Storage racks shall meet the force and displacement requirements required of nonbuilding structures supported by other structures, including the force and displacement effects caused by amplifications of upper-story motions. In no case shall the value of V *be taken as less than the value of* F_p *determined in accordance with Section 13.3.1 of ASCE/SEI 7, where* R_p *is taken equal to* R*, and* a_p *is taken equal to 2.5*.

15.5.3.2

Modify Section 7.2.2 of ANSI/RMI MH 16.1 as follows:

7.2.2 Base Plate Design

Once the required bearing area has been determined from the allowable bearing stress F'_p *the minimum thickness of the base plate is determined by rational analysis or by appropriate test using a test load 1.5 times the ASD design load or the factored LRFD load. Design forces that include seismic loads for anchorage of steel storage racks to concrete or masonry shall be determined using load combinations with overstrength provided in Section 12.4.3.2 of ASCE/SEI 7. The overstrength factor shall be taken as 2.0.*

Anchorage of steel storage racks to concrete shall be in accordance with the requirements of Section 15.4.9 of ASCE/SEI 7. Upon request, information shall be given to the owner or the owner's agent on the location, size, and pressures under the column base plates of each type of upright frame in the installation. When rational analysis is used to determine base plate thickness and other applicable standards do not apply, the base plate shall be permitted to be designed for the following loading conditions, where applicable: (balance of section unchanged)

15.5.3.3

Modify Section 7.2.4 of ANSI/RMI MH 16.1 as follows:

7.2.4 Shims

Shims may be used under the base plate to maintain the plumbness of the storage rack. The shims shall be made of a material that meets or exceeds the design bearing strength (LRFD) or allowable bearing strength (ASD) of the floor. The shim size and location under the base plate shall be equal to or greater than the required base plate size and location.

~~*In no case shall the total thickness of any set of shims under a base plate exceed six times the diameter of the largest anchor bolt used in that base.*~~

Shims ~~*that are a total thickness of less than or equal to six times the anchor bolt diameter under bases with less than two anchor bolts*~~ *shall be interlocked or welded together in a fashion that is capable of transferring all the shear forces at the base.*

~~*Shims that are a total thickness of less than or equal to two times the anchor bolt diameter need not be interlocked or welded together*~~.

Bending in the anchor associated with shims or grout under the base plate shall be taken into account in the design of the anchor bolts.

15.5.3.4 Alternative

As an alternative to ANSI MH 16.1 as modified above, steel storage racks shall be permitted to be designed in accordance with the requirements of

Sections 15.1, 15.2, 15.3, 15.5.1, and 15.5.3.5 through 15.5.3.8 of this standard.

15.5.3.5 General Requirements

Steel storage racks shall satisfy the force requirements of this section.

EXCEPTION: Steel storage racks supported at the base are permitted to be designed as structures with an R of 4, provided that the seismic requirements of this standard are met. Higher values of R are permitted to be used where the detailing requirements of reference documents listed in Section 14.1.1 are met. The importance factor, I_e, for storage racks in structures open to the public, such as warehouse retail stores, shall be taken equal to 1.5.

15.5.3.6 Operating Weight

Steel storage racks shall be designed for each of the following conditions of operating weight, W or W_p.

a. Weight of the rack plus every storage level loaded to 67 percent of its rated load capacity.
b. Weight of the rack plus the highest storage level only loaded to 100 percent of its rated load capacity.

The design shall consider the actual height of the center of mass of each storage load component.

15.5.3.7 Vertical Distribution of Seismic Forces

For all steel storage racks, the vertical distribution of seismic forces shall be as specified in Section 12.8.3 and in accordance with the following:

a. The base shear, V, of the typical structure shall be the base shear of the steel storage rack where loaded in accordance with Section 15.5.3.6.
b. The base of the structure shall be the floor supporting the steel storage rack. Each steel storage level of the rack shall be treated as a level of the structure with heights h_i and h_x measured from the base of the structure.
c. The factor k is permitted to be taken as 1.0.

15.5.3.8 Seismic Displacements

Steel storage rack installations shall accommodate the seismic displacement of the storage racks and their contents relative to all adjacent or attached components and elements. The assumed total relative displacement for storage racks shall be not less than 5 percent of the structural height above the base, h_n, unless a smaller value is justified by test data or analysis in accordance with Section 11.1.4.

15.5.4 Electrical Power Generating Facilities

15.5.4.1 General

Electrical power generating facilities are power plants that generate electricity by steam turbines, combustion turbines, diesel generators, or similar turbo machinery.

15.5.4.2 Design Basis

In addition to the requirements of Section 15.5.1, electrical power generating facilities shall be designed using this standard and the appropriate factors contained in Section 15.4.

15.5.5 Structural Towers for Tanks and Vessels

15.5.5.1 General

In addition to the requirements of Section 15.5.1, structural towers that support tanks and vessels shall be designed to meet the requirements of Section 15.3. In addition, the following special considerations shall be included:

a. The distribution of the lateral base shear from the tank or vessel onto the supporting structure shall consider the relative stiffness of the tank and resisting structural elements.
b. The distribution of the vertical reactions from the tank or vessel onto the supporting structure shall consider the relative stiffness of the tank and resisting structural elements. Where the tank or vessel is supported on grillage beams, the calculated vertical reaction due to weight and overturning shall be increased at least 20 percent to account for nonuniform support. The grillage beam and vessel attachment shall be designed for this increased design value.
c. Seismic displacements of the tank and vessel shall consider the deformation of the support structure where determining P-delta effects or evaluating required clearances to prevent pounding of the tank on the structure.

15.5.6 Piers and Wharves

15.5.6.1 General

Piers and wharves are structures located in waterfront areas that project into a body of water or that parallel the shoreline.

15.5.6.2 Design Basis

In addition to the requirements of Section 15.5.1, piers and wharves that are accessible to the general public, such as cruise ship terminals and piers with retail or commercial offices or restaurants, shall be designed to comply with this standard. Piers and wharves that are not accessible to the general public are beyond the scope of this section.

The design shall account for the effects of liquefaction and soil failure collapse mechanisms, as well as consider all applicable marine loading combinations, such as mooring, berthing, wave, and current on piers and wharves as required. Structural detailing shall consider the effects of the marine environment.

15.6 GENERAL REQUIREMENTS FOR NONBUILDING STRUCTURES NOT SIMILAR TO BUILDINGS

Nonbuilding structures that do not have lateral and vertical seismic force-resisting systems that are similar to buildings shall be designed in accordance with this standard as modified by this section and the specific reference documents. Loads and load distributions shall not be less demanding than those determined in this standard. The combination of earthquake load effects, E, shall be determined in accordance with Section 12.4.2.

EXCEPTION: The redundancy factor, ρ, per Section 12.3.4 shall be taken as 1.

15.6.1 Earth-Retaining Structures

This section applies to all earth-retaining structures assigned to Seismic Design Category D, E, or F. The lateral earth pressures due to earthquake ground motions shall be determined in accordance with Section 11.8.3.

The risk category shall be determined by the proximity of the earth-retaining structure to other buildings and structures. If failure of the earth-retaining structure would affect the adjacent building or structure, the risk category shall not be less than that of the adjacent building or structure. Earth-retaining walls are permitted to be designed for seismic loads as either yielding or nonyielding walls. Cantilevered reinforced concrete or masonry retaining walls shall be assumed to be yielding walls and shall be designed as simple flexural wall elements.

15.6.2 Stacks and Chimneys

Stacks and chimneys are permitted to be either lined or unlined and shall be constructed from concrete, steel, or masonry. Steel stacks, concrete stacks, steel chimneys, concrete chimneys, and liners shall be designed to resist seismic lateral forces determined from a substantiated analysis using reference documents. Interaction of the stack or chimney with the liners shall be considered. A minimum separation shall be provided between the liner and chimney equal to C_d times the calculated differential lateral drift.

Concrete chimneys and stacks shall be designed in accordance with the requirements of ACI 307 except that (1) the design base shear shall be determined based on Section 15.4.1 of this standard; (2) the seismic coefficients shall be based on the values provided in Table 15.4-2, and (3) openings shall be detailed as required below. When modal response spectrum analysis is used for design, the procedures of Section 12.9 shall be permitted to be used.

For concrete chimneys and stacks assigned to SDC D, E, and F, splices for vertical rebar shall be staggered such that no more than 50% of the bars are spliced at any section and alternate lap splices are staggered by the development length. In addition, where the loss of cross-sectional area is greater than 10%, cross sections in the regions of breachings/openings shall be designed and detailed for vertical force, shear force, and bending moment demands along the vertical direction, determined for the affected cross section using an overstrength factor of 1.5. The region where the overstrength factor applies shall extend above and below the opening(s) by a distance equal to half of the width of the largest opening in the affected region. Appropriate reinforcement development lengths shall be provided beyond the required region of overstrength. The jamb regions around each opening shall be detailed using the column tie requirements in Section 7.10.5 of ACI 318. Such detailing shall extend for a jamb width of a minimum of two times the wall thickness and for a height of the opening height plus twice the wall thickness above and below the opening, but no less than the development length of the longitudinal bars. Where the existence of a footing or base mat precludes the ability to achieve the extension distance below the opening and within the stack, the jamb reinforcing shall be extended and developed into the footing or base mat. The percentage of longitudinal reinforcement in jamb regions shall meet the requirements of Section 10.9 of ACI 318 for compression members.

15.6.3 Amusement Structures

Amusement structures are permanently fixed structures constructed primarily for the conveyance and entertainment of people. Amusement structures shall be designed to resist seismic lateral forces determined from a substantiated analysis using reference documents.

15.6.4 Special Hydraulic Structures

Special hydraulic structures are structures that are contained inside liquid-containing structures. These structures are exposed to liquids on both wall surfaces at the same head elevation under normal operating conditions. Special hydraulic structures are subjected to out-of-plane forces only during an earthquake where the structure is subjected to differential hydrodynamic fluid forces. Examples of special hydraulic structures include separation walls, baffle walls, weirs, and other similar structures.

15.6.4.1 Design Basis

Special hydraulic structures shall be designed for out-of-phase movement of the fluid. Unbalanced forces from the motion of the liquid must be applied simultaneously "in front of" and "behind" these elements.

Structures subject to hydrodynamic pressures induced by earthquakes shall be designed for rigid body and sloshing liquid forces and their own inertia force. The height of sloshing shall be determined and compared to the freeboard height of the structure. Interior elements, such as baffles or roof supports, also shall be designed for the effects of unbalanced forces and sloshing.

15.6.5 Secondary Containment Systems

Secondary containment systems, such as impoundment dikes and walls, shall meet the requirements of the applicable standards for tanks and vessels and the authority having jurisdiction.

Secondary containment systems shall be designed to withstand the effects of the maximum considered earthquake ground motion where empty and two-thirds of the maximum considered earthquake ground motion where full including all hydrodynamic forces as determined in accordance with the procedures of Section 11.4. Where determined by the risk assessment required by Section 1.5.2 or by the authority having jurisdiction that the site may be subject to aftershocks of the same magnitude as the maximum considered motion, secondary containment systems shall be designed to withstand the effects of the maximum considered earthquake ground motion where full including all hydrodynamic forces as determined in accordance with the procedures of Section 11.4.

15.6.5.1 Freeboard

Sloshing of the liquid within the secondary containment area shall be considered in determining the height of the impound. Where the primary containment has not been designed with a reduction in the structure category (i.e., no reduction in importance factor I_e) as permitted by Section 1.5.3, no freeboard provision is required. Where the primary containment has been designed for a reduced structure category (i.e., importance factor I_e reduced) as permitted by Section 1.5.3, a minimum freeboard, δ_s, shall be provided where

$$\delta_s = 0.42DS_{ac} \qquad (15.6\text{-}1)$$

where S_{ac} is the spectral acceleration of the convective component and is determined according to the procedures of Section 15.7.6.1 using 0.5 percent damping. For circular impoundment dikes, D shall be taken as the diameter of the impoundment dike. For rectangular impoundment dikes, D shall be taken as the plan dimension of the impoundment dike, L, for the direction under consideration.

15.6.6 Telecommunication Towers

Self-supporting and guyed telecommunication towers shall be designed to resist seismic lateral forces determined from a substantiated analysis using reference documents.

15.7 TANKS AND VESSELS

15.7.1 General

This section applies to all tanks, vessels, bins, and silos, and similar containers storing liquids, gases, and granular solids supported at the base (hereafter referred to generically as "tanks and vessels"). Tanks and vessels covered herein include reinforced concrete, prestressed concrete, steel, aluminum, and fiber-reinforced plastic materials. Tanks supported on elevated levels in buildings shall be designed in accordance with Section 15.3.

15.7.2 Design Basis

Tanks and vessels storing liquids, gases, and granular solids shall be designed in accordance with this standard and shall be designed to meet the requirements of the applicable reference documents listed in Chapter 23. Resistance to seismic forces shall be determined from a substantiated analysis based on the applicable reference documents listed in Chapter 23.

a. Damping for the convective (sloshing) force component shall be taken as 0.5 percent.
b. Impulsive and convective components shall be combined by the direct sum or the square root of

the sum of the squares (SRSS) method where the modal periods are separated. If significant modal coupling may occur, the complete quadratic combination (CQC) method shall be used.

c. Vertical earthquake forces shall be considered in accordance with the applicable reference document. If the reference document permits the user the option of including or excluding the vertical earthquake force to comply with this standard, it shall be included. For tanks and vessels not covered by a reference document, the forces due to the vertical acceleration shall be defined as follows:
   (1) Hydrodynamic vertical and lateral forces in tank walls: The increase in hydrostatic pressures due to the vertical excitation of the contained liquid shall correspond to an effective increase in unit weight, γ_L, of the stored liquid equal to $0.2S_{DS}\,\gamma_L$.
   (2) Hydrodynamic hoop forces in cylindrical tank walls: In a cylindrical tank wall, the hoop force per unit height, N_h, at height y from the base, associated with the vertical excitation of the contained liquid, shall be computed in accordance with Eq. 15.7-1.

$$N_h = 0.2S_{DS}\gamma_L\left(H_L - y\right)\left(\frac{D_i}{2}\right) \qquad (15.7\text{-}1)$$

where

D_i = inside tank diameter
H_L = liquid height inside the tank
y = distance from base of the tank to height being investigated
γ_L = unit weight of stored liquid

   (3) Vertical inertia forces in cylindrical and rectangular tank walls: Vertical inertia forces associated with the vertical acceleration of the structure itself shall be taken equal to $0.2S_{DS}W$.

15.7.3 Strength and Ductility

Structural members that are part of the seismic force-resisting system shall be designed to provide the following:

a. Connections to seismic force-resisting elements, excluding anchors (bolts or rods) embedded in concrete, shall be designed to develop Ω_0 times the calculated connection design force. For anchors (bolts or rods) embedded in concrete, the design of the anchor embedment shall meet the requirements of Section 15.7.5. Additionally, the connection of the anchors to the tank or vessel shall be designed to develop the lesser of the strength of the anchor in tension as determined by the reference document or Ω_0 times the calculated anchor design force. The overstrength requirements of Section 12.4.3, and the Ω_0 values tabulated in Table 15.4-2, do not apply to the design of walls, including interior walls, of tanks or vessels.
b. Penetrations, manholes, and openings in shell elements shall be designed to maintain the strength and stability of the shell to carry tensile and compressive membrane shell forces.
c. Support towers for tanks and vessels with irregular bracing, unbraced panels, asymmetric bracing, or concentrated masses shall be designed using the requirements of Section 12.3.2 for irregular structures. Support towers using chevron or eccentric braced framing shall comply with the seismic requirements of this standard. Support towers using tension-only bracing shall be designed such that the full cross-section of the tension element can yield during overload conditions.
d. In support towers for tanks and vessels, compression struts that resist the reaction forces from tension braces shall be designed to resist the lesser of the yield load of the brace, A_gF_y, or Ω_o times the calculated tension load in the brace.
e. The vessel stiffness relative to the support system (foundation, support tower, skirt, etc.) shall be considered in determining forces in the vessel, the resisting elements, and the connections.
f. For concrete liquid-containing structures, system ductility, and energy dissipation under unfactored loads shall not be allowed to be achieved by inelastic deformations to such a degree as to jeopardize the serviceability of the structure. Stiffness degradation and energy dissipation shall be allowed to be obtained either through limited microcracking, or by means of lateral force resistance mechanisms that dissipate energy without damaging the structure.

15.7.4 Flexibility of Piping Attachments

Design of piping systems connected to tanks and vessels shall consider the potential movement of the connection points during earthquakes and provide sufficient flexibility to avoid release of the product by failure of the piping system. The piping system and supports shall be designed so as not to impart significant mechanical loading on the attachment to the tank or vessel shell. Mechanical devices that add flexibility, such as bellows, expansion joints, and other flexible apparatus, are permitted to be used where

Table 15.7-1 Minimum Design Displacements for Piping Attachments

Condition	Displacements (in.)
Mechanically Anchored Tanks and Vessels	
Upward vertical displacement relative to support or foundation	1 (25.4 mm)
Downward vertical displacement relative to support or foundation	0.5 (12.7 mm)
Range of horizontal displacement (radial and tangential) relative to support or foundation	0.5 (12.7 mm)
Self-Anchored Tanks or Vessels (at grade)	
Upward vertical displacement relative to support or foundation	
If designed in accordance with a reference document as modified by this standard	
Anchorage ratio less than or equal to 0.785 (indicates no uplift)	1 (25.4 mm)
Anchorage ratio greater than 0.785 (indicates uplift)	4 (101.1 mm)
If designed for seismic loads in accordance with this standard but not covered by a reference document	
For tanks and vessels with a diameter less than 40 ft	8 (202.2 mm)
For tanks and vessels with a diameter equal to or greater than 40 ft	12 (0.305 m)
Downward vertical displacement relative to support or foundation	
For tanks with a ringwall/mat foundation	0.5 (12.7 mm)
For tanks with a berm foundation	1 (25.4 mm)
Range of horizontal displacement (radial and tangential) relative to support or foundation	2 (50.8mm)

they are designed for seismic displacements and defined operating pressure.

Unless otherwise calculated, the minimum displacements in Table 15.7-1 shall be assumed. For attachment points located above the support or foundation elevation, the displacements in Table 15.7-1 shall be increased to account for drift of the tank or vessel relative to the base of support. The piping system and tank connection shall also be designed to tolerate C_d times the displacements given in Table 15.7-1 without rupture, although permanent deformations and inelastic behavior in the piping supports and tank shell is permitted. For attachment points located above the support or foundation elevation, the displacements in Table 15.7-1 shall be increased to account for drift of the tank or vessel. The values given in Table 15.7-1 do not include the influence of relative movements of the foundation and piping anchorage points due to foundation movements (e.g., settlement, seismic displacements). The effects of the foundation movements shall be included in the piping system design including the determination of the mechanical loading on the tank or vessel, and the total displacement capacity of the mechanical devices intended to add flexibility.

The anchorage ratio, J, for self-anchored tanks shall comply with the criteria shown in Table 15.7-2 and is defined as

$$J = \frac{M_{rw}}{D^2 (w_t + w_a)} \quad (15.7\text{-}2)$$

Table 15.7-2 Anchorage Ratio

J Anchorage Ratio	Criteria
$J < 0.785$	No uplift under the design seismic overturning moment. The tank is self-anchored.
$0.785 < J < 1.54$	Tank is uplifting, but the tank is stable for the design load providing the shell compression requirements are satisfied. The tank is self-anchored.
$J > 1.54$	Tank is not stable and shall be mechanically anchored for the design load.

where

$$w_t = \frac{W_s}{\pi D} + w_r \quad (15.7\text{-}3)$$

w_r = roof load acting on the shell in pounds per foot (N/m) of shell circumference. Only permanent roof loads shall be included. Roof live load shall not be included

w_a = maximum weight of the tank contents that may be used to resist the shell overturning moment in pounds per foot (N/m) of shell circumference. Usually consists of an annulus of liquid limited by the bending strength of the tank bottom or annular plate

M_{rw} = the overturning moment applied at the bottom of the shell due to the seismic design loads in foot-pounds (N-m) (also known as the "ringwall moment")

D = tank diameter in feet

W_s = total weight of tank shell in pounds

15.7.5 Anchorage

Tanks and vessels at grade are permitted to be designed without anchorage where they meet the requirements for unanchored tanks in reference documents. Tanks and vessels supported above grade on structural towers or building structures shall be anchored to the supporting structure.

The following special detailing requirements shall apply to steel tank and vessel anchor bolts in SDC C, D, E, and F. Anchorage shall be in accordance with Section 15.4.9, whereby the anchor embedment into the concrete shall be designed to develop the steel strength of the anchor in tension. The steel strength of the anchor in tension shall be determined in accordance with ACI 318, Appendix D, Eq. D-3. The anchor shall have a minimum gauge length of eight diameters. Post-installed anchors are permitted to be used in accordance with Section 15.4.9.3 provided the anchor embedment into the concrete is designed to develop the steel strength of the anchor in tension. In either case, the load combinations with overstrength of Section 12.4.3 are not to be used to size the anchor bolts for tanks and horizontal and vertical vessels.

15.7.6 Ground-Supported Storage Tanks for Liquids

15.7.6.1 General

Ground-supported, flat bottom tanks storing liquids shall be designed to resist the seismic forces calculated using one of the following procedures:

a. The base shear and overturning moment calculated as if the tank and the entire contents are a rigid mass system per Section 15.4.2 of this standard.
b. Tanks or vessels storing liquids in Risk Category IV, or with a diameter greater than 20 ft (6.1 m), shall be designed to consider the hydrodynamic pressures of the liquid in determining the equivalent lateral forces and lateral force distribution per the applicable reference documents listed in Chapter 23 and the requirements of Section 15.7 of this standard.
c. The force and displacement requirements of Section 15.4 of this standard.

The design of tanks storing liquids shall consider the impulsive and convective (sloshing) effects and their consequences on the tank, foundation, and attached elements. The impulsive component corresponds to the high-frequency amplified response to the lateral ground motion of the tank roof, the shell, and the portion of the contents that moves in unison with the shell. The convective component corresponds to the low-frequency amplified response of the contents in the fundamental sloshing mode. Damping for the convective component shall be 0.5 percent for the sloshing liquid unless otherwise defined by the reference document. The following definitions shall apply:

D_i = inside diameter of tank or vessel

H_L = design liquid height inside the tank or vessel

L = inside length of a rectangular tank, parallel to the direction of the earthquake force being investigated

N_h = hydrodynamic hoop force per unit height in the wall of a cylindrical tank or vessel

T_c = natural period of the first (convective) mode of sloshing

T_i = fundamental period of the tank structure and impulsive component of the content

V_i = base shear due to impulsive component from weight of tank and contents

V_c = base shear due to the convective component of the effective sloshing mass

y = distance from base of the tank to level being investigated

γ_L = unit weight of stored liquid

The seismic base shear is the combination of the impulsive and convective components:

$$V = V_i + V_c \tag{15.7-4}$$

where

$$V_i = \frac{S_{ai} W_i}{\left(\frac{R}{I_e}\right)} \tag{15.7-5}$$

$$V_c = \frac{S_{ac} I_e}{1.5} W_c \tag{15.7-6}$$

S_{ai} = the spectral acceleration as a multiplier of gravity including the site impulsive components at period T_i and 5 percent damping

For $T_i \le T_s$

$$S_{ai} = S_{DS} \tag{15.7-7}$$

For $T_s < T_i \le T_L$

$$S_{ai} = \frac{S_{D1}}{T_i} \tag{15.7-8}$$

For $T_i > T_L$

$$S_{ai} = \frac{S_{D1}T_L}{T_i^2} \quad (15.7\text{-}9)$$

NOTES:

a. Where a reference document is used in which the spectral acceleration for the tank shell, and the impulsive component of the liquid is independent of T_i, then $S_{ai} = S_{DS}$.
b. Equations 15.7-8 and 15.7-9 shall not be less than the minimum values required in Section 15.4.1 Item 2 multiplied by R/I_e.
c. For tanks in Risk Category IV, the value of the importance factor, I_e, used for freeboard determination only shall be taken as 1.0.
d. For tanks in Risk Categories I, II, and III, the value of T_L used for freeboard determination is permitted to be set equal to 4 s. The value of the importance factor, I_e, used for freeboard determination for tanks in Risk Categories I, II, and III shall be the value determined from Table 1.5-1.
e. Impulsive and convective seismic forces for tanks are permitted to be combined using the square root of the sum of the squares (SRSS) method in lieu of the direct sum method shown in Section 15.7.6 and its related subsections.

S_{ac} = the spectral acceleration of the sloshing liquid (convective component) based on the sloshing period T_c and 0.5 percent damping

For $T_c \le T_L$:

$$S_{ac} = \frac{1.5S_{D1}}{T_c} \le 1.5S_{DS} \quad (15.7\text{-}10)$$

For $T_c > T_L$:

$$S_{ac} = \frac{1.5S_{D1}T_L}{T_c^2} \quad (15.7\text{-}11)$$

EXCEPTION: For $T_c > 4$ s, S_{ac} is permitted be determined by a site-specific study using one or more of the following methods: (i) the procedures found in Chapter 21, provided such procedures, which rely on ground-motion attenuation equations for computing response spectra, cover the natural period band containing T_c, (ii) ground-motion simulation methods employing seismological models of fault rupture and wave propagation, and (iii) analysis of representative strong-motion accelerogram data with reliable long-period content extending to periods greater than T_c. Site-specific values of S_{ac} shall be based on one standard deviation determinations. However, in no case shall the value of S_{ac} be taken as less than the value determined in accordance with Eq. 15.7-11 using 50% of the mapped value of T_L from Chapter 22.

The 80 percent limit on S_a required by Sections 21.3 and 21.4 shall not apply to the determination of site-specific values of S_{ac}, which satisfy the requirements of this exception. In determining the value of S_{ac}, the value of T_L shall not be less than 4 s where

$$T_c = 2\pi\sqrt{\frac{D}{3.68g\tanh\left(\frac{3.68H}{D}\right)}} \quad (15.7\text{-}12)$$

and where

D = the tank diameter in ft (m), H = liquid height in ft (m), and g = acceleration due to gravity in consistent units
W_i = impulsive weight (impulsive component of liquid, roof and equipment, shell, bottom, and internal elements)
W_c = the portion of the liquid weight sloshing

15.7.6.1.1 Distribution of Hydrodynamic and Inertia Forces Unless otherwise required by the appropriate reference document listed in Chapter 23, the method given in ACI 350.3 is permitted to be used to determine the vertical and horizontal distribution of the hydrodynamic and inertia forces on the walls of circular and rectangular tanks.

15.7.6.1.2 Sloshing Sloshing of the stored liquid shall be taken into account in the seismic design of tanks and vessels in accordance with the following requirements:

a. The height of the sloshing wave, δ_s, shall be computed using Eq. 15.7-13 as follows:

$$\delta_s = 0.42D_iI_eS_{ac} \quad (15.7\text{-}13)$$

For cylindrical tanks, D_i shall be the inside diameter of the tank; for rectangular tanks, the term D_i shall be replaced by the longitudinal plan dimension of the tank, L, for the direction under consideration.
b. The effects of sloshing shall be accommodated by means of one of the following:
   1. A minimum freeboard in accordance with Table 15.7-3.
   2. A roof and supporting structure designed to contain the sloshing liquid in accordance with subsection 3 below.
   3. For open-top tanks or vessels only, an overflow spillway around the tank or vessel perimeter.

Table 15.7-3 Minimum Required Freeboard

Value of S_{DS}	Risk Category		
	I or II	III	IV
$S_{DS} < 0.167g$	*a*	*a*	δ_s[c]
$0.167g \le S_{DS} < 0.33g$	*a*	*a*	δ_s[c]
$0.33g \le S_{DS} < 0.50g$	*a*	$0.7\delta_s$[b]	δ_s[c]
$S_{DS} \ge 0.50g$	*a*	$0.7\delta_s$[b]	δ_s[c]

[a]NOTE: No minimum freeboard is required.
[c]Freeboard equal to the calculated wave height, δ_s, is required unless one of the following alternatives is provided: (1) Secondary containment is provided to control the product spill. (2) The roof and supporting structure are designed to contain the sloshing liquid.
[b]A freeboard equal to $0.7\delta_s$ is required unless one of the following alternatives is provided: (1) Secondary containment is provided to control the product spill. (2) The roof and supporting structure are designed to contain the sloshing liquid.

c. If the sloshing is restricted because the freeboard is less than the computed sloshing height, then the roof and supporting structure shall be designed for an equivalent hydrostatic head equal to the computed sloshing height less the freeboard. In addition, the design of the tank shall use the confined portion of the convective (sloshing) mass as an additional impulsive mass.

15.7.6.1.3 Equipment and Attached Piping Equipment, piping, and walkways or other appurtenances attached to the structure shall be designed to accommodate the displacements imposed by seismic forces. For piping attachments, see Section 15.7.4.

15.7.6.1.4 Internal Elements The attachments of internal equipment and accessories that are attached to the primary liquid or pressure retaining shell or bottom or that provide structural support for major elements (e.g., a column supporting the roof rafters) shall be designed for the lateral loads due to the sloshing liquid in addition to the inertial forces by a substantiated analysis method.

15.7.6.1.5 Sliding Resistance The transfer of the total lateral shear force between the tank or vessel and the subgrade shall be considered:

a. For unanchored flat bottom steel tanks, the overall horizontal seismic shear force is permitted to be resisted by friction between the tank bottom and the foundation or subgrade. Unanchored storage tanks shall be designed such that sliding will not occur where the tank is full of stored product. The maximum calculated seismic base shear, V, shall not exceed

$$V < W \tan 30° \qquad (15.7\text{-}14)$$

W shall be determined using the effective seismic weight of the tank, roof, and contents after reduction for coincident vertical earthquake. Lower values of the friction factor shall be used if the design of the tank bottom to supporting foundation does not justify the friction value above (e.g., leak detection membrane beneath the bottom with a lower friction factor, smooth bottoms, etc.). Alternatively, the friction factor is permitted to be determined by testing in accordance with Section 11.1.4.

b. No additional lateral anchorage is required for anchored steel tanks designed in accordance with reference documents.

c. The lateral shear transfer behavior for special tank configurations (e.g., shovel bottoms, highly crowned tank bottoms, tanks on grillage) can be unique and are beyond the scope of this standard.

15.7.6.1.6 Local Shear Transfer Local transfer of the shear from the roof to the wall and the wall of the tank into the base shall be considered. For cylindrical tanks and vessels, the peak local tangential shear per unit length shall be calculated by

$$v_{max} = \frac{2V}{\pi D} \qquad (15.7\text{-}15)$$

a. Tangential shear in flat bottom steel tanks shall be transferred through the welded connection to the steel bottom. This transfer mechanism is deemed acceptable for steel tanks designed in accordance with the reference documents where $S_{DS} < 1.0g$.

b. For concrete tanks with a sliding base where the lateral shear is resisted by friction between the tank wall and the base, the friction coefficient value used for design shall not exceed tan 30°.

c. Fixed-base or hinged-base concrete tanks transfer the horizontal seismic base shear shared by membrane (tangential) shear and radial shear into the foundation. For anchored flexible-base concrete tanks, the majority of the base shear is resisted by membrane (tangential) shear through the anchoring system with only insignificant vertical bending in the wall. The connection between the wall and floor shall be designed to resist the maximum tangential shear.

15.7.6.1.7 Pressure Stability For steel tanks, the internal pressure from the stored product stiffens thin cylindrical shell structural elements subjected to membrane compression forces. This stiffening effect is permitted to be considered in resisting seismically induced compressive forces if permitted by the reference document or the authority having jurisdiction.

15.7.6.1.8 Shell Support Steel tanks resting on concrete ring walls or slabs shall have a uniformly supported annulus under the shell. Uniform support shall be provided by one of the following methods:

a. Shimming and grouting the annulus.
b. Using fiberboard or other suitable padding.
c. Using butt-welded bottom or annular plates resting directly on the foundation.
d. Using closely spaced shims (without structural grout) provided that the localized bearing loads are considered in the tank wall and foundation to prevent local crippling and spalling.

Anchored tanks shall be shimmed and grouted. Local buckling of the steel shell for the peak compressive force due to operating loads and seismic overturning shall be considered.

15.7.6.1.9 Repair, Alteration, or Reconstruction Repairs, modifications, or reconstruction (i.e., cut down and re-erect) of a tank or vessel shall conform to industry standard practice and this standard. For welded steel tanks storing liquids, see API 653 and the applicable reference document listed in Chapter 23. Tanks that are relocated shall be re-evaluated for the seismic loads for the new site and the requirements of new construction in accordance with the appropriate reference document and this standard.

15.7.7 Water Storage and Water Treatment Tanks and Vessels

15.7.7.1 Welded Steel

Welded steel water storage tanks and vessels shall be designed in accordance with the seismic requirements of AWWA D100.

15.7.7.2 Bolted Steel

Bolted steel water storage structures shall be designed in accordance with the seismic requirements of AWWA D103 except that the design input forces of AWWA D100 shall be modified in the same manner shown in Section 15.7.7.1 of this standard.

15.7.7.3 Reinforced and Prestressed Concrete

Reinforced and prestressed concrete tanks shall be designed in accordance with the seismic requirements of AWWA D110, AWWA D115, or ACI 350.3 except that the importance factor, I_e, shall be determined according to Section 15.4.1.1, the response modification coefficient, R, shall be taken from Table 15.4-2, and the design input forces for strength design procedures shall be determined using the procedures of ACI 350.3 except

a. S_{ac} shall be substituted for C_c in ACI 350.3 Section 9.4.2 using Eqs. 15.7-10 for $T_c \le T_L$ and 15.7-11. for $T_c > T_L$ from Section 15.7.6.1; and
b. The value of C_t from ACI 350.3 Section 9.4.3 shall be determined using the procedures of Section 15.7.2(c). The values of I, Ri, and b as defined in ACI 350.3 shall be taken as 1.0 in the determination of vertical seismic effects.

15.7.8 Petrochemical and Industrial Tanks and Vessels Storing Liquids

15.7.8.1 Welded Steel

Welded steel petrochemical and industrial tanks and vessels storing liquids under an internal pressure of less than or equal to 2.5 psig (17.2 kpa g) shall be designed in accordance with the seismic requirements of API 650. Welded steel petrochemical and industrial tanks and vessels storing liquids under an internal pressure of greater than 2.5 psig (17.2 kpa g) and less than or equal to 15 psig (104.4 kpa g) shall be designed in accordance with the seismic requirements of API 620.

15.7.8.2 Bolted Steel

Bolted steel tanks used for storage of production liquids. API 12B covers the material, design, and erection requirements for vertical, cylindrical, aboveground bolted tanks in nominal capacities of 100 to 10,000 barrels for production service. Unless required by the authority having jurisdiction, these temporary structures need not be designed for seismic loads. If design for seismic load is required, the loads are permitted to be adjusted for the temporary nature of the anticipated service life.

15.7.8.3 Reinforced and Prestressed Concrete

Reinforced concrete tanks for the storage of petrochemical and industrial liquids shall be designed in accordance with the force requirements of Section 15.7.7.3.

15.7.9 Ground-Supported Storage Tanks for Granular Materials

15.7.9.1 General

The intergranular behavior of the material shall be considered in determining effective mass and load paths, including the following behaviors:

a. Increased lateral pressure (and the resulting hoop stress) due to loss of the intergranular friction of the material during the seismic shaking.
b. Increased hoop stresses generated from temperature changes in the shell after the material has been compacted.
c. Intergranular friction, which can transfer seismic shear directly to the foundation.

15.7.9.2 Lateral Force Determination

The lateral forces for tanks and vessels storing granular materials at grade shall be determined by the requirements and accelerations for short period structures (i.e., S_{DS}).

15.7.9.3 Force Distribution to Shell and Foundation

15.7.9.3.1 Increased Lateral Pressure The increase in lateral pressure on the tank wall shall be added to the static design lateral pressure but shall not be used in the determination of pressure stability effects on the axial buckling strength of the tank shell.

15.7.9.3.2 Effective Mass A portion of a stored granular mass will act with the shell (the effective mass). The effective mass is related to the physical characteristics of the product, the height-to-diameter (H/D) ratio of the tank, and the intensity of the seismic event. The effective mass shall be used to determine the shear and overturning loads resisted by the tank.

15.7.9.3.3 Effective Density The effective density factor (that part of the total stored mass of product that is accelerated by the seismic event) shall be determined in accordance with ACI 313.

15.7.9.3.4 Lateral Sliding For granular storage tanks that have a steel bottom and are supported such that friction at the bottom to foundation interface can resist lateral shear loads, no additional anchorage to prevent sliding is required. For tanks without steel bottoms (i.e., the material rests directly on the foundation), shear anchorage shall be provided to prevent sliding.

15.7.9.3.5 Combined Anchorage Systems If separate anchorage systems are used to prevent overturning and sliding, the relative stiffness of the systems shall be considered in determining the load distribution.

15.7.9.4 Welded Steel Structures

Welded steel granular storage structures shall be designed in accordance with the seismic requirements of this standard. Component allowable stresses and materials shall be per AWWA D100, except the allowable circumferential membrane stresses and material requirements in API 650 shall apply.

15.7.9.5 Bolted Steel Structures

Bolted steel granular storage structures shall be designed in accordance with the seismic requirements of this section. Component allowable stresses and materials shall be per AWWA D103.

15.7.9.6 Reinforced Concrete Structures Reinforced concrete structures for the storage of granular materials shall be designed in accordance with the seismic force requirements of this standard and the requirements of ACI 313.

15.7.9.7 Prestressed Concrete Structures

Prestressed concrete structures for the storage of granular materials shall be designed in accordance with the seismic force requirements of this standard and the requirements of ACI 313.

15.7.10 Elevated Tanks and Vessels for Liquids and Granular Materials

15.7.10.1 General

This section applies to tanks, vessels, bins, and hoppers that are elevated above grade where the supporting tower is an integral part of the structure, or where the primary function of the tower is to support the tank or vessel. Tanks and vessels that are supported within buildings or are incidental to the primary function of the tower are considered mechanical equipment and shall be designed in accordance with Chapter 13.

Elevated tanks shall be designed for the force and displacement requirements of the applicable reference document or Section 15.4.

15.7.10.2 Effective Mass

The design of the supporting tower or pedestal, anchorage, and foundation for seismic overturning shall assume the material stored is a rigid mass acting at the volumetric center of gravity. The effects of fluid–structure interaction are permitted to be considered in determining the forces, effective period, and mass centroids of the system if the following requirements are met:

a. The sloshing period, T_c is greater than $3T$ where T = natural period of the tank with confined liquid (rigid mass) and supporting structure.
b. The sloshing mechanism (i.e., the percentage of convective mass and centroid) is determined for the specific configuration of the container by detailed fluid–structure interaction analysis or testing.

Soil–structure interaction is permitted to be included in determining T providing the requirements of Chapter 19 are met.

15.7.10.3 P-Delta Effects

The lateral drift of the elevated tank shall be considered as follows:

a. The design drift, the elastic lateral displacement of the stored mass center of gravity, shall be increased by the factor C_d for evaluating the additional load in the support structure.
b. The base of the tank shall be assumed to be fixed rotationally and laterally.
c. Deflections due to bending, axial tension, or compression shall be considered. For pedestal tanks with a height-to-diameter ratio less than 5, shear deformations of the pedestal shall be considered.
d. The dead load effects of roof-mounted equipment or platforms shall be included in the analysis.
e. If constructed within the plumbness tolerances specified by the reference document, initial tilt need not be considered in the P-delta analysis.

15.7.10.4 Transfer of Lateral Forces into Support Tower

For post supported tanks and vessels that are cross-braced:

a. The bracing shall be installed in such a manner as to provide uniform resistance to the lateral load (e.g., pretensioning or tuning to attain equal sag).
b. The additional load in the brace due to the eccentricity between the post to tank attachment and the line of action of the bracing shall be included.
c. Eccentricity of compression strut line of action (elements that resist the tensile pull from the bracing rods in the seismic force-resisting systems) with their attachment points shall be considered.
d. The connection of the post or leg with the foundation shall be designed to resist both the vertical and lateral resultant from the yield load in the bracing assuming the direction of the lateral load is oriented to produce the maximum lateral shear at the post to foundation interface. Where multiple rods are connected to the same location, the anchorage shall be designed to resist the concurrent tensile loads in the braces.

15.7.10.5 Evaluation of Structures Sensitive to Buckling Failure

Shell structures that support substantial loads may exhibit a primary mode of failure from localized or general buckling of the support pedestal or skirt due to seismic loads. Such structures may include single pedestal water towers, skirt-supported process vessels, and similar single member towers. Where the structural assessment concludes that buckling of the support is the governing primary mode of failure, structures specified in this standard to be designed to subsections a and b below and those that are assigned as Risk Category IV shall be designed to resist the seismic forces as follows:

a. The seismic response coefficient for this evaluation shall be in accordance with Section 12.8.1.1 of this standard with I_e/R set equal to 1.0. Soil–structure and fluid–structure interaction is permitted to be utilized in determining the structural response. Vertical or orthogonal combinations need not be considered.
b. The resistance of the structure shall be defined as the critical buckling resistance of the element, that is, a factor of safety set equal to 1.0.

15.7.10.6 Welded Steel Water Storage Structures

Welded steel elevated water storage structures shall be designed and detailed in accordance with the seismic requirements of AWWA D100 with the structural height limits imposed by Table 15.4-2.

15.7.10.7 Concrete Pedestal (Composite) Tanks

Concrete pedestal (composite) elevated water storage structures shall be designed in accordance with the requirements of ACI 371R except that the design input forces shall be modified as follows:

In Eq. 4-8a of ACI 371R,

For $T_s < T \leq 2.5$ s, replace the term $\dfrac{1.2C_v}{RT^{2/3}}$ with

$$\frac{S_{D1}}{T\left(\dfrac{R}{I_e}\right)} \tag{15.7-24}$$

In Eq. 4-8b of ACI 371R, replace the term $\frac{2.5C_a}{R}$ with

$$\frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \tag{15.7-25}$$

In Eq. 4-9 of ACI 371R, replace the term $0.5C_a$ with

$$0.2S_{DS} \tag{15.7-26}$$

15.7.10.7.1 Analysis Procedures The equivalent lateral force procedure is permitted for all concrete pedestal tanks and shall be based on a fixed-base, single degree-of-freedom model. All mass, including the liquid, shall be considered rigid unless the sloshing mechanism (i.e., the percentage of convective mass and centroid) is determined for the specific configuration of the container by detailed fluid–structure interaction analysis or testing. Soil–structure interaction is permitted to be included. A more rigorous analysis is permitted.

15.7.10.7.2 Structure Period The fundamental period of vibration of the structure shall be established using the uncracked structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The period used to calculate the seismic response coefficient shall not exceed 2.5 s.

15.7.11 Boilers and Pressure Vessels

15.7.11.1 General

Attachments to the pressure boundary, supports, and seismic force-resisting anchorage systems for boilers and pressure vessels shall be designed to meet the force and displacement requirements of Section 15.3 or 15.4 and the additional requirements of this section. Boilers and pressure vessels categorized as Risk Categories III or IV shall be designed to meet the force and displacement requirements of Section 15.3 or 15.4.

15.7.11.2 ASME Boilers and Pressure Vessels

Boilers or pressure vessels designed and constructed in accordance with ASME BPVC shall be deemed to meet the requirements of this section provided that the force and displacement requirements of Section 15.3 or 15.4 are used with appropriate scaling of the force and displacement requirements to the working stress design basis.

15.7.11.3 Attachments of Internal Equipment and Refractory

Attachments to the pressure boundary for internal and external ancillary components (refractory, cyclones, trays, etc.) shall be designed to resist the seismic forces specified in this standard to safeguard against rupture of the pressure boundary. Alternatively, the element attached is permitted to be designed to fail prior to damaging the pressure boundary provided that the consequences of the failure do not place the pressure boundary in jeopardy. For boilers or vessels containing liquids, the effect of sloshing on the internal equipment shall be considered if the equipment can damage the integrity of the pressure boundary.

15.7.11.4 Coupling of Vessel and Support Structure

Where the mass of the operating vessel or vessels supported is greater than 25 percent of the total mass of the combined structure, the structure and vessel designs shall consider the effects of dynamic coupling between each other. Coupling with adjacent, connected structures such as multiple towers shall be considered if the structures are interconnected with elements that will transfer loads from one structure to the other.

15.7.11.5 Effective Mass

Fluid–structure interaction (sloshing) shall be considered in determining the effective mass of the stored material providing sufficient liquid surface exists for sloshing to occur and the T_c is greater than $3T$. Changes to or variations in material density with pressure and temperature shall be considered.

15.7.11.6 Other Boilers and Pressure Vessels

Boilers and pressure vessels designated Risk-Category IV, but not designed and constructed in accordance with the requirements of ASME BPVC, shall meet the following requirements:

The seismic loads in combination with other service loads and appropriate environmental effects shall not exceed the material strength shown in Table 15.7-4.

Consideration shall be made to mitigate seismic impact loads for boiler or vessel elements constructed of nonductile materials or vessels operated in such a way that material ductility is reduced (e.g., low temperature applications).

15.7.11.7 Supports and Attachments for Boilers and Pressure Vessels

Attachments to the pressure boundary and support for boilers and pressure vessels shall meet the following requirements:

Table 15.7-4 Maximum Material Strength

Material	Minimum Ratio F_u/F_y	Max. Material Strength Vessel Material	Max. Material Strength Threaded Material[a]
Ductile (e.g., steel, aluminum, copper)	1.33[b]	90%[d]	70%[d]
Semiductile	1.2[c]	70%[d]	50%[d]
Nonductile (e.g., cast iron, ceramics, fiberglass)	NA	25%[e]	20%[e]

[a]Threaded connection to vessel or support system.
[b]Minimum 20% elongation per the ASTM material specification.
[d]Based on material minimum specified yield strength.
[c]Minimum 15% elongation per the ASTM material specification.
[e]Based on material minimum specified tensile strength.

a. Attachments and supports transferring seismic loads shall be constructed of ductile materials suitable for the intended application and environmental conditions.
b. Anchorage shall be in accordance with Section 15.4.9, whereby the anchor embedment into the concrete is designed to develop the steel strength of the anchor in tension. The steel strength of the anchor in tension shall be determined in accordance with ACI 318 Appendix D Eq. D-3. The anchor shall have a minimum gauge length of eight diameters. The load combinations with overstrength of Section 12.4.3 are not to be used to size the anchor bolts for tanks and horizontal and vertical vessels.
c. Seismic supports and attachments to structures shall be designed and constructed so that the support or attachment remains ductile throughout the range of reversing seismic lateral loads and displacements.
d. Vessel attachments shall consider the potential effect on the vessel and the support for uneven vertical reactions based on variations in relative stiffness of the support members, dissimilar details, nonuniform shimming, or irregular supports. Uneven distribution of lateral forces shall consider the relative distribution of the resisting elements, the behavior of the connection details, and vessel shear distribution.

The requirements of Sections 15.4 and 15.7.10.5 shall also be applicable to this section.

15.7.12 Liquid and Gas Spheres

15.7.12.1 General

Attachments to the pressure or liquid boundary, supports, and seismic force-resisting anchorage systems for liquid and gas spheres shall be designed to meet the force and displacement requirements of Section 15.3 or 15.4 and the additional requirements of this section. Spheres categorized as Risk Category III or IV shall themselves be designed to meet the force and displacement requirements of Section 15.3 or 15.4.

15.7.12.2 ASME Spheres

Spheres designed and constructed in accordance with Section VIII of ASME BPVC shall be deemed to meet the requirements of this section providing the force and displacement requirements of Section 15.3 or 15.4 are used with appropriate scaling of the force and displacement requirements to the working stress design basis.

15.7.12.3 Attachments of Internal Equipment and Refractory

Attachments to the pressure or liquid boundary for internal and external ancillary components (refractory, cyclones, trays, etc.) shall be designed to resist the seismic forces specified in this standard to safeguard against rupture of the pressure boundary. Alternatively, the element attached to the sphere could be designed to fail prior to damaging the pressure or liquid boundary providing the consequences of the failure does not place the pressure boundary in jeopardy. For spheres containing liquids, the effect of sloshing on the internal equipment shall be considered if the equipment can damage the pressure boundary.

15.7.12.4 Effective Mass

Fluid–structure interaction (sloshing) shall be considered in determining the effective mass of the stored material providing sufficient liquid surface exists for sloshing to occur and the T_c is greater than $3T$. Changes to or variations in fluid density shall be considered.

15.7.12.5 Post and Rod Supported

For post supported spheres that are cross-braced:

a. The requirements of Section 15.7.10.4 shall also be applicable to this section.
b. The stiffening effect (reduction in lateral drift) from pretensioning of the bracing shall be considered in determining the natural period.
c. The slenderness and local buckling of the posts shall be considered.
d. Local buckling of the sphere shell at the post attachment shall be considered.
e. For spheres storing liquids, bracing connections shall be designed and constructed to develop the minimum published yield strength of the brace. For spheres storing gas vapors only, bracing connection shall be designed for Ω_0 times the maximum design load in the brace. Lateral bracing connections directly attached to the pressure or liquid boundary are prohibited.

15.7.12.6 Skirt Supported

For skirt-supported spheres, the following requirements shall apply:

a. The requirements of Section 15.7.10.5 shall also apply.
b. The local buckling of the skirt under compressive membrane forces due to axial load and bending moments shall be considered.
c. Penetration of the skirt support (manholes, piping, etc.) shall be designed and constructed to maintain the strength of the skirt without penetrations.

15.7.13 Refrigerated Gas Liquid Storage Tanks and Vessels

15.7.13.1 General

Tanks and facilities for the storage of liquefied hydrocarbons and refrigerated liquids shall meet the requirements of this standard. Low-pressure welded steel storage tanks for liquefied hydrocarbon gas (e.g., LPG, butane, etc.) and refrigerated liquids (e.g., ammonia) shall be designed in accordance with the requirements of Section 15.7.8 and API 620.

15.7.14 Horizontal, Saddle Supported Vessels for Liquid or Vapor Storage

15.7.14.1 General

Horizontal vessels supported on saddles (sometimes referred to as “blimps”) shall be designed to meet the force and displacement requirements of Section 15.3 or 15.4.

15.7.14.2 Effective Mass

Changes to or variations in material density shall be considered. The design of the supports, saddles, anchorage, and foundation for seismic overturning shall assume the material stored is a rigid mass acting at the volumetric center of gravity.

15.7.14.3 Vessel Design

Unless a more rigorous analysis is performed

a. Horizontal vessels with a length-to-diameter ratio of 6 or more are permitted to be assumed to be a simply supported beam spanning between the saddles for determining the natural period of vibration and global bending moment.
b. For horizontal vessels with a length-to-diameter ratio of less than 6, the effects of “deep beam shear” shall be considered where determining the fundamental period and stress distribution.
c. Local bending and buckling of the vessel shell at the saddle supports due to seismic load shall be considered. The stabilizing effects of internal pressure shall not be considered to increase the buckling resistance of the vessel shell.
d. If the vessel is a combination of liquid and gas storage, the vessel and supports shall be designed both with and without gas pressure acting (assume piping has ruptured and pressure does not exist).

Chapter 16
SEISMIC RESPONSE HISTORY PROCEDURES

16.1 LINEAR RESPONSE HISTORY PROCEDURE

Where linear response history procedure is performed the requirements of this chapter shall be satisfied.

16.1.1 Analysis Requirements

A linear response history analysis shall consist of an analysis of a linear mathematical model of the structure to determine its response, through methods of numerical integration, to suites of ground motion acceleration histories compatible with the design response spectrum for the site. The analysis shall be performed in accordance with the requirements of this section.

16.1.2 Modeling

Mathematical models shall conform to the requirements of Section 12.7.

16.1.3 Ground Motion

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

16.1.3.1 Two-Dimensional Analysis

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

16.1.3.2 Three-Dimensional Analysis

Where three-dimensional analyses are performed, ground motions shall consist of pairs of appropriate horizontal ground motion acceleration components that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of recorded ground motion pairs is not available, appropriate simulated ground motion pairs are permitted to be used to make up the total number required. For each pair of horizontal ground motion components, a square root of the sum of the squares (SRSS) spectrum shall be constructed by taking the SRSS of the 5 percent-damped response spectra for the scaled components (where an identical scale factor is applied to both components of a pair). Each pair of motions shall be scaled such that in the period range from $0.2T$ to $1.5T$, the average of the SRSS spectra from all horizontal component pairs does not fall below the corresponding ordinate of the response spectrum used in the design, determined in accordance with Section 11.4.5 or 11.4.7.

At sites within 3 miles (5 km) of the active fault that controls the hazard, each pair of components shall be rotated to the fault-normal and fault-parallel directions of the causative fault and shall be scaled so that the average of the fault-normal components is not less than the MCE_R response spectrum for the period range from $0.2T$ to $1.5T$.

16.1.4 Response Parameters

For each ground motion analyzed, the individual response parameters shall be multiplied by the following scalar quantities:

a. Force response parameters shall be multiplied by I_e/R, where I_e is the importance factor determined in accordance with Section 11.5.1 and R is the Response Modification Coefficient selected in accordance with Section 12.2.1.
b. Drift quantities shall be multiplied by C_d/R, where C_d is the deflection amplification factor specified in Table 12.2-1.

For each ground motion i, where i is the designation assigned to each ground motion, the maximum value of the base shear, V_i, member forces, Q_{Ei}, scaled as indicated in the preceding text and story drifts, Δ_i, at each story as defined in Section 12.8.6 shall be determined. Where the maximum scaled base shear predicted by the analysis, V_i, is less than 85 percent of the value of V determined using the minimum value of C_s set forth in Eq. 12.8-5 or when located where S_1 is equal to or greater than 0.6g, the minimum value of C_s set forth in Eq. 12.8-6, the scaled member forces, Q_{Ei}, shall be additionally multiplied by $\frac{V}{Vi}$ where V is the minimum base shear that has been determined using the minimum value of C_s set forth in Eq. 12.8-5, or when located where S_1 is equal to or greater than 0.6g, the minimum value of C_s set forth in Eq. 12.8-6. Where the maximum scaled base shear predicted by the analysis, V_i, is less than $0.85C_sW$, where C_s is from Eq. 12.8-6, drifts shall be multiplied by $0.85\frac{C_sW}{V_i}$.

If at least seven ground motions are analyzed, the design member forces used in the load combinations of Section 12.4.2.1 and the design story drift used in the evaluation of drift in accordance with Section 12.12.1 are permitted to be taken respectively as the average of the scaled Q_{Ei} and Δ_i values determined from the analyses and scaled as indicated in the preceding text. If fewer than seven ground motions are analyzed, the design member forces and the design story drift shall be taken as the maximum value of the scaled Q_{Ei} and Δ_i values determined from the analyses.

Where this standard requires consideration of the seismic load effects including overstrength factor of Section 12.4.3, the value of $\Omega_0 Q_E$ need not be taken larger than the maximum of the unscaled value, Q_{Ei}, obtained from the analyses.

16.1.5 Horizontal Shear Distribution

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

16.2 NONLINEAR RESPONSE HISTORY PROCEDURE

Where nonlinear response history procedure is performed the requirements of Section 16.2 shall be satisfied.

16.2.1 Analysis Requirements

A nonlinear response history analysis shall consist of an analysis of a mathematical model of the structure that directly accounts for the nonlinear hysteretic behavior of the structure's elements to determine its response through methods of numerical integration to suites of ground motion acceleration histories compatible with the design response spectrum for the site. The analysis shall be performed in accordance with this section. See Section 12.1.1 for limitations on the use of this procedure.

16.2.2 Modeling

A mathematical model of the structure shall be constructed that represents the spatial distribution of mass throughout the structure. The hysteretic behavior of elements shall be modeled consistent with suitable laboratory test data and shall account for all significant yielding, strength degradation, stiffness degradation, and hysteretic pinching indicated by such test data. Strength of elements shall be based on expected values considering material overstrength, strain hardening, and hysteretic strength degradation. Linear properties, consistent with the requirements of Section 12.7.3, are permitted to be used for those elements demonstrated by the analysis to remain within their linear range of response. The structure shall be assumed to have a fixed-base, or alternatively, it is permitted to use realistic assumptions with regard to the stiffness and load-carrying characteristics of the foundations consistent with site-specific soils data and rational principles of engineering mechanics.

For regular structures with independent orthogonal seismic force-resisting systems, independent 2-D models are permitted to be constructed to represent each system. For structures having a horizontal structural irregularity of Type 1a, 1b, 4, or 5 of Table 12.3-1 or structures without independent orthogonal systems, a 3-D model incorporating a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis at each level of the structure shall be used. Where the diaphragms are not rigid compared to the vertical elements of the seismic force-resisting system, the model should include representation of the diaphragm's flexibility and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response.

16.2.3 Ground Motion and Other Loading

Ground motion shall conform to the requirements of Section 16.1.3. The structure shall be analyzed for the effects of these ground motions simultaneously with the effects of dead load in combination with not less than 25 percent of the required live loads.

16.2.4 Response Parameters

For each ground motion analyzed, individual response parameters consisting of the maximum value of the individual member forces, Q_{Ei}, member inelastic deformations, ψ_i, and story drifts, Δ_i, at each story shall be determined, where i is the designation assigned to each ground motion.

If at least seven ground motions are analyzed, the design values of member forces, Q_E, member inelastic deformations, ψ, and story drift, Δ, are permitted to be taken as the average of the Q_{Ei}, ψ_i, and Δ_i values determined from the analyses. If fewer than seven ground motions are analyzed, the design member forces, Q_E, design member inelastic deformations, ψ, and the design story drift, Δ, shall be taken as the maximum value of the Q_{Ei}, ψ_i, and Δ_i values determined from the analyses.

16.2.4.1 Member Strength

The adequacy of members to resist the combination of load effects of Section 12.4 need not be evaluated.

EXCEPTION: Where this standard requires consideration of the seismic load effects including overstrength factor of Section 12.4.3, the maximum value of Q_{Ei} obtained from the suite of analyses shall be taken in place of the quantity $\Omega_0 Q_E$.

16.2.4.2 Member Deformation

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

16.2.4.3 Story Drift

The design story drift, Δ_i, obtained from the analyses shall not exceed 125 percent of the drift limit specified in Section 12.12.1.

16.2.5 Design Review

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

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

Chapter 17

SEISMIC DESIGN REQUIREMENTS FOR SEISMICALLY ISOLATED STRUCTURES

17.1 GENERAL

Every seismically isolated structure and every portion thereof shall be designed and constructed in accordance with the requirements of this section and the applicable requirements of this standard.

17.1.1 Variations in Material Properties

The analysis of seismically isolated structures, including the substructure, isolators, and superstructure, shall consider variations in seismic isolator material properties over the projected life of the structure including changes due to aging, contamination, environmental exposure, loading rate, scragging, and temperature.

17.1.2 Definitions

DISPLACEMENT:

Design Displacement: The design earthquake lateral displacement, excluding additional displacement due to actual and accidental torsion, required for design of the isolation system.

Total Design Displacement: The design earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for design of the isolation system or an element thereof.

Total Maximum Displacement: The maximum considered earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for verification of the stability of the isolation system or elements thereof, design of structure separations, and vertical load testing of isolator unit prototypes.

DISPLACEMENT RESTRAINT SYSTEM: A collection of structural elements that limits lateral displacement of seismically isolated structures due to the maximum considered earthquake.

EFFECTIVE DAMPING: The value of equivalent viscous damping corresponding to energy dissipated during cyclic response of the isolation system.

EFFECTIVE STIFFNESS: The value of the lateral force in the isolation system, or an element thereof, divided by the corresponding lateral displacement.

ISOLATION INTERFACE: The boundary between the upper portion of the structure, which is isolated, and the lower portion of the structure, which moves rigidly with the ground.

ISOLATION SYSTEM: The collection of structural elements that includes all individual isolator units, all structural elements that transfer force between elements of the isolation system, and all connections to other structural elements. The isolation system also includes the wind-restraint system, energy-dissipation devices, and/or the displacement restraint system if such systems and devices are used to meet the design requirements of this chapter.

ISOLATOR UNIT: A horizontally flexible and vertically stiff structural element of the isolation system that permits large lateral deformations under design seismic load. An isolator unit is permitted to be used either as part of, or in addition to, the weight-supporting system of the structure.

MAXIMUM DISPLACEMENT: The maximum considered earthquake lateral displacement, excluding additional displacement due to actual and accidental torsion.

SCRAGGING: Cyclic loading or working of rubber products, including elastomeric isolators, to effect a reduction in stiffness properties, a portion of which will be recovered over time.

WIND-RESTRAINT SYSTEM: The collection of structural elements that provides restraint of the seismic-isolated structure for wind loads. The wind-restraint system is permitted to be either an integral part of isolator units or a separate device.

17.1.3 Notation

B_D = numerical coefficient as set forth in Table 17.5-1 for effective damping equal to β_D

B_M = numerical coefficient as set forth in Table 17.5-1 for effective damping equal to β_M

b = shortest plan dimension of the structure, in ft (mm) measured perpendicular to d

D_D = design displacement, in in. (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Eq. 17.5-1

D'_D = design displacement, in in. (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Eq. 17.6-1

D_M = maximum displacement, in in. (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Eq. 17.5-3

D'_M = maximum displacement, in in. (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Eq. 17.6-2

D_{TD} = total design displacement, in in. (mm), of an element of the isolation system including both translational displacement at the center of rigidity and the component of torsional displacement in the direction under consideration, as prescribed by Eq. 17.5-5

D_{TM} = total maximum displacement, in in. (mm), of an element of the isolation system including both translational displacement at the center of rigidity and the component of torsional displacement in the direction under consideration, as prescribed by Eq. 17.5-6

d = longest plan dimension of the structure, in ft (mm)

E_{loop} = energy dissipated in kips-in. (kN-mm), in an isolator unit during a full cycle of reversible load over a test displacement range from Δ^+ to Δ^-, as measured by the area enclosed by the loop of the force-deflection curve

e = actual eccentricity, in ft (mm), measured in plan between the center of mass of the structure above the isolation interface and the center of rigidity of the isolation system, plus accidental eccentricity, in ft. (mm), taken as 5 percent of the maximum building dimension perpendicular to the direction of force under consideration

F^- = minimum negative force in an isolator unit during a single cycle of prototype testing at a displacement amplitude of Δ^-

F^+ = maximum positive force in kips (kN) in an isolator unit during a single cycle of prototype testing at a displacement amplitude of Δ^+

F_x = total force distributed over the height of the structure above the isolation interface as prescribed by Eq. 17.5-9

$k_{D\max}$ = maximum effective stiffness, in kips/in. (kN/mm), of the isolation system at the design displacement in the horizontal direction under consideration, as prescribed by Eq. 17.8-3

$k_{D\min}$ = minimum effective stiffness, in kips/in. (kN/mm), of the isolation system at the design displacement in the horizontal direction under consideration, as prescribed by Eq. 17.8-4

$k_{M\max}$ = maximum effective stiffness, in kips/in. (kN/mm), of the isolation system at the maximum displacement in the horizontal direction under consideration, as prescribed by Eq. 17.8-5

$k_{M\min}$ = minimum effective stiffness, in kips/in. (kN/mm), of the isolation system at the maximum displacement in the horizontal direction under consideration, as prescribed by Eq. 17.8-6

k_{eff} = effective stiffness of an isolator unit, as prescribed by Eq. 17.8-1

L = effect of live load in Chapter 17

T_D = effective period, in s, of the seismically isolated structure at the design displacement in the direction under consideration, as prescribed by Eq. 17.5-2

T_M = effective period, in s, of the seismically isolated structure at the maximum displacement in the direction under consideration, as prescribed by Eq. 17.5-4

V_b = total lateral seismic design force or shear on elements of the isolation system or elements below isolation system, as prescribed by Eq. 17.5-7

V_s = total lateral seismic design force or shear on elements above the isolation system, as prescribed by Eq. 17.5-8

y = distance, in ft (mm), between the center of rigidity of the isolation system rigidity and the element of interest measured perpendicular to the direction of seismic loading under consideration

β_D = effective damping of the isolation system at the design displacement, as prescribed by Eq. 17.8-7

β_M = effective damping of the isolation system at the maximum displacement, as prescribed by Eq. 17.8-8

β_{eff} = effective damping of the isolation system, as prescribed by Eq. 17.8-2

Δ^+ = maximum positive displacement of an isolator unit during each cycle of prototype testing

Δ^- = minimum negative displacement of an isolator unit during each cycle of prototype testing

ΣE_D = total energy dissipated, in kips-in. (kN-mm), in the isolation system during a full cycle of response at the design displacement, D_D

ΣE_M = total energy dissipated, in kips-in. (kN-mm), in the isolation system during a full cycle of response at the maximum displacement, D_M

$\Sigma|F_D^+|_{max}$ = sum, for all isolator units, of the maximum absolute value of force, in kips (kN), at a positive displacement equal to D_D

$\Sigma|F_D^+|_{min}$ = sum, for all isolator units, of the minimum absolute value of force, in kips (kN), at a positive displacement equal to D_D

$\Sigma|F_D^-|_{max}$ = sum, for all isolator units, of the maximum absolute value of force, in kips (kN), at a negative displacement equal to D_D

$\Sigma|F_D^-|_{min}$ = sum, for all isolator units, of the minimum absolute value of force, in kips (kN), at a negative displacement equal to D_D

$\Sigma|F_M^+|_{max}$ = sum, for all isolator units, of the maximum absolute value of force, in kips (kN), at a positive displacement equal to D_M

$\Sigma|F_M^+|_{min}$ = sum, for all isolator units, of the minimum absolute value of force, in kips (kN), at a positive displacement equal to D_M

$\Sigma|F_M^-|_{max}$ = sum, for all isolator units, of the maximum absolute value of force, in kips (kN), at a negative displacement equal to D_M

$\Sigma|F_M^-|_{min}$ = sum, for all isolator units, of the minimum absolute value of force, in kips (kN), at a negative displacement equal to D_M

17.2 GENERAL DESIGN REQUIREMENTS

17.2.1 Importance Factor

All portions of the structure, including the structure above the isolation system, shall be assigned a risk category in accordance with Table 1.5-1. The importance factor, I_e, shall be taken as 1.0 for a seismically isolated structure, regardless of its risk category assignment.

17.2.2 MCE$_R$ Spectral Response Acceleration Parameters, S_{MS} and S_{M1}

The MCE$_R$ spectral response acceleration parameters S_{MS} and S_{M1} shall be determined in accordance with Section 11.4.3.

17.2.3 Configuration

Each structure shall be designated as having a structural irregularity based on the structural configuration above the isolation system.

17.2.4 Isolation System

17.2.4.1 Environmental Conditions

In addition to the requirements for vertical and lateral loads induced by wind and earthquake, the isolation system shall provide for other environmental conditions including aging effects, creep, fatigue, operating temperature, and exposure to moisture or damaging substances.

17.2.4.2 Wind Forces

Isolated structures shall resist design wind loads at all levels above the isolation interface. At the isolation interface, a wind-restraint system shall be provided to limit lateral displacement in the isolation system to a value equal to that required between floors of the structure above the isolation interface in accordance with Section 17.5.6.

17.2.4.3 Fire Resistance

Fire resistance for the isolation system shall meet that required for the columns, walls, or other such gravity-bearing elements in the same region of the structure.

17.2.4.4 Lateral Restoring Force

The isolation system shall be configured to produce a restoring force such that the lateral force at the total design displacement is at least $0.025W$ greater than the lateral force at 50 percent of the total design displacement.

17.2.4.5 Displacement Restraint

The isolation system shall not be configured to include a displacement restraint that limits lateral displacement due to the maximum considered earthquake to less than the total maximum displacement unless the seismically isolated structure is designed in accordance with the following criteria where more stringent than the requirements of Section 17.2:

1. Maximum considered earthquake response is calculated in accordance with the dynamic analysis requirements of Section 17.6, explicitly considering the nonlinear characteristics of the isolation system and the structure above the isolation system.
2. The ultimate capacity of the isolation system and structural elements below the isolation system shall exceed the strength and displacement demands of the maximum considered earthquake.
3. The structure above the isolation system is checked for stability and ductility demand of the maximum considered earthquake.
4. The displacement restraint does not become effective at a displacement less than 0.75 times the total design displacement unless it is demonstrated by analysis that earlier engagement does not result in unsatisfactory performance.

17.2.4.6 Vertical-Load Stability

Each element of the isolation system shall be designed to be stable under the design vertical load where subjected to a horizontal displacement equal to the total maximum displacement. The design vertical load shall be computed using load combination 5 of Section 2.3.2 for the maximum vertical load and load combination 7 of Section 12.4.2.3 for the minimum vertical load where S_{DS} in these equations is replaced by S_{MS}. The vertical loads that result from application of horizontal seismic forces, Q_E, shall be based on peak response due to the maximum considered earthquake.

17.2.4.7 Overturning

The factor of safety against global structural overturning at the isolation interface shall not be less than 1.0 for required load combinations. All gravity and seismic loading conditions shall be investigated. Seismic forces for overturning calculations shall be based on the maximum considered earthquake, and W shall be used for the vertical restoring force.

Local uplift of individual elements shall not be allowed unless the resulting deflections do not cause overstress or instability of the isolator units or other structure elements.

17.2.4.8 Inspection and Replacement

a. Access for inspection and replacement of all components of the isolation system shall be provided.
b. A registered design professional shall complete a final series of inspections or observations of structure separation areas and components that cross the isolation interface prior to the issuance of the certificate of occupancy for the seismically isolated structure. Such inspections and observations shall indicate that the conditions allow free and unhindered displacement of the structure to maximum design levels and that all components that cross the isolation interface as installed are able to accommodate the stipulated displacements.
c. Seismically isolated structures shall have a monitoring, inspection, and maintenance program for the isolation system established by the registered design professional responsible for the design of the isolation system.
d. Remodeling, repair, or retrofitting at the isolation system interface, including that of components that cross the isolation interface, shall be performed under the direction of a registered design professional.

17.2.4.9 Quality Control

A quality control testing program for isolator units shall be established by the registered design professional responsible for the structural design.

17.2.5 Structural System

17.2.5.1 Horizontal Distribution of Force

A horizontal diaphragm or other structural elements shall provide continuity above the isolation interface and shall have adequate strength and ductility to transmit forces (due to nonuniform ground motion) from one part of the structure to another.

17.2.5.2 Building Separations

Minimum separations between the isolated structure and surrounding retaining walls or other fixed obstructions shall not be less than the total maximum displacement.

17.2.5.3 Nonbuilding Structures

Nonbuilding structures shall be designed and constructed in accordance with the requirements of Chapter 15 using design displacements and forces calculated in accordance with Sections 17.5 or 17.6.

17.2.6 Elements of Structures and Nonstructural Components

Parts or portions of an isolated structure, permanent nonstructural components and the attachments to them, and the attachments for permanent equipment supported by a structure shall be designed to resist seismic forces and displacements as prescribed by this section and the applicable requirements of Chapter 13.

17.2.6.1 Components at or above the Isolation Interface

Elements of seismically isolated structures and nonstructural components, or portions thereof, that are at or above the isolation interface shall be designed to resist a total lateral seismic force equal to the maximum dynamic response of the element or component under consideration.

EXCEPTION: Elements of seismically isolated structures and nonstructural components or portions designed to resist seismic forces and displacements as prescribed in Chapter 12 or 13 as appropriate.

17.2.6.2 Components Crossing the Isolation Interface

Elements of seismically isolated structures and nonstructural components, or portions thereof, that cross the isolation interface shall be designed to withstand the total maximum displacement.

17.2.6.3 Components below the Isolation Interface

Elements of seismically isolated structures and nonstructural components, or portions thereof, that are below the isolation interface shall be designed and constructed in accordance with the requirements of Section 12.1 and Chapter 13.

17.3 GROUND MOTION FOR ISOLATED SYSTEMS

17.3.1 Design Spectra

The site-specific ground motion procedures set forth in Chapter 21 are permitted to be used to determine ground motions for any structure. For structures on Site Class F sites, site response analysis shall be performed in accordance with Section 21.1. For seismically isolated structures on sites with S_1 greater than or equal to 0.6, a ground motion hazard analysis shall be performed in accordance with Section 21.2. Structures that do not require or use site-specific ground motion procedures shall be analyzed using the design spectrum for the design earthquake developed in accordance with Section 11.4.5.

A spectrum shall be constructed for the MCE_R ground motion. The spectrum for MCE_R ground motions shall not be taken as less than 1.5 times the spectrum for the design earthquake ground motions.

17.3.2 Ground Motion Histories

Where response-history procedures are used, ground motions shall consist of pairs of appropriate horizontal ground motion acceleration components developed per Section 16.1.3.2 except that $0.2T$ and $1.5T$ shall be replaced by $0.5T_D$ and $1.25T_M$, respectively, where T_D and T_M are defined in Section 17.5.3.

17.4 ANALYSIS PROCEDURE SELECTION

Seismically isolated structures except those defined in Section 17.4.1 shall be designed using the dynamic procedures of Section 17.6.

17.4.1 Equivalent Lateral Force Procedure

The equivalent lateral force procedure of Section 17.5 is permitted to be used for design of a seismically isolated structure provided that

1. The structure is located at a site with S_1 less than $0.60g$.
2. The structure is located on a Site Class A, B, C, or D.
3. The structure above the isolation interface is less than or equal to four stories or 65 ft (19.8 m) in structural height, h_n, measured from the base as defined in Section 11.2.
4. The effective period of the isolated structure at the maximum displacement, T_M, is less than or equal to 3.0 s.
5. The effective period of the isolated structure at the design displacement, T_D, is greater than three times the elastic, fixed-base period of the structure above the isolation system as determined by Eq. 12.8-7 or 12.8-8.
6. The structure above the isolation system is of regular configuration.
7. The isolation system meets all of the following criteria:
   a. The effective stiffness of the isolation system at the design displacement is greater than one-third of the effective stiffness at 20 percent of the design displacement.
   b. The isolation system is capable of producing a restoring force as specified in Section 17.2.4.4.
   c. The isolation system does not limit maximum considered earthquake displacement to less than the total maximum displacement.

17.4.2 Dynamic Procedures

The dynamic procedures of Section 17.6 are permitted to be used as specified in this section.

17.4.2.1 Response-Spectrum Procedure

Response-spectrum analysis shall not be used for design of a seismically isolated structure unless:

1. The structure is located on a Site Class A, B, C, or D.
2. The isolation system meets the criteria of Item 7 of Section 17.4.1.

17.4.2.2 Response-History Procedure

The response-history procedure is permitted for design of any seismically isolated structure and shall be used for design of all seismically isolated structures not meeting the criteria of Section 17.4.2.1.

17.5 EQUIVALENT LATERAL FORCE PROCEDURE

17.5.1 General

Where the equivalent lateral force procedure is used to design seismically isolated structures, the requirements of this section shall apply.

17.5.2 Deformation Characteristics of the Isolation System

Minimum lateral earthquake design displacements and forces on seismically isolated structures shall be based on the deformation characteristics of the isolation system. The deformation characteristics of the isolation system shall explicitly include the effects of the wind-restraint system if such a system is used to meet the design requirements of this standard. The deformation characteristics of the isolation system shall be based on properly substantiated tests performed in accordance with Section 17.8.

17.5.3 Minimum Lateral Displacements

17.5.3.1 Design Displacement

The isolation system shall be designed and constructed to withstand minimum lateral earthquake displacements, D_D, that act in the direction of each of the main horizontal axes of the structure using Eq. 17.5-1:

$$D_D = \frac{g S_{D1} T_D}{4\pi^2 B_D} \qquad (17.5\text{-}1)$$

where

g = acceleration due to gravity. The units for g are in./s^2 (mm/s^2) if the units of the design displacement, D_D, are in. (mm)

S_{D1} = design 5 percent damped spectral acceleration parameter at 1-s period in units of g-s, as determined in Section 11.4.4

Table 17.5-1 Damping Coefficient, B_D or B_M

Effective Damping, β_D or β_M (percentage of critical)[a,b]	B_D or B_M Factor
≤2	0.8
5	1.0
10	1.2
20	1.5
30	1.7
40	1.9
≥50	2.0

[a]The damping coefficient shall be based on the effective damping of the isolation system determined in accordance with the requirements of Section 17.8.5.2.

[b]The damping coefficient shall be based on linear interpolation for effective damping values other than those given.

T_D = effective period of the seismically isolated structure in seconds, at the design displacement in the direction under consideration, as prescribed by Eq. 17.5-2

B_D = numerical coefficient related to the effective damping of the isolation system at the design displacement, β_D, as set forth in Table 17.5-1

17.5.3.2 Effective Period at Design Displacement

The effective period of the isolated structure at design displacement, T_D, shall be determined using the deformational characteristics of the isolation system and Eq. 17.5-2:

$$T_D = 2\pi\sqrt{\frac{W}{k_{D\min} g}} \qquad (17.5\text{-}2)$$

where

W = effective seismic weight of the structure above the isolation interface as defined in Section 12.7.2

$k_{D\min}$ = minimum effective stiffness in kips/in. (kN/mm) of the isolation system at the design displacement in the horizontal direction under consideration, as prescribed by Eq. 17.8-4

g = acceleration due to gravity

17.5.3.3 Maximum Displacement

The maximum displacement of the isolation system, D_M, in the most critical direction of horizontal response shall be calculated using Eq. 17.5-3:

$$D_M = \frac{g S_{M1} T_M}{4\pi^2 B_M} \qquad (17.5\text{-}3)$$

where

g = acceleration of gravity
S_{M1} = maximum considered earthquake 5 percent damped spectral acceleration parameter at 1-s period, in units of g-s, as determined in Section 11.4.3
T_M = effective period, in seconds, of the seismically isolated structure at the maximum displacement in the direction under consideration, as prescribed by Eq. 17.5-4
B_M = numerical coefficient related to the effective damping of the isolation system at the maximum displacement, β_M, as set forth in Table 17.5-1

17.5.3.4 Effective Period at Maximum Displacement

The effective period of the isolated structure at maximum displacement, T_M, shall be determined using the deformational characteristics of the isolation system and Eq. 17.5-4:

$$T_M = 2\pi\sqrt{\frac{W}{k_{M\min}g}} \qquad (17.5\text{-}4)$$

where

W = effective seismic weight of the structure above the isolation interface as defined in Section 12.7.2 (kip or kN)
$k_{M\min}$ = minimum effective stiffness, in kips/in. (kN/mm), of the isolation system at the maximum displacement in the horizontal direction under consideration, as prescribed by Eq. 17.8-6
g = the acceleration of gravity

17.5.3.5 Total Displacement

The total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, of elements of the isolation system shall include additional displacement due to actual and accidental torsion calculated from the spatial distribution of the lateral stiffness of the isolation system and the most disadvantageous location of eccentric mass.

The total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, of elements of an isolation system with uniform spatial distribution of lateral stiffness shall not be taken as less than that prescribed by Eqs. 17.5-5 and 17.5-6:

$$D_{TD} = D_D\left[1 + y\frac{12e}{b^2 + d^2}\right] \qquad (17.5\text{-}5)$$

$$D_{TM} = D_M\left[1 + y\frac{12e}{b^2 + d^2}\right] \qquad (17.5\text{-}6)$$

where

D_D = design displacement at the center of rigidity of the isolation system in the direction under consideration as prescribed by Eq. 17.5-1
D_M = maximum displacement at the center of rigidity of the isolation system in the direction under consideration as prescribed by Eq. 17.5-3
y = the distance between the centers of rigidity of the isolation system and the element of interest measured perpendicular to the direction of seismic loading under consideration
e = the actual eccentricity measured in plan between the center of mass of the structure above the isolation interface and the center of rigidity of the isolation system, plus accidental eccentricity, in ft (mm), taken as 5 percent of the longest plan dimension of the structure perpendicular to the direction of force under consideration
b = the shortest plan dimension of the structure measured perpendicular to d
d = the longest plan dimension of the structure

EXCEPTION: The total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, are permitted to be taken as less than the value prescribed by Eqs. 17.5-5 and 17.5-6, respectively, but not less than 1.1 times D_D and D_M, respectively, provided the isolation system is shown by calculation to be configured to resist torsion accordingly.

17.5.4 Minimum Lateral Forces

17.5.4.1 Isolation System and Structural Elements below the Isolation System

The isolation system, the foundation, and all structural elements below the isolation system shall be designed and constructed to withstand a minimum lateral seismic force, V_b, using all of the appropriate requirements for a nonisolated structure and as prescribed by Eq. 17.5-7:

$$V_b = k_{D\max}D_D \qquad (17.5\text{-}7)$$

where

$k_{D\max}$ = maximum effective stiffness, in kips/in. (kN/mm), of the isolation system at the design displacement in the horizontal direction under consideration as prescribed by Eq. 17.8-3
D_D = design displacement, in in. (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Eq. 17.5-1

V_b shall not be taken as less than the maximum force in the isolation system at any displacement up to and including the design displacement.

17.5.4.2 Structural Elements above the Isolation System

The structure above the isolation system shall be designed and constructed to withstand a minimum shear force, V_s, using all of the appropriate requirements for a nonisolated structure and as prescribed by Eq. 17.5-8:

$$V_s = \frac{k_{D\max} D_D}{R_I} \tag{17.5-8}$$

where

$k_{D\max}$ = maximum effective stiffness, in kips/in. (kN/mm), of the isolation system at the design displacement in the horizontal direction under consideration
D_D = design displacement, in in. (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Eq. 17.5-1
R_I = numerical coefficient related to the type of seismic force-resisting system above the isolation system

The R_I factor shall be based on the type of seismic force-resisting system used for the structure above the isolation system and shall be three-eighths of the value of R given in Table 12.2-1, with a maximum value not greater than 2.0 and a minimum value not less than 1.0.

17.5.4.3 Limits on V_s

The value of V_s shall not be taken as less than the following:

1. The lateral seismic force required by Section 12.8 for a fixed-base structure of the same effective seismic weight, W, and a period equal to the isolated period, T_D.
2. The base shear corresponding to the factored design wind load.
3. The lateral seismic force required to fully activate the isolation system (e.g., the yield level of a softening system, the ultimate capacity of a sacrificial wind-restraint system, or the break-away friction level of a sliding system) multiplied by 1.5.

17.5.5 Vertical Distribution of Force

The shear force V_s shall be distributed over the height of the structure above the isolation interface using Eq. 17.5-9:

$$F_x = \frac{V_s w_x h_x}{\sum_{i=1}^{n} w_i h_i} \tag{17.5-9}$$

where

F_x = portion of V_s that is assigned to Level x
V_s = total lateral seismic design force or shear on elements above the isolation system as prescribed by Eq. 17.5-8
w_x = portion of W that is located at or assigned to Level x
h_x = height above the base of Level x

At each level designated as x, the force, F_x, shall be applied over the area of the structure in accordance with the mass distribution at the level.

17.5.6 Drift Limits

The maximum story drift of the structure above the isolation system shall not exceed $0.015h_{sx}$. The drift shall be calculated by Eq. 12.8-15 with C_d for the isolated structure equal to R_I as defined in Section 17.5.4.2.

17.6 DYNAMIC ANALYSIS PROCEDURES

17.6.1 General

Where dynamic analysis is used to design seismically isolated structures, the requirements of this section shall apply.

17.6.2 Modeling

The mathematical models of the isolated structure including the isolation system, the seismic force-resisting system, and other structural elements shall conform to Section 12.7.3 and to the requirements of Sections 17.6.2.1 and 17.6.2.2.

17.6.2.1 Isolation System

The isolation system shall be modeled using deformational characteristics developed and verified by test in accordance with the requirements of Section 17.5.2. The isolation system shall be modeled with sufficient detail to

a. Account for the spatial distribution of isolator units.
b. Calculate translation, in both horizontal directions, and torsion of the structure above the isolation

interface considering the most disadvantageous location of eccentric mass.

c. Assess overturning/uplift forces on individual isolator units.
d. Account for the effects of vertical load, bilateral load, and/or the rate of loading if the force-deflection properties of the isolation system are dependent on one or more of these attributes.

The total design displacement and total maximum displacement across the isolation system shall be calculated using a model of the isolated structure that incorporates the force-deflection characteristics of nonlinear elements of the isolation system and the seismic force-resisting system.

17.6.2.2 Isolated Structure

The maximum displacement of each floor and design forces and displacements in elements of the seismic force-resisting system are permitted to be calculated using a linear elastic model of the isolated structure provided that both of the following conditions are met:

1. Stiffness properties assumed for the nonlinear components of the isolation system are based on the maximum effective stiffness of the isolation system; and
2. All elements of the seismic force-resisting system of the structure above the isolation system remain elastic for the design earthquake.

Seismic force-resisting systems with elastic elements include, but are not limited to, irregular structural systems designed for a lateral force not less than 100 percent of V_s and regular structural systems designed for a lateral force not less than 80 percent of V_s, where V_s is determined in accordance with Section 17.5.4.2.

17.6.3 Description of Procedures

17.6.3.1 General

Response-spectrum and response-history procedures shall be performed in accordance with Section 12.9 and Chapter 16, and the requirements of this section.

17.6.3.2 Input Earthquake

The design earthquake ground motions shall be used to calculate the total design displacement of the isolation system and the lateral forces and displacements in the isolated structure. The maximum considered earthquake shall be used to calculate the total maximum displacement of the isolation system.

17.6.3.3 Response-Spectrum Procedure

Response-spectrum analysis shall be performed using a modal damping value for the fundamental mode in the direction of interest not greater than the effective damping of the isolation system or 30 percent of critical, whichever is less. Modal damping values for higher modes shall be selected consistent with those that would be appropriate for response-spectrum analysis of the structure above the isolation system assuming a fixed base.

Response-spectrum analysis used to determine the total design displacement and the total maximum displacement shall include simultaneous excitation of the model by 100 percent of the ground motion in the critical direction and 30 percent of the ground motion in the perpendicular, horizontal direction. The maximum displacement of the isolation system shall be calculated as the vectorial sum of the two orthogonal displacements.

The design shear at any story shall not be less than the story shear resulting from application of the story forces calculated using Eq. 17.5-9 and a value of V_s equal to the base shear obtained from the response-spectrum analysis in the direction of interest.

17.6.3.4 Response-History Procedure

Where a response-history procedure is performed, a suite of not fewer than three pairs of appropriate ground motions shall be used in the analysis; the ground motion pairs shall be selected and scaled in accordance with Section 17.3.2.

Each pair of ground motion components shall be applied simultaneously to the model considering the most disadvantageous location of eccentric mass. The maximum displacement of the isolation system shall be calculated from the vectorial sum of the two orthogonal displacements at each time step.

The parameters of interest shall be calculated for each ground motion used for the response-history analysis. If seven or more pairs of ground motions are used for the response-history analysis, the average value of the response parameter of interest is permitted to be used for design. If fewer than seven pairs of ground motions are used for analysis, the maximum value of the response parameter of interest shall be used for design.

17.6.4 Minimum Lateral Displacements and Forces

17.6.4.1 Isolation System and Structural Elements below the Isolation System

The isolation system, foundation, and all structural elements below the isolation system shall be designed using all of the appropriate requirements for a nonisolated structure and the forces obtained from the dynamic analysis without reduction, but the design lateral force shall not be taken as less than 90 percent of V_b determined in accordance as prescribed by Eq. 17.5-7.

The total design displacement of the isolation system shall not be taken as less than 90 percent of D_{TD} as specified by Section 17.5.3.5. The total maximum displacement of the isolation system shall not be taken as less than 80 percent of D_{TM} as prescribed by Section 17.5.3.5.

The limits on displacements specified by this section shall be evaluated using values of D_{TD} and D_{TM} determined in accordance with Section 17.5.5 except that D'_D is permitted to be used in lieu of D_D and D'_M is permitted to be used in lieu of D_M as prescribed in Eqs. 17.6-1 and 17.6-2:

$$D'_D = \frac{D_D}{\sqrt{1+(T/T_D)^2}} \quad (17.6\text{-}1)$$

$$D'_M = \frac{D_M}{\sqrt{1+(T/T_M)^2}} \quad (17.6\text{-}2)$$

where

D_D = design displacement, in in. (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Eq. 17.5-1

D_M = maximum displacement in in. (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Eq. 17.5-3

T = elastic, fixed-base period of the structure above the isolation system as determined by Section 12.8.2

T_D = effective period of seismically isolated structure in s, at the design displacement in the direction under consideration, as prescribed by Eq. 17.5-2

T_M = effective period, in s, of the seismically isolated structure, at the maximum displacement in the direction under consideration, as prescribed by Eq. 17.5-4

17.6.4.2 Structural Elements above the Isolation System

Subject to the procedure-specific limits of this section, structural elements above the isolation system shall be designed using the appropriate requirements for a nonisolated structure and the forces obtained from the dynamic analysis reduced by a factor of R_I as determined in accordance with Section 17.5.4.2. The design lateral shear force on the structure above the isolation system, if regular in configuration, shall not be taken as less than 80 percent of V_s, or less than the limits specified by Section 17.5.4.3.

EXCEPTION: The lateral shear force on the structure above the isolation system, if regular in configuration, is permitted to be taken as less than 80 percent, but shall not be less than 60 percent of V_s, where the response-history procedure is used for analysis of the seismically isolated structure.

The design lateral shear force on the structure above the isolation system, if irregular in configuration, shall not be taken as less than V_s or less than the limits specified by Section 17.5.4.3.

EXCEPTION: The design lateral shear force on the structure above the isolation system, if irregular in configuration, is permitted to be taken as less than 100 percent, but shall not be less than 80 percent of V_s, where the response-history procedure is used for analysis of the seismically isolated structure.

17.6.4.3 Scaling of Results

Where the factored lateral shear force on structural elements, determined using either response-spectrum or response-history procedure, is less than the minimum values prescribed by Sections 17.6.4.1 and 17.6.4.2, all response parameters, including member forces and moments, shall be adjusted upward proportionally.

17.6.4.4 Drift Limits

Maximum story drift corresponding to the design lateral force including displacement due to vertical deformation of the isolation system shall not exceed the following limits:

1. The maximum story drift of the structure above the isolation system calculated by response-spectrum analysis shall not exceed $0.015h_{sx}$.
2. The maximum story drift of the structure above the isolation system calculated by response-history analysis based on the force-deflection characteristics of nonlinear elements of the seismic force-resisting system shall not exceed $0.020h_{sx}$.

Drift shall be calculated using Eq. 12.8-15 with the C_d of the isolated structure equal to R_I as defined in Section 17.5.4.2.

The secondary effects of the maximum considered earthquake lateral displacement of the structure above the isolation system combined with gravity forces shall be investigated if the story drift ratio exceeds $0.010/R_I$.

17.7 DESIGN REVIEW

A design review of the isolation system and related test programs shall be performed by an independent engineering team including persons licensed in the appropriate disciplines and experienced in seismic analysis methods and the theory and application of seismic isolation. Isolation system design review shall include, but not be limited to, the following:

1. Review of site-specific seismic criteria including the development of site-specific spectra and ground motion histories and all other design criteria developed specifically for the project.
2. Review of the preliminary design including the determination of the total design displacement, the total maximum displacement, and the lateral force level.
3. Overview and observation of prototype testing (Section 17.8).
4. Review of the final design of the entire structural system and all supporting analyses.
5. Review of the isolation system quality control testing program (Section 17.2.4.9).

17.8 TESTING

17.8.1 General

The deformation characteristics and damping values of the isolation system used in the design and analysis of seismically isolated structures shall be based on tests of a selected sample of the components prior to construction as described in this section.

The isolation system components to be tested shall include the wind-restraint system if such a system is used in the design.

The tests specified in this section are for establishing and validating the design properties of the isolation system and shall not be considered as satisfying the manufacturing quality control tests of Section 17.2.4.9.

17.8.2 Prototype Tests

Prototype tests shall be performed separately on two full-size specimens (or sets of specimens, as appropriate) of each predominant type and size of isolator unit of the isolation system. The test specimens shall include the wind-restraint system as well as individual isolator units if such systems are used in the design. Specimens tested shall not be used for construction unless accepted by the registered design professional responsible for the design of the structure and approved by the authority having jurisdiction.

17.8.2.1 Record

For each cycle of each test, the force-deflection and hysteretic behavior of the test specimen shall be recorded.

17.8.2.2 Sequence and Cycles

The following sequence of tests shall be performed for the prescribed number of cycles at a vertical load equal to the average dead load plus one-half the effects due to live load on all isolator units of a common type and size:

1. Twenty fully reversed cycles of loading at a lateral force corresponding to the wind design force.
2. Three fully reversed cycles of loading at each of the following increments of the total design displacement—$0.25D_D$, $0.5D_D$, $1.0D_D$, and $1.0D_M$ where D_D and D_M are as determined in Sections 17.5.3.1 and 17.5.3.3, respectively, or Section 17.6 as appropriate.
3. Three fully reversed cycles of loading at the total maximum displacement, $1.0D_{TM}$.
4. $30S_{D1}/S_{DS}B_D$, but not less than 10, fully reversed cycles of loading at 1.0 times the total design displacement, $1.0D_{TD}$.

If an isolator unit is also a vertical-load-carrying element, then item 2 of the sequence of cyclic tests specified in the preceding text shall be performed for two additional vertical load cases specified in Section 17.2.4.6. The load increment due to earthquake overturning, Q_E, shall be equal to or greater than the peak earthquake vertical force response corresponding to the test displacement being evaluated. In these tests, the combined vertical load shall be taken as the typical or average downward force on all isolator units of a common type and size.

17.8.2.3 Units Dependent on Loading Rates

If the force-deflection properties of the isolator units are dependent on the rate of loading, each set of tests specified in Section 17.8.2.2 shall be performed dynamically at a frequency equal to the inverse of the effective period, T_D.

If reduced-scale prototype specimens are used to quantify rate-dependent properties of isolators, the reduced-scale prototype specimens shall be of the same type and material and be manufactured with the same processes and quality as full-scale prototypes and shall be tested at a frequency that represents full-scale prototype loading rates.

The force-deflection properties of an isolator unit shall be considered to be dependent on the rate of loading if the measured property (effective stiffness or effective damping) at the design displacement when tested at any frequency in the range of 0.1 to 2.0 times the inverse of T_D is different from the property when tested at a frequency equal to the inverse of T_D by more than 15 percent.

17.8.2.4 Units Dependent on Bilateral Load

If the force-deflection properties of the isolator units are dependent on bilateral load, the tests specified in Sections 17.8.2.2 and 17.8.2.3 shall be augmented to include bilateral load at the following increments of the total design displacement, D_{TD}: 0.25 and 1.0, 0.5 and 1.0, 0.75 and 1.0, and 1.0 and 1.0

If reduced-scale prototype specimens are used to quantify bilateral-load-dependent properties, the reduced-scale specimens shall be of the same type and material and manufactured with the same processes and quality as full-scale prototypes.

The force-deflection properties of an isolator unit shall be considered to be dependent on bilateral load if the effective stiffness where subjected to bilateral loading is different from the effective stiffness where subjected to unilateral loading, by more than 15 percent.

17.8.2.5 Maximum and Minimum Vertical Load

Isolator units that carry vertical load shall be statically tested for maximum and minimum downward vertical load at the total maximum displacement. In these tests, the combined vertical loads shall be taken as specified in Section 17.2.4.6 on any one isolator of a common type and size. The dead load, D, and live load, L, are specified in Section 12.4. The seismic load E is given by Eqs. 12.4-1 and 12.4-2 where S_{DS} in these equations is replaced by S_{MS} and the vertical loads that result from application of horizontal seismic forces, Q_E, shall be based on the peak response due to the maximum considered earthquake.

17.8.2.6 Sacrificial Wind-Restraint Systems

If a sacrificial wind-restraint system is to be utilized, its ultimate capacity shall be established by test.

17.8.2.7 Testing Similar Units

Prototype tests are not required if an isolator unit is of similar size and of the same type and material as a prototype isolator unit that has been previously tested using the specified sequence of tests.

17.8.3 Determination of Force-Deflection Characteristics

The force-deflection characteristics of the isolation system shall be based on the cyclic load tests of prototype isolator specified in Section 17.8.2.

As required, the effective stiffness of an isolator unit, k_{eff}, shall be calculated for each cycle of loading as prescribed by Eq. 17.8-1:

$$k_{\text{eff}} = \frac{|F^+| + |F^-|}{|\Delta^+| + |\Delta^-|} \tag{17.8-1}$$

where F^+ and F^- are the positive and negative forces, at Δ^+ and Δ^-, respectively.

As required, the effective damping, β_{eff}, of an isolator unit shall be calculated for each cycle of loading by Eq. 17.8-2:

$$\beta_{\text{eff}} = \frac{2}{\pi} \frac{E_{\text{loop}}}{k_{\text{eff}} \left(|\Delta^+| + |\Delta^-|\right)^2} \tag{17.8-2}$$

where the energy dissipated per cycle of loading, E_{loop}, and the effective stiffness, k_{eff}, shall be based on peak test displacements of Δ^+ and Δ^-.

17.8.4 Test Specimen Adequacy

The performance of the test specimens shall be deemed adequate if the following conditions are satisfied:

1. The force-deflection plots for all tests specified in Section 17.8.2 have a positive incremental force-resisting capacity.
2. For each increment of test displacement specified in item 2 of Section 17.8.2.2 and for each vertical load case specified in Section 17.8.2.2,
   a. For each test specimen, the difference between the effective stiffness at each of the three cycles of test and the average value of effective stiffness is no greater than 15 percent.
   b. For each cycle of test, the difference between effective stiffness of the two test specimens of a common type and size of the isolator unit and the average effective stiffness is no greater than 15 percent.
3. For each specimen there is no greater than a 20 percent change in the initial effective stiffness over the cycles of test specified in item 4 of Section 17.8.2.2.

4. For each specimen there is no greater than a 20 percent decrease in the initial effective damping over the cycles of test specified in item 4 of Section 17.8.2.2.
5. All specimens of vertical-load-carrying elements of the isolation system remain stable where tested in accordance with Section 17.8.2.5.

17.8.5 Design Properties of the Isolation System

17.8.5.1 Maximum and Minimum Effective Stiffness

At the design displacement, the maximum and minimum effective stiffness of the isolated system, $k_{D\max}$ and $k_{D\min}$, shall be based on the cyclic tests of item 2 of Section 17.8.2.2 and calculated using Eqs. 17.8-3 and 17.8-4:

$$k_{Dmax} = \frac{\sum|F_D^+|_{max} + \sum|F_D^-|_{max}}{2D_D} \qquad (17.8\text{-}3)$$

$$k_{D\min} = \frac{\sum|F_D^+|_{\min} + \sum|F_D^-|_{\min}}{2D_D} \qquad (17.8\text{-}4)$$

At the maximum displacement, the maximum and minimum effective stiffness of the isolation system, $k_{M\max}$ and $k_{M\min}$, shall be based on the cyclic tests of item 3 of Section 17.8.2.2 and calculated using Eqs. 17.8-5 and 17.8-6:

$$k_{M\max} = \frac{\sum|F_M^+|_{\max} + \sum|F_M^-|_{\max}}{2D_M} \qquad (17.8\text{-}5)$$

$$k_{M\min} = \frac{\sum|F_M^+|_{\min} + |F_M^-|_{\min}}{2D_M} \qquad (17.8\text{-}6)$$

The maximum effective stiffness of the isolation system, $k_{D\max}$ (or $k_{M\max}$), shall be based on forces from the cycle of prototype testing at a test displacement equal to D_D (or D_M) that produces the largest value of effective stiffness. Minimum effective stiffness of the isolation system, $k_{D\min}$ (or $k_{M\min}$), shall be based on forces from the cycle of prototype testing at a test displacement equal to D_D (or D_M) that produces the smallest value of effective stiffness.

For isolator units that are found by the tests of Sections 17.8.2.2, 17.8.2.3, and 17.8.2.4 to have force-deflection characteristics that vary with vertical load, rate of loading, or bilateral load, respectively, the values of $k_{D\max}$ and $k_{M\max}$ shall be increased and the values of $k_{D\min}$ and $k_{M\min}$ shall be decreased, as necessary, to bound the effects of measured variation in effective stiffness.

17.8.5.2 Effective Damping

At the design displacement, the effective damping of the isolation system, β_D, shall be based on the cyclic tests of item 2 of Section 17.8.2.2 and calculated using Eq. 17.8-7:

$$\beta_D = \frac{\sum E_D}{2\pi k_{D\max} D_D^2} \qquad (17.8\text{-}7)$$

In Eq. 17.8-7, the total energy dissipated per cycle of design displacement response, ΣE_D, shall be taken as the sum of the energy dissipated per cycle in all isolator units measured at a test displacement equal to D_D and shall be based on forces and deflections from the cycle of prototype testing at test displacement D_D that produces the smallest values of effective damping.

At the maximum displacement, the effective damping of the isolation system, β_M, shall be based on the cyclic tests of item 2 of Section 17.8.2.2 and calculated using Eq. 17.8-8

$$\beta_M = \frac{\sum E_M}{2\pi k_{M\max} D_M^2} \qquad (17.8\text{-}8)$$

In Eq. 17.8-8, the total energy dissipated per cycle of design displacement response, ΣE_M, shall be taken as the sum of the energy dissipated per cycle in all isolator units measured at a test displacement equal to D_M and shall be based on forces and deflections from the cycle of prototype testing at test displacement D_M that produces the smallest value of effective damping.

Chapter 18

SEISMIC DESIGN REQUIREMENTS FOR STRUCTURES WITH DAMPING SYSTEMS

18.1 GENERAL

18.1.1 Scope

Every structure with a damping system and every portion thereof shall be designed and constructed in accordance with the requirements of this standard as modified by this section. Where damping devices are used across the isolation interface of a seismically isolated structure, displacements, velocities, and accelerations shall be determined in accordance with Chapter 17.

18.1.2 Definitions

The following definitions apply to the provisions of Chapter 18:

DAMPING DEVICE: A flexible structural element of the damping system that dissipates energy due to relative motion of each end of the device. Damping devices include all pins, bolts, gusset plates, brace extensions, and other components required to connect damping devices to the other elements of the structure. Damping devices may be classified as either displacement-dependent or velocity-dependent, or a combination thereof, and may be configured to act in either a linear or nonlinear manner.

DAMPING SYSTEM: The collection of structural elements that includes all the individual damping devices, all structural elements or bracing required to transfer forces from damping devices to the base of the structure, and the structural elements required to transfer forces from damping devices to the seismic force-resisting system.

DISPLACEMENT-DEPENDENT DAMPING DEVICE: The force response of a displacement-dependent damping device is primarily a function of the relative displacement between each end of the device. The response is substantially independent of the relative velocity between each of the devices and/or the excitation frequency.

VELOCITY-DEPENDENT DAMPING DEVICE: The force-displacement relation for a velocity-dependent damping device is primarily a function of the relative velocity between each end of the device and could also be a function of the relative displacement between each end of the device.

18.1.3 Notation

The following notations apply to the provisions of this chapter:

B_{1D} = numerical coefficient as set forth in Table 18.6-1 for effective damping equal to β_{ml} (m = 1) and period of structure equal to T_{1D}

B_{1E} = numerical coefficient as set forth in Table 18.6-1 for the effective damping equal to $\beta_I + \beta_{V1}$ and period equal to T_1

B_{1M} = numerical coefficient as set forth in Table 18.6-1 for effective damping equal to β_{mM} (m = 1) and period of structure equal to T_{1M}

B_{mD} = numerical coefficient as set forth in Table 18.6-1 for effective damping equal to β_{ml} and period of structure equal to T_m

B_{mM} = numerical coefficient as set forth in Table 18.6-1 for effective damping equal to β_{mM} and period of structure equal to T_m

B_R = numerical coefficient as set forth in Table 18.6-1 for effective damping equal to β_R and period of structure equal to T_R

B_{V+I} = numerical coefficient as set forth in Table 18.6-1 for effective damping equal to the sum of viscous damping in the fundamental mode of vibration of the structure in the direction of interest, β_{Vm} (m = 1), plus inherent damping, β_I, and period of structure equal to T_1

C_{mFD} = force coefficient as set forth in Table 18.7-1

C_{mFV} = force coefficient as set forth in Table 18.7-2

C_{S1} = seismic response coefficient of the fundamental mode of vibration of the structure in the direction of interest, Section 18.4.2.4 or 18.5.2.4 (m = 1)

C_{Sm} = seismic response coefficient of the m^{th} mode of vibration of the structure in the direction of interest, Section 18.4.2.4 (m = 1) or Section 18.4.2.6 (m > 1)

C_{SR} = seismic response coefficient of the residual mode of vibration of the structure in the direction of interest, Section 18.5.2.8

D_{1D} = fundamental mode design displacement at the center rigidity of the roof level of the structure in the direction under consideration, Section 18.5.3.2

D_{1M} = fundamental mode maximum displacement at the center of rigidity of the roof level of the structure in the direction under consideration, Section 18.5.3.5

D_{mD} = design displacement at the center of rigidity of the roof level of the structure due to the m^{th} mode of vibration in the direction under consideration, Section 18.4.3.2

D_{mM} = maximum displacement at the center of rigidity of the roof level of the structure due to the m^{th} mode of vibration in the direction under consideration, Section 18.4.3.5

D_{RD} = residual mode design displacement at the center of rigidity of the roof level of the structure in the direction under consideration, Section 18.5.3.2

D_{RM} = residual mode maximum displacement at the center of rigidity of the roof level of the structure in the direction under consideration, Section 18.5.3.5

D_Y = displacement at the center of rigidity of the roof level of the structure at the effective yield point of the seismic force-resisting system, Section 18.6.3

f_i = lateral force at Level i of the structure distributed approximately in accordance with Section 12.8.3, Section 18.5.2.3

F_{i1} = inertial force at Level i (or mass point i) in the fundamental mode of vibration of the structure in the direction of interest, Section 18.5.2.9

F_{im} = inertial force at Level i (or mass point i) in the m^{th} mode of vibration of the structure in the direction of interest, Section 18.4.2.7

F_{iR} = inertial force at Level i (or mass point i) in the residual mode of vibration of the structure in the direction of interest, Section 18.5.2.9

h_r = height of the structure above the base to the roof level, Section 18.5.2.3

q_H = hysteresis loop adjustment factor as determined in Section 18.6.2.2.1

Q_{DSD} = force in an element of the damping system required to resist design seismic forces of displacement-dependent damping devices, Section 18.7.2.5

Q_{mDSV} = forces in an element of the damping system required to resist design seismic forces of velocity-dependent damping devices due to the m^{th} mode of vibration of the structure in the direction of interest, Section 18.7.2.5

Q_{mSFRS} = force in an element of the damping system equal to the design seismic force of the m^{th} mode of vibration of the structure in the direction of interest, Section 18.7.2.5

T_1 = the fundamental period of the structure in the direction under consideration

T_{1D} = effective period, in seconds, of the fundamental mode of vibration of the structure at the design displacement in the direction under consideration, as prescribed by Section 18.4.2.5 or 18.5.2.5

T_{1M} = effective period, in seconds, of the fundamental mode of vibration of the structure at the maximum displacement in the direction under consideration, as prescribed by Section 18.4.2.5 or 18.5.2.5

T_R = period, in seconds, of the residual mode of vibration of the structure in the direction under consideration, Section 18.5.2.7

V_m = design value of the seismic base shear of the m^{th} mode of vibration of the structure in the direction of interest, Section 18.4.2.2

$V_{\min}$ = minimum allowable value of base shear permitted for design of the seismic force-resisting system of the structure in the direction of interest, Section 18.2.2.1

V_R = design value of the seismic base shear of the residual mode of vibration of the structure in a given direction, as determined in Section 18.5.2.6

$\bar{W}_1$ = effective fundamental mode seismic weight determined in accordance with Eq. 18.4-2b for $m = 1$

$\bar{W}_R$ = effective residual mode seismic weight determined in accordance with Eq. 18.5-13

α = velocity exponent relating damping device force to damping device velocity

β_{mD} = total effective damping of the m^{th} mode of vibration of the structure in the direction of interest at the design displacement, Section 18.6.2

β_{mM} = total effective damping of the m^{th} mode of vibration of the structure in the direction of interest at the maximum displacement, Section 18.6.2

β_{HD} = component of effective damping of the structure in the direction of interest due to post-yield hysteretic behavior of the seismic force-resisting system and elements of the damping system at effective ductility demand μ_D, Section 18.6.2.2

β_{HM} = component of effective damping of the structure in the direction of interest due to post-yield hysteretic behavior of the seismic force-resisting system and elements of the damping system at effective ductility demand, μ_M, Section 18.6.2.2

β_I = component of effective damping of the structure due to the inherent dissipation of energy by elements of the structure, at or just below the effective yield displacement of the seismic force-resisting system, Section 18.6.2.1

β_R = total effective damping in the residual mode of vibration of the structure in the direction of interest, calculated in accordance with Section 18.6.2 (using $\mu_D = 1.0$ and $\mu_M = 1.0$)

β_{Vm} = component of effective damping of the m^{th} mode of vibration of the structure in the direction of interest due to viscous dissipation of energy by the damping system, at or just below the effective yield displacement of the seismic force-resisting system, Section 18.6.2.3

δ_i = elastic deflection of Level i of the structure due to applied lateral force, f_i, Section 18.5.2.3

δ_{i1D} = fundamental mode design deflection of Level i at the center of rigidity of the structure in the direction under consideration, Section 18.5.3.1

δ_{iD} = total design deflection of Level i at the center of rigidity of the structure in the direction under consideration, Section 18.5.3

δ_{iM} = total maximum deflection of Level i at the center of rigidity of the structure in the direction under consideration, Section 18.5.3

δ_{iRD} = residual mode design deflection of Level i at the center of rigidity of the structure in the direction under consideration, Section 18.5.3.1

δ_{im} = deflection of Level i in the m^{th} mode of vibration at the center of rigidity of the structure in the direction under consideration, Section 18.6.2.3

Δ_{1D} = design story drift due to the fundamental mode of vibration of the structure in the direction of interest, Section 18.5.3.3

Δ_D = total design story drift of the structure in the direction of interest, Section 18.5.3.3

Δ_M = total maximum story drift of the structure in the direction of interest, Section 18.5.3

Δ_{mD} = design story drift due to the m^{th} mode of vibration of the structure in the direction of interest, Section 18.4.3.3

Δ_{RD} = design story drift due to the residual mode of vibration of the structure in the direction of interest, Section 18.5.3.3

μ = effective ductility demand on the seismic force-resisting system in the direction of interest

μ_D = effective ductility demand on the seismic force-resisting system in the direction of interest due to the design earthquake ground motions, Section 18.6.3

μ_M = effective ductility demand on the seismic force-resisting system in the direction of interest due to the maximum considered earthquake ground motions, Section 18.6.3

μ_{max} = maximum allowable effective ductility demand on the seismic force-resisting system due to the design earthquake ground motions, Section 18.6.4

ϕ_{i1} = displacement amplitude at Level i of the fundamental mode of vibration of the structure in the direction of interest, normalized to unity at the roof level, Section 18.5.2.3

ϕ_{iR} = displacement amplitude at Level i of the residual mode of vibration of the structure in the direction of interest normalized to unity at the roof level, Section 18.5.2.7

Γ_1 = participation factor of the fundamental mode of vibration of the structure in the direction of interest, Section 18.4.2.3 or 18.5.2.3 ($m = 1$)

Γ_m = participation factor in the m^{th} mode of vibration of the structure in the direction of interest, Section 18.4.2.3

Γ_R = participation factor of the residual mode of vibration of the structure in the direction of interest, Section 18.5.2.7

∇_{1D} = design story velocity due to the fundamental mode of vibration of the structure in the direction of interest, Section 18.5.3.4

∇_D = total design story velocity of the structure in the direction of interest, Section 18.4.3.4

∇_M = total maximum story velocity of the structure in the direction of interest, Section 18.5.3

∇_{mD} = design story velocity due to the m^{th} mode of vibration of the structure in the direction of interest, Section 18.4.3.4

18.2 GENERAL DESIGN REQUIREMENTS

18.2.1 Seismic Design Category A

Seismic Design Category A structures with a damping system shall be designed using the design spectral response acceleration determined in accordance with Section 11.4.4 and the analysis methods and design requirements for Seismic Design Category B structures.

18.2.2 System Requirements

Design of the structure shall consider the basic requirements for the seismic force-resisting system and the damping system as defined in the following sections. The seismic force-resisting system shall have the required strength to meet the forces defined in Section 18.2.2.1. The combination of the seismic force-resisting system and the damping system is permitted to be used to meet the drift requirement.

18.2.2.1 Seismic Force-Resisting System

Structures that contain a damping system are required to have a seismic force-resisting system that, in each lateral direction, conforms to one of the types indicated in Table 12.2-1.

The design of the seismic force-resisting system in each direction shall satisfy the requirements of Section 18.7 and the following:

1. The seismic base shear used for design of the seismic force-resisting system shall not be less than V_{min}, where V_{min} is determined as the greater of the values computed using Eqs. 18.2-1 and 18.2-2:

$$V_{min} = \frac{V}{B_{V+I}} \qquad (18.2\text{-}1)$$

$$V_{min} = 0.75V \qquad (18.2\text{-}2)$$

where

V = seismic base shear in the direction of interest, determined in accordance with Section 12.8

B_{V+I} = numerical coefficient as set forth in Table 18.6-1 for effective damping equal to the sum of viscous damping in the fundamental mode of vibration of the structure in the direction of interest, β_{Vm} ($m = 1$), plus inherent damping, β_I, and period of structure equal to T_1

EXCEPTION: The seismic base shear used for design of the seismic force-resisting system shall not be taken as less than $1.0V$, if either of the following conditions apply:

a. In the direction of interest, the damping system has less than two damping devices on each floor level, configured to resist torsion.
b. The seismic force-resisting system has horizontal irregularity Type 1b (Table 12.3-1) or vertical irregularity Type 1b (Table 12.3-2).

2. Minimum strength requirements for elements of the seismic force-resisting system that are also elements of the damping system or are otherwise required to resist forces from damping devices shall meet the additional requirements of Section 18.7.2.

18.2.2.2 Damping System

Elements of the damping system shall be designed to remain elastic for design loads including unreduced seismic forces of damping devices as required in Section 18.7.2.1, unless it is shown by analysis or test that inelastic response of elements would not adversely affect damping system function and inelastic response is limited in accordance with the requirements of Section 18.7.2.6.

18.2.3 Ground Motion

18.2.3.1 Design Spectra

Spectra for the design earthquake ground motions and maximum considered earthquake ground motions developed in accordance with Section 17.3.1 shall be used for the design and analysis of a structure with a damping system. Site-specific design spectra shall be developed and used for design of a structure with a damping system if either of the following conditions apply:

1. The structure is located on a Class F site.
2. The structure is located at a site with S_1 greater than or equal to 0.6.

18.2.3.2 Ground Motion Histories

Ground motion histories for the design earthquake and the maximum considered earthquake developed in accordance with Section 17.3.2 shall be used for design and analysis of all structures with a damping system if either of the following conditions apply:

1. The structure is located at a site with S_1 greater than or equal to 0.6.
2. The damping system is explicitly modeled and analyzed using the response-history analysis method.

18.2.4 Procedure Selection

A structure with a damping system shall be designed using linear procedures, nonlinear procedures, or a combination of linear and nonlinear procedures, as permitted in this section.

Regardless of the analysis method used, the peak dynamic response of the structure and elements of the damping system shall be confirmed by using the nonlinear response-history procedure if the structure is located at a site with S_1 greater than or equal to 0.6.

18.2.4.1 Nonlinear Procedures

The nonlinear procedures of Section 18.3 are permitted to be used for design of all structures with damping systems.

18.2.4.2 Response-Spectrum Procedure

The response-spectrum procedure of Section 18.4 is permitted to be used for design of a structure with a damping system provided that

1. In the direction of interest, the damping system has at least two damping devices in each story, configured to resist torsion.
2. The total effective damping of the fundamental mode, β_{mD} ($m = 1$), of the structure in the direction of interest is not greater than 35 percent of critical.

18.2.4.3 Equivalent Lateral Force Procedure

The equivalent lateral force procedure of Section 18.5 is permitted to be used for design of a structure with a damping system provided that

1. In the direction of interest, the damping system has at least two damping devices in each story, configured to resist torsion.
2. The total effective damping of the fundamental mode, β_{mD} ($m = 1$), of the structure in the direction of interest is not greater than 35 percent of critical.
3. The seismic force-resisting system does not have horizontal irregularity Type 1a or 1b (Table 12.3-1) or vertical irregularity Type 1a, 1b, 2, or 3 (Table 12.3-2).
4. Floor diaphragms are rigid as defined in Section 12.3.1.
5. The height of the structure above the base does not exceed 100 ft (30 m).

18.2.5 Damping System

18.2.5.1 Device Design

The design, construction, and installation of damping devices shall be based on response to maximum considered earthquake ground motions and consideration of the following:

1. Low-cycle, large-displacement degradation due to seismic loads.
2. High-cycle, small-displacement degradation due to wind, thermal, or other cyclic loads.
3. Forces or displacements due to gravity loads.
4. Adhesion of device parts due to corrosion or abrasion, biodegradation, moisture, or chemical exposure.
5. Exposure to environmental conditions, including, but not limited to, temperature, humidity, moisture, radiation (e.g., ultraviolet light), and reactive or corrosive substances (e.g., salt water).

Damping devices subject to failure by low-cycle fatigue shall resist wind forces without slip, movement, or inelastic cycling.

The design of damping devices shall incorporate the range of thermal conditions, device wear, manufacturing tolerances, and other effects that cause device properties to vary during the design life of the device.

18.2.5.2 Multiaxis Movement

Connection points of damping devices shall provide sufficient articulation to accommodate simultaneous longitudinal, lateral, and vertical displacements of the damping system.

18.2.5.3 Inspection and Periodic Testing

Means of access for inspection and removal of all damping devices shall be provided.

The registered design professional responsible for design of the structure shall establish an appropriate inspection and testing schedule for each type of damping device to ensure that the devices respond in a dependable manner throughout their design life. The degree of inspection and testing shall reflect the established in-service history of the damping devices and the likelihood of change in properties over the design life of the devices.

18.2.5.4 Quality Control

As part of the quality assurance plan developed in accordance with Section 11A.1.2, the registered design professional responsible for the structural design shall establish a quality control plan for the manufacture of damping devices. As a minimum, this plan shall include the testing requirements of Section 18.9.2.

18.3 NONLINEAR PROCEDURES

The stiffness and damping properties of the damping devices used in the models shall be based on or verified by testing of the damping devices as specified in Section 18.9. The nonlinear force-deflection characteristics of damping devices shall be modeled, as required, to explicitly account for device dependence on frequency, amplitude, and duration of seismic loading.

18.3.1 Nonlinear Response-History Procedure

A nonlinear response-history analysis shall utilize a mathematical model of the structure and the damping system as provided in Section 16.2.2 and this section. The model shall directly account for the nonlinear hysteretic behavior of elements of the structure and the damping devices to determine its response.

The analysis shall be performed in accordance with Section 16.2 together with the requirements of this section. Inherent damping of the structure shall not be taken as greater than 5 percent of critical unless test data consistent with levels of deformation at or just below the effective yield displacement of the seismic force-resisting system support higher values.

If the calculated force in an element of the seismic force-resisting system does not exceed 1.5 times its nominal strength, that element is permitted to be modeled as linear.

18.3.1.1 Damping Device Modeling

Mathematical models of displacement-dependent damping devices shall include the hysteretic behavior of the devices consistent with test data and accounting for all significant changes in strength, stiffness, and hysteretic loop shape. Mathematical models of velocity-dependent damping devices shall include the velocity coefficient consistent with test data. If this coefficient changes with time and/or temperature, such behavior shall be modeled explicitly. The elements of damping devices connecting damper units to the structure shall be included in the model.

EXCEPTION: If the properties of the damping devices are expected to change during the duration of the time history analysis, the dynamic response is permitted to be enveloped by the upper and lower limits of device properties. All these limit cases for variable device properties must satisfy the same conditions as if the time-dependent behavior of the devices were explicitly modeled.

18.3.1.2 Response Parameters

In addition to the response parameters given in Section 16.2.4, for each ground motion used for response-history analysis, individual response parameters consisting of the maximum value of the discrete damping device forces, displacements, and velocities, in the case of velocity-dependent devices, shall be determined.

If at least seven pairs of ground motions are used for response-history analysis, the design values of the damping device forces, displacements, and velocities are permitted to be taken as the average of the values determined by the analyses. If less than seven pairs of ground motions are used for response-history analysis, the design damping device forces, displacements, and velocities shall be taken as the maximum value determined by the analyses. A minimum of three pairs of ground motions shall be used.

18.3.2 Nonlinear Static Procedure

The nonlinear modeling described in Section 16.2.2 and the lateral loads described in Section 16.2 shall be applied to the seismic force-resisting system. The resulting force-displacement curve shall be used in lieu of the assumed effective yield displacement, D_Y, of Eq. 18.6-10 to calculate the effective ductility demand due to the design earthquake ground motions, μ_D, and due to the maximum considered earthquake ground motions, μ_M, in Eqs. 18.6-8 and 18.6-9, respectively. The value of (R/C_d) shall be taken as 1.0 in Eqs. 18.4-4, 18.4-5, 18.4-8, and 18.4-9 for the response-spectrum procedure, and in Eqs. 18.5-6, 18.5-7, and 18.5-15 for the equivalent lateral force procedure.

18.4 RESPONSE-SPECTRUM PROCEDURE

Where the response-spectrum procedure is used to analyze a structure with a damping system, the requirements of this section shall apply.

18.4.1 Modeling

A mathematical model of the seismic force-resisting system and damping system shall be constructed that represents the spatial distribution of mass, stiffness, and damping throughout the structure. The model and analysis shall comply with the requirements of Section 12.9 for the seismic force-resisting system and to the requirements of this section for the damping system. The stiffness and damping properties of the damping devices used in the models shall be based on or verified by testing of the damping devices as specified in Section 18.9.

The elastic stiffness of elements of the damping system other than damping devices shall be explicitly modeled. Stiffness of damping devices shall be modeled depending on damping device type as follows:

1. Displacement-dependent damping devices: Displacement-dependent damping devices shall be modeled with an effective stiffness that represents damping device force at the response displacement of interest (e.g., design story drift). Alternatively, the stiffness of hysteretic and friction damping devices is permitted to be excluded from response spectrum analysis provided design forces in displacement-dependent damping devices, Q_{DSD}, are applied to the model as external loads (Section 18.7.2.5).
2. Velocity-dependent damping devices: Velocity-dependent damping devices that have a stiffness component (e.g., viscoelastic damping devices) shall be modeled with an effective stiffness corresponding to the amplitude and frequency of interest.

18.4.2 Seismic Force-Resisting System

18.4.2.1 Seismic Base Shear

The seismic base shear, V, of the structure in a given direction shall be determined as the combination of modal components, V_m, subject to the limits of Eq. 18.4-1:

$$V \geq V_{\min} \tag{18.4-1}$$

The seismic base shear, V, of the structure shall be determined by the sum of the square root method (SRSS) or complete quadratic combination of modal base shear components, V_m.

18.4.2.2 Modal Base Shear

Modal base shear of the m^{th} mode of vibration, V_m, of the structure in the direction of interest shall be determined in accordance with Eqs. 18.4-2:

$$V_m = C_{sm}\bar{W} \tag{18.4-2a}$$

$$\bar{W}_m = \frac{\left(\sum_{i=1}^{n} w_i \phi_{im}\right)^2}{\sum_{i=1}^{n} w_i \phi_{im}^2} \tag{18.4-2b}$$

where

C_{sm} = seismic response coefficient of the m^{th} mode of vibration of the structure in the direction of interest as determined from Section 18.4.2.4 (m = 1) or Section 18.4.2.6 (m > 1)

$\bar{W}_m$ = effective seismic weight of the m^{th} mode of vibration of the structure

18.4.2.3 Modal Participation Factor

The modal participation factor of the m^{th} mode of vibration, Γ_m, of the structure in the direction of interest shall be determined in accordance with Eq. 18.4-3:

$$\Gamma_m = \frac{\bar{W}_m}{\sum_{i=1}^{n} w_i \phi_{im}} \tag{18.4-3}$$

where

ϕ_{im} = displacement amplitude at the i^{th} level of the structure in the m^{th} mode of vibration in the direction of interest, normalized to unity at the roof level.

18.4.2.4 Fundamental Mode Seismic Response Coefficient

The fundamental mode (m = 1) seismic response coefficient, C_{S1}, in the direction of interest shall be determined in accordance with Eqs. 18.4-4 and 18.4-5:

For $T_{1D} < T_S$,

$$C_{S1} = \left(\frac{R}{C_d}\right)\frac{S_{DS}}{\Omega_0 B_{1D}} \tag{18.4-4}$$

For $T_{1D} \geq T_S$,

$$C_{S1} = \left(\frac{R}{C_d}\right)\frac{S_{D1}}{T_{1D}\left(\Omega_0 B_{1D}\right)} \tag{18.4-5}$$

18.4.2.5 Effective Fundamental Mode Period Determination

The effective fundamental mode (m = 1) period at the design earthquake ground motion, T_{1D}, and at the MCE_R ground motion, T_{1M}, shall be based on either explicit consideration of the post-yield force deflection characteristics of the structure or determined in accordance with Eqs. 18.4-6 and 18.4-7:

$$T_{1D} = T_1\sqrt{\mu_D} \tag{18.4-6}$$

$$T_{1M} = T_1\sqrt{\mu_M} \tag{18.4-7}$$

18.4.2.6 Higher Mode Seismic Response Coefficient

Higher mode ($m > 1$) seismic response coefficient, C_{Sm}, of the m^{th} mode of vibration ($m > 1$) of the structure in the direction of interest shall be determined in accordance with Eqs. 18.4-8 and 18.4-9:

For $T_m < T_S$,

$$C_{Sm} = \left(\frac{R}{C_d}\right)\frac{S_{D1}}{\Omega_0 B_{mD}} \quad (18.4\text{-}8)$$

For $T_m \geq T_S$,

$$C_{Sm} = \left(\frac{R}{C_d}\right)\frac{S_{D1}}{T_m\,(\Omega_0 B_{mD})} \quad (18.4\text{-}9)$$

where

T_m = period, in seconds, of the m^{th} mode of vibration of the structure in the direction under consideration

B_{mD} = numerical coefficient as set forth in Table 18.6-1 for effective damping equal to β_{mD} and period of the structure equal to T_m

18.4.2.7 Design Lateral Force

Design lateral force at Level i due to the m^{th} mode of vibration, F_{im}, of the structure in the direction of interest shall be determined in accordance with Eq. 18.4-10:

$$F_{im} = w_i \phi_{im} \frac{\Gamma_m}{\bar{W}_m} V_m \quad (18.4\text{-}10)$$

Design forces in elements of the seismic force-resisting system shall be determined by the SRSS or complete quadratic combination of modal design forces.

18.4.3 Damping System

Design forces in damping devices and other elements of the damping system shall be determined on the basis of the floor deflection, story drift, and story velocity response parameters described in the following sections.

Displacements and velocities used to determine maximum forces in damping devices at each story shall account for the angle of orientation of each device from the horizontal and consider the effects of increased response due to torsion required for design of the seismic force-resisting system.

Floor deflections at Level i, δ_{iD} and δ_{iM}, story drifts, Δ_D and Δ_M, and story velocities, ∇_D and ∇_M, shall be calculated for both the design earthquake ground motions and the maximum considered earthquake ground motions, respectively, in accordance with this section.

18.4.3.1 Design Earthquake Floor Deflection

The deflection of structure due to the design earthquake ground motions at Level i in the m^{th} mode of vibration, δ_{imD}, of the structure in the direction of interest shall be determined in accordance with Eq. 18.4-11:

$$\delta_{imD} = D_{mD}\phi_{im} \quad (18.4\text{-}11)$$

The total design deflection at each floor of the structure shall be calculated by the SRSS or complete quadratic combination of modal design earthquake deflections.

18.4.3.2 Design Earthquake Roof Displacement

Fundamental ($m = 1$) and higher mode ($m > 1$) roof displacements due to the design earthquake ground motions, D_{1D} and D_{mD}, of the structure in the direction of interest shall be determined in accordance with Eqs. 18.4-12 and to 18.4-13:

For $m = 1$,

$$D_{1D} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{DS}T_{1D}^2}{B_{1D}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{DS}T_1^2}{B_{1E}},\ T_{1D} < T_S \quad (18.4\text{-}12a)$$

$$D_{1D} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{D1}T_{1D}}{B_{1D}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{D1}T_1}{B_{1E}},\ T_{1D} \geq T_S \quad (18.4\text{-}12b)$$

For $m > 1$,

$$D_{mD} = \left(\frac{g}{4\pi^2}\right)\Gamma_m \frac{S_{D1}T_m}{B_{mD}} \leq \left(\frac{g}{4\pi^2}\right)\Gamma_m \frac{S_{DS}T_m^2}{B_{mD}} \quad (18.4\text{-}13)$$

18.4.3.3 Design Earthquake Story Drift

Design story drift in the fundamental mode, Δ_{1D}, and higher modes, Δ_{mD} ($m > 1$), of the structure in the direction of interest shall be calculated in accordance with Section 12.8.6 using modal roof displacements of Section 18.4.3.2.

Total design story drift, Δ_D, shall be determined by the SRSS or complete quadratic combination of modal design earthquake drifts.

18.4.3.4 Design Earthquake Story Velocity

Design story velocity in the fundamental mode, ∇_{1D}, and higher modes, ∇_{mD} ($m > 1$), of the structure in the direction of interest shall be calculated in accordance with Eqs. 18.4-14 and 18.4-15:

$$\text{For } m = 1,\ \nabla_{1D} = 2\pi\frac{\Delta_{1D}}{T_{1D}} \quad (18.4\text{-}14)$$

$$\text{For } m > 1,\ \nabla_{mD} = 2\pi\frac{\Delta_{mD}}{T_m} \quad (18.4\text{-}15)$$

Total design story velocity, Δ_D, shall be determined by the SRSS or complete quadratic combination of modal design velocities.

18.4.3.5 Maximum Considered Earthquake Response

Total modal maximum floor deflection at Level i, design story drift values, and design story velocity values shall be based on Sections 18.4.3.1, 18.4.3.3, and 18.4.3.4, respectively, except design roof displacement shall be replaced by maximum roof displacement. Maximum roof displacement of the structure in the direction of interest shall be calculated in accordance with Eqs. 18.4-16 and to 18.4-17:

For $m = 1$,

$$D_{1M} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{MS}T_{1M}^2}{B_{1M}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{MS}T_1^2}{B_{1E}},\ T_{1M} < T_S \quad (18.4\text{-}16a)$$

$$D_{1M} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{M1}T_{1M}}{B_{1M}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{M1}T_1}{B_{1E}},\ T_{1M} \geq T_S \quad (18.4\text{-}16b)$$

For $m > 1$,

$$D_{mM} = \left(\frac{g}{4\pi^2}\right)\Gamma_m \frac{S_{M1}T_m}{B_{mM}} \leq \left(\frac{g}{4\pi^2}\right)\Gamma_m \frac{S_{MS}T_m^2}{B_{mM}} \quad (18.4\text{-}17)$$

where

B_{mM} = numerical coefficient as set forth in Table 18.6-1 for effective damping equal to β_{mM} and period of the structure equal to T_m

18.5 EQUIVALENT LATERAL FORCE PROCEDURE

Where the equivalent lateral force procedure is used to design structures with a damping system, the requirements of this section shall apply.

18.5.1 Modeling

Elements of the seismic force-resisting system shall be modeled in a manner consistent with the requirements of Section 12.8. For purposes of analysis, the structure shall be considered to be fixed at the base.

Elements of the damping system shall be modeled as required to determine design forces transferred from damping devices to both the ground and the seismic force-resisting system. The effective stiffness of velocity-dependent damping devices shall be modeled.

Damping devices need not be explicitly modeled provided effective damping is calculated in accordance with the procedures of Section 18.6 and used to modify response as required in Sections 18.5.2 and 18.5.3.

The stiffness and damping properties of the damping devices used in the models shall be based on or verified by testing of the damping devices as specified in Section 18.9.

18.5.2 Seismic Force-Resisting System

18.5.2.1 Seismic Base Shear

The seismic base shear, V, of the seismic force-resisting system in a given direction shall be determined as the combination of the two modal components, V_1 and V_R, in accordance with Eq. 18.5-1:

$$V = \sqrt{V_1^2 + V_R^2} \geq V_{min} \quad (18.5\text{-}1)$$

where

V_1 = design value of the seismic base shear of the fundamental mode in a given direction of response, as determined in Section 18.5.2.2

V_R = design value of the seismic base shear of the residual mode in a given direction, as determined in Section 18.5.2.6

V_{min} = minimum allowable value of base shear permitted for design of the seismic force-resisting system of the structure in direction of the interest, as determined in Section 18.2.2.1

18.5.2.2 Fundamental Mode Base Shear

The fundamental mode base shear, V_1, shall be determined in accordance with Eq. 18.5-2:

$$V_1 = C_{S1}\overline{W}_1 \quad (18.5\text{-}2)$$

where

C_{S1} = the fundamental mode seismic response coefficient, as determined in Section 18.5.2.4

$\overline{W}_1$ = the effective fundamental mode seismic weight including portions of the live load as defined by Eq. 18.4-2b for $m = 1$

18.5.2.3 Fundamental Mode Properties

The fundamental mode shape, ϕ_{i1}, and participation factor, Γ_1, shall be determined by either dynamic analysis using the elastic structural properties and deformational characteristics of the resisting elements or using Eqs. 18.5-3 and 18.5-4:

$$\phi_{i1} = \frac{h_i}{h_r} \quad (18.5\text{-}3)$$

$$\Gamma_1 = \frac{\bar{W}_1}{\sum_{i=1}^{n} w_i \phi_{i1}} \qquad (18.5\text{-}4)$$

where

h_i = the height above the base to Level i
h_r = the height of the structure above the base to the roof level
w_i = the portion of the total effective seismic weight, W, located at or assigned to Level i

The fundamental period, T_1, shall be determined either by dynamic analysis using the elastic structural properties and deformational characteristics of the resisting elements, or using Eq. 18.5-5 as follows:

$$T_1 = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i \delta_1^2}{g \sum_{i=1}^{n} f_i \delta_i}} \qquad (18.5\text{-}5)$$

where

f_i = lateral force at Level i of the structure distributed in accordance with Section 12.8.3
δ_i = elastic deflection at Level i of the structure due to applied lateral forces f_i

18.5.2.4 Fundamental Mode Seismic Response Coefficient

The fundamental mode seismic response coefficient, C_{S1}, shall be determined using Eq. 18.5-6 or 18.5-7:

For $T_{1D} < T_S$,

$$C_{S1} = \left(\frac{R}{C_d}\right)\frac{S_{D1}}{\Omega_0 B_{1D}} \qquad (18.5\text{-}6)$$

For $T_{1D} \geq T_S$,

$$C_{S1} = \left(\frac{R}{C_d}\right)\frac{S_{D1}}{T_{1D}\,(\Omega_0 B_{1D})} \qquad (18.5\text{-}7)$$

where

S_{DS} = the design spectral response acceleration parameter in the short period range
S_{D1} = the design spectral response acceleration parameter at a period of 1 s
B_{1D} = numerical coefficient as set forth in Table 18.6-1 for effective damping equal to β_{mD} (m = 1) and period of the structure equal to T_{1D}

18.5.2.5 Effective Fundamental Mode Period Determination

The effective fundamental mode period at the design earthquake, T_{1D}, and at the maximum considered earthquake, T_{1M}, shall be based on explicit consideration of the post-yield force deflection characteristics of the structure or shall be calculated using Eqs. 18.5-8 and 18.5-9:

$$T_{1D} = T_1\sqrt{\mu_D} \qquad (18.5\text{-}8)$$

$$T_{1M} = T_1\sqrt{\mu_M} \qquad (18.5\text{-}9)$$

18.5.2.6 Residual Mode Base Shear

Residual mode base shear, V_R, shall be determined in accordance with Eq. 18.5-10:

$$V_R = C_{SR}\bar{W}_R \qquad (18.5\text{-}10)$$

where

C_{SR} = the residual mode seismic response coefficient as determined in Section 18.5.2.8
$\bar{W}_R$ = the effective residual mode effective weight of the structure determined using Eq. 18.5-13

18.5.2.7 Residual Mode Properties

Residual mode shape, ϕ_{iR}, participation factor, Γ_R, effective residual mode seismic weight of the structure, $\bar{W}_R$, and effective period, T_R, shall be determined using Eqs. 18.5-11 through 18.5-14:

$$\phi_{iR} = \frac{1-\Gamma_1\phi_{i1}}{1-\Gamma_1} \qquad (18.5\text{-}11)$$

$$\Gamma_R = 1 - \Gamma_1 \qquad (18.5\text{-}12)$$

$$\bar{W}_R = \mathrm{W} - \bar{W}_1 \qquad (18.5\text{-}13)$$

$$T_R = 0.4T_1 \qquad (18.5\text{-}14)$$

18.5.2.8 Residual Mode Seismic Response Coefficient

The residual mode seismic response coefficient, C_{SR}, shall be determined in accordance with Eq. 18.5-15:

$$C_{SR} = \left(\frac{R}{C_d}\right)\frac{S_{DS}}{\Omega_0 B_R} \qquad (18.5\text{-}15)$$

where

B_R = numerical coefficient as set forth in Table 18.6-1 for effective damping equal to β_R, and period of the structure equal to T_R

18.5.2.9 Design Lateral Force

The design lateral force in elements of the seismic force-resisting system at Level i due to fundamental mode response, F_{i1}, and residual mode response, F_{iR}, of the structure in the direction of interest shall be determined in accordance with Eqs. 18.5-16 and 18.5-17:

$$F_{i1} = w_i \phi_{i1} \frac{\Gamma_1}{\bar{W}_1} V_1 \qquad (18.5\text{-}16)$$

$$F_{iR} = w_i \phi_{iR} \frac{\Gamma_R}{\bar{W}_R} V_R \qquad (18.5\text{-}17)$$

Design forces in elements of the seismic force-resisting system shall be determined by taking the SRSS of the forces due to fundamental and residual modes.

18.5.3 Damping System

Design forces in damping devices and other elements of the damping system shall be determined on the basis of the floor deflection, story drift, and story velocity response parameters described in the following sections.

Displacements and velocities used to determine maximum forces in damping devices at each story shall account for the angle of orientation of each device from the horizontal and consider the effects of increased response due to torsion required for design of the seismic force-resisting system.

Floor deflections at Level i, δ_{iD} and δ_{iM}, story drifts, Δ_D and Δ_M, and story velocities, ∇_D and ∇_M, shall be calculated for both the design earthquake ground motions and the maximum considered earthquake ground motions, respectively, in accordance with the following sections.

18.5.3.1 Design Earthquake Floor Deflection

The total design deflection at each floor of the structure in the direction of interest shall be calculated as the SRSS of the fundamental and residual mode floor deflections. The fundamental and residual mode deflections due to the design earthquake ground motions, δ_{i1D} and δ_{iRD}, at the center of rigidity of Level i of the structure in the direction of interest shall be determined using Eqs. 18.5-18 and 18.5-19:

$$\delta_{i1D} = D_{1D}\phi_{i1} \qquad (18.5\text{-}18)$$

$$\delta_{iRD} = D_{RD}\phi_{iR} \qquad (18.5\text{-}19)$$

where

D_{1D} = fundamental mode design displacement at the center of rigidity of the roof level of the structure in the direction under consideration, Section 18.5.3.2

D_{RD} = residual mode design displacement at the center of rigidity of the roof level of the structure in the direction under consideration, Section 18.5.3.2

18.5.3.2 Design Earthquake Roof Displacement

Fundamental and residual mode displacements due to the design earthquake ground motions, D_{1D} and D_{1R}, at the center of rigidity of the roof level of the structure in the direction of interest shall be determined using Eqs. 18.5-20 and 18.5-21:

$$D_{1D} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{DS}T_{1D}^2}{B_{1D}} \ge \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{DS}T_1^2}{B_{1D}},\ T_{1D} < T_S \qquad (18.5\text{-}20a)$$

$$D_{1D} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{D1}T_{1D}}{B_{1D}} \ge \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{D1}T_1}{B_{1E}},\ T_{1D} \ge T_S \qquad (18.5\text{-}20b)$$

$$D_{RD} = \left(\frac{g}{4\pi^2}\right)\Gamma_R \frac{S_{D1}T_R}{B_R} \le \left(\frac{g}{4\pi^2}\right)\Gamma_R \frac{S_{DS}T_R^2}{B_R} \qquad (18.5\text{-}21)$$

18.5.3.3 Design Earthquake Story Drift

Design story drifts, Δ_D, in the direction of interest shall be calculated using Eq. 18.5-22:

$$\Delta_D = \sqrt{\Delta_{1D}^2 + \Delta_{RD}^2} \qquad (18.5\text{-}22)$$

where

Δ_{1D} = design story drift due to the fundamental mode of vibration of the structure in the direction of interest

Δ_{RD} = design story drift due to the residual mode of vibration of the structure in the direction of interest

Modal design story drifts, Δ_{1D} and Δ_{RD}, shall be determined as the difference of the deflections at the top and bottom of the story under consideration using the floor deflections of Section 18.5.3.1.

18.5.3.4 Design Earthquake Story Velocity

Design story velocities, ∇_D, in the direction of interest shall be calculated in accordance with Eqs. 18.5-23 through 18.5-25:

$$\nabla_D = \sqrt{\nabla_{1D}^2 + \nabla_{RD}^2} \qquad (18.5\text{-}23)$$

$$\nabla_{1D} = 2\pi \frac{\Delta_{1D}}{T_{1D}} \qquad (18.5\text{-}24)$$

$$\nabla_{RD} = 2\pi \frac{\Delta_{RD}}{T_R} \quad (18.5\text{-}25)$$

where

∇_{1D} = design story velocity due to the fundamental mode of vibration of the structure in the direction of interest

∇_{RD} = design story velocity due to the residual mode of vibration of the structure in the direction of interest

18.5.3.5 Maximum Considered Earthquake Response

Total and modal maximum floor deflections at Level i, design story drifts, and design story velocities shall be based on the equations in Sections 18.5.3.1, 18.5.3.3, and 18.5.3.4, respectively, except that design roof displacements shall be replaced by maximum roof displacements. Maximum roof displacements shall be calculated in accordance with Eqs. 18.5-26 and 18.5-27:

$$D_{1M} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{MS}T_{1M}^2}{B_{1M}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{MS}T_1^2}{B_{1E}},\ T_{1M} < T_S \quad (18.5\text{-}26a)$$

$$D_{1M} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{M1}T_{1M}}{B_{1M}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{M1}T_1}{B_{1E}},\ T_{1M} \geq T_S \quad (18.5\text{-}26b)$$

$$D_{RM} = \left(\frac{g}{4\pi^2}\right)\Gamma_R \frac{S_{M1}T_R}{B_R} \leq \left(\frac{g}{4\pi^2}\right)\Gamma_R \frac{S_{MS}T_R^2}{B_R} \quad (18.5\text{-}27)$$

where

S_{M1} = the MCE$_R$, 5 percent damped, spectral response acceleration parameter at a period of 1 s adjusted for site class effects as defined in Section 11.4.3

S_{MS} = the MCE$_R$, 5 percent damped, spectral response acceleration parameter at short periods adjusted for site class effects as defined in Section 11.4.3

B_{1M} = numerical coefficient as set forth in Table 18.6-1 for effective damping equal to β_{mM} (m = 1) and period of structure equal to T_{1M}

18.6 DAMPED RESPONSE MODIFICATION

As required in Sections 18.4 and 18.5, response of the structure shall be modified for the effects of the damping system.

Table 18.6-1 Damping Coefficient, B_{V+I}, B_{1D}, B_R, B_{1M}, B_{mD}, B_{mM} (Where Period of the Structure ≥ T_0)

Effective Damping, β (percentage of critical)	B_{v+I}, B_{1D}, B_R, B_{1M}, B_{mD}, B_{mM} (where period of the structure ≥ T_0)
≤2	0.8
5	1.0
10	1.2
20	1.5
30	1.8
40	2.1
50	2.4
60	2.7
70	3.0
80	3.3
90	3.6
≥100	4.0

18.6.1 Damping Coefficient

Where the period of the structure is greater than or equal to T_0, the damping coefficient shall be as prescribed in Table 18.6-1. Where the period of the structure is less than T_0, the damping coefficient shall be linearly interpolated between a value of 1.0 at a 0-second period for all values of effective damping and the value at period T_0 as indicated in Table 18.6-1.

18.6.2 Effective Damping

The effective damping at the design displacement, β_{mD}, and at the maximum displacement, β_{mM}, of the m^{th} mode of vibration of the structure in the direction under consideration shall be calculated using Eqs. 18.6-1 and 18.6-2:

$$\beta_{mD} = \beta_I + \beta_{Vm}\sqrt{\mu_D} + \beta_{HD} \quad (18.6\text{-}1)$$

$$\beta_{mM} = \beta_I + \beta_{Vm}\sqrt{\mu_M} + \beta_{HM} \quad (18.6\text{-}2)$$

where

β_{HD} = component of effective damping of the structure in the direction of interest due to post-yield hysteretic behavior of the seismic force-resisting system and elements of the damping system at effective ductility demand, μ_D

β_{HM} = component of effective damping of the structure in the direction of interest due to post-yield hysteretic behavior of the seismic force-resisting system and elements of the damping system at effective ductility demand, μ_M

β_I = component of effective damping of the structure due to the inherent dissipation of energy by elements of the structure, at or just below the effective yield displacement of the seismic force-resisting system

β_{Vm} = component of effective damping of the m^{th} mode of vibration of the structure in the direction of interest due to viscous dissipation of energy by the damping system, at or just below the effective yield displacement of the seismic force-resisting system

μ_D = effective ductility demand on the seismic force-resisting system in the direction of interest due to the design earthquake ground motions

μ_M = effective ductility demand on the seismic force-resisting system in the direction of interest due to the maximum considered earthquake ground motions

Unless analysis or test data supports other values, the effective ductility demand of higher modes of vibration in the direction of interest shall be taken as 1.0.

18.6.2.1 Inherent Damping

Inherent damping, β_I, shall be based on the material type, configuration, and behavior of the structure and nonstructural components responding dynamically at or just below yield of the seismic force-resisting system. Unless analysis or test data supports other values, inherent damping shall be taken as not greater than 5 percent of critical for all modes of vibration.

18.6.2.2 Hysteretic Damping

Hysteretic damping of the seismic force-resisting system and elements of the damping system shall be based either on test or analysis or shall be calculated using Eqs. 18.6-3 and 18.6-4:

$$\beta_{HD} = q_H\,(0.64 - \beta_I)\left(1 - \frac{1}{\mu_D}\right) \qquad (18.6\text{-}3)$$

$$\beta_{HM} = q_H\,(0.64 - \beta_I)\left(1 - \frac{1}{\mu_M}\right) \qquad (18.6\text{-}4)$$

where

q_H = hysteresis loop adjustment factor, as defined in Section 18.6.2.2.1

μ_D = effective ductility demand on the seismic force-resisting system in the direction of interest due to the design earthquake ground motions

μ_M = effective ductility demand on the seismic force-resisting system in the direction of interest due to the maximum considered earthquake ground motions

Unless analysis or test data supports other values, the hysteretic damping of higher modes of vibration in the direction of interest shall be taken as zero.

18.6.2.2.1 Hysteresis Loop Adjustment Factor The calculation of hysteretic damping of the seismic force-resisting system and elements of the damping system shall consider pinching and other effects that reduce the area of the hysteresis loop during repeated cycles of earthquake demand. Unless analysis or test data support other values, the fraction of full hysteretic loop area of the seismic force-resisting system used for design shall be taken as equal to the factor, q_H, calculated using Eq. 18.6-5:

$$q_H = 0.67\frac{T_S}{T_1} \qquad (18.6\text{-}5)$$

where

T_S = period defined by the ratio, S_{D1}/S_{DS}

T_1 = period of the fundamental mode of vibration of the structure in the direction of the interest

The value of q_H shall not be taken as greater than 1.0 and need not be taken as less than 0.5.

18.6.2.3 Viscous Damping

Viscous damping of the m^{th} mode of vibration of the structure, β_{Vm}, shall be calculated using Eqs. 18.6-6 and 18.6-7:

$$\beta_{Vm} = \frac{\sum_j W_{mj}}{4\pi W_m} \qquad (18.6\text{-}6)$$

$$W_m = \frac{1}{2}\sum_j F_{im}\delta_{im} \qquad (18.6\text{-}7)$$

where

W_{mj} = work done by j^{th} damping device in one complete cycle of dynamic response corresponding to the m^{th} mode of vibration of the structure in the direction of interest at modal displacements, δ_{im}

W_m = maximum strain energy in the m^{th} mode of vibration of the structure in the direction of interest at modal displacements, δ_{im}

F_{im} = m^{th} mode inertial force at Level i

δ_{im} = deflection of Level i in the m^{th} mode of vibration at the center of rigidity of the structure in the direction under consideration

Viscous modal damping of displacement-dependent damping devices shall be based on a response amplitude equal to the effective yield displacement of the structure.

The calculation of the work done by individual damping devices shall consider orientation and participation of each device with respect to the mode of vibration of interest. The work done by individual damping devices shall be reduced as required to account for the flexibility of elements, including pins, bolts, gusset plates, brace extensions, and other components that connect damping devices to other elements of the structure.

18.6.3 Effective Ductility Demand

The effective ductility demand on the seismic force-resisting system due to the design earthquake ground motions, μ_D, and due to the maximum considered earthquake ground motions, μ_M, shall be calculated using Eqs. 18.6-8, 18.6-9, and 18.6-10:

$$\mu_D = \frac{D_{1D}}{D_Y} \geq 1.0 \qquad (18.6\text{-}8)$$

$$\mu_M = \frac{D_{1M}}{D_Y} \geq 1.0 \qquad (18.6\text{-}9)$$

$$D_Y = \left(\frac{g}{4\pi^2}\right)\left(\frac{\Omega_0 C_d}{R}\right)\Gamma_1 C_{S1} T_1^2 \qquad (18.6\text{-}10)$$

where

D_{1D} = fundamental mode design displacement at the center of rigidity of the roof level of the structure in the direction under consideration, Section 18.4.3.2 or 18.5.3.2

D_{1M} = fundamental mode maximum displacement at the center of rigidity of the roof level of the structure in the direction under consideration, Section 18.4.3.5 or 18.5.3.5

D_Y = displacement at the center of rigidity of the roof level of the structure at the effective yield point of the seismic force-resisting system

R = response modification coefficient from Table 12.2-1

C_d = deflection amplification factor from Table 12.2-1

Ω_0 = overstrength factor from Table 12.2-1

Γ_1 = participation factor of the fundamental mode of vibration of the structure in the direction of interest, Section 18.4.2.3 or 18.5.2.3 (m = 1)

C_{S1} = seismic response coefficient of the fundamental mode of vibration of the structure in the direction of interest, Section 18.4.2.4 or 18.5.2.4 (m = 1)

T_1 = period of the fundamental mode of vibration of the structure in the direction of interest

The design ductility demand, μ_D, shall not exceed the maximum value of effective ductility demand, μ_{max}, given in Section 18.6.4.

18.6.4 Maximum Effective Ductility Demand

For determination of the hysteresis loop adjustment factor, hysteretic damping, and other parameters, the maximum value of effective ductility demand, μ_{max}, shall be calculated using Eqs. 18.6-11 and 18.6-12:

For $T_{1D} \leq T_S$,

$$\mu_{max} = 0.5[(R/(\Omega_0 I_e))^2 + 1] \qquad (18.6\text{-}11)$$

For $T_1 \geq T_S$,

$$\mu_{max} = R/(\Omega_0 I_e) \qquad (18.6\text{-}12)$$

where

I_e = the importance factor determined in accordance with Section 11.5.1

T_{1D} = effective period of the fundamental mode of vibration of the structure at the design displacement in the direction under consideration

For $T_1 < T_S < T_{1D}$, μ_{max} shall be determined by linear interpolation between the values of Eqs. 18.6-11 and 18.6-12.

18.7 SEISMIC LOAD CONDITIONS AND ACCEPTANCE CRITERIA

For the nonlinear procedures of Section 18.3, the seismic force-resisting system, damping system, loading conditions, and acceptance criteria for response parameters of interest shall conform with Section 18.7.1. Design forces and displacements determined in accordance with the response-spectrum procedure of Section 18.4 or the equivalent lateral force procedure of Section 18.5 shall be checked using the strength design criteria of this standard and the seismic loading conditions of Section 18.7.1 and 18.7.2.

18.7.1 Nonlinear Procedures

Where nonlinear procedures are used in analysis, the seismic force-resisting system, damping system, seismic loading conditions, and acceptance criteria shall conform to the following subsections.

18.7.1.1 Seismic Force-Resisting System

The seismic force-resisting system shall satisfy the strength requirements of Section 12.2.1 using the seismic base shear, V_{min}, as given by Section 18.2.2.1. The story drift shall be determined using the design earthquake ground motions.

18.7.1.2 Damping Systems

The damping devices and their connections shall be sized to resist the forces, displacements, and velocities from the maximum considered earthquake ground motions.

18.7.1.3 Combination of Load Effects

The effects on the damping system due to gravity loads and seismic forces shall be combined in accordance with Section 12.4 using the effect of horizontal seismic forces, Q_E, determined in accordance with the analysis. The redundancy factor, ρ, shall be taken equal to 1.0 in all cases, and the seismic load effect with overstrength factor of Section 12.4.3 need not apply to the design of the damping system.

18.7.1.4 Acceptance Criteria for the Response Parameters of Interest

The damping system components shall be evaluated using the strength design criteria of this standard using the seismic forces and seismic loading conditions determined from the nonlinear procedures and $\phi = 1.0$. The members of the seismic force-resisting system need not be evaluated where using the nonlinear procedure forces.

18.7.2 Response-Spectrum and Equivalent Lateral Force Procedures

Where response-spectrum or equivalent lateral force procedures are used in analysis, the seismic force-resisting system, damping system, seismic loading conditions, and acceptance criteria shall conform to the following subsections.

18.7.2.1 Seismic Force-Resisting System

The seismic force-resisting system shall satisfy the requirements of Section 12.2.1 using seismic base shear and design forces determined in accordance with Section 18.4.2 or 18.5.2.

The design story drift, Δ_D, as determined in either Section 18.4.3.3 or 18.5.3.3 shall not exceed (R/C_d) times the allowable story drift, as obtained from Table 12.12-1, considering the effects of torsion as required in Section 12.12.1.

18.7.2.2 Damping System

The damping system shall satisfy the requirements of Section 12.2.1 for seismic design forces and seismic loading conditions determined in accordance with this section.

18.7.2.3 Combination of Load Effects

The effects on the damping system and its components due to gravity loads and seismic forces shall be combined in accordance with Section 12.4 using the effect of horizontal seismic forces, Q_E, determined in accordance with Section 18.7.2.5. The redundancy factor, ρ, shall be taken equal to 1.0 in all cases, and the seismic load effect with overstrength factor of Section 12.4.3 need not apply to the design of the damping system.

18.7.2.4 Modal Damping System Design Forces

Modal damping system design forces shall be calculated on the basis of the type of damping devices and the modal design story displacements and velocities determined in accordance with either Section 18.4.3 or 18.5.3.

Modal design story displacements and velocities shall be increased as required to envelop the total design story displacements and velocities determined in accordance with Section 18.3 where peak response is required to be confirmed by response-history analysis.

1. Displacement-dependent damping devices: Design seismic force in displacement-dependent damping devices shall be based on the maximum force in the device at displacements up to and including the design story drift, Δ_D.
2. Velocity-dependent damping devices: Design seismic force in each mode of vibration in velocity-dependent damping devices shall be based on the maximum force in the device at velocities up to and including the design story velocity for the mode of interest.

Displacements and velocities used to determine design forces in damping devices at each story shall account for the angle of orientation of the damping device from the horizontal and consider the effects of increased floor response due to torsional motions.

18.7.2.5 Seismic Load Conditions and Combination of Modal Responses

Seismic design force, Q_E, in each element of the damping system shall be taken as the maximum force of the following three loading conditions:

1. Stage of maximum displacement: Seismic design force at the stage of maximum displacement shall be calculated in accordance with Eq. 18.7-1:

$$Q_E = \Omega_0 \sqrt{\sum_m (Q_{mSFRS})^2} \pm Q_{DSD} \qquad (18.7\text{-}1)$$

where

Q_{mSFRS} = force in an element of the damping system equal to the design seismic force of the m^{th} mode of vibration of the structure in the direction of interest

Q_{DSD} = force in an element of the damping system required to resist design seismic forces of displacement-dependent damping devices

Seismic forces in elements of the damping system, Q_{DSD}, shall be calculated by imposing design forces of displacement-dependent damping devices on the damping system as pseudostatic forces. Design seismic forces of displacement-dependent damping devices shall be applied in both positive and negative directions at peak displacement of the structure.

2. Stage of maximum velocity: Seismic design force at the stage of maximum velocity shall be calculated in accordance with Eq. 18.7-2:

$$Q_E = \sqrt{\sum_m (Q_{mDSV})^2} \tag{18.7-2}$$

where

Q_{mDSV} = force in an element of the damping system required to resist design seismic forces of velocity-dependent damping devices due to the m^{th} mode of vibration of the structure in the direction of interest

Modal seismic design forces in elements of the damping system, Q_{mDSV}, shall be calculated by imposing modal design forces of velocity-dependent devices on the nondeformed damping system as pseudostatic forces. Modal seismic design forces shall be applied in directions consistent with the deformed shape of the mode of interest. Horizontal restraint forces shall be applied at each floor Level i of the nondeformed damping system concurrent with the design forces in velocity-dependent damping devices such that the horizontal displacement at each level of the structure is zero. At each floor Level i, restraint forces shall be proportional to and applied at the location of each mass point.

3. Stage of maximum acceleration: Seismic design force at the stage of maximum acceleration shall be calculated in accordance with Eq. 18.7-3:

$$Q_E = \sqrt{\sum_m (C_{mFD}\Omega_0 Q_{mSFRS} + C_{mFV} Q_{mDSV})^2} \pm Q_{DSD} \tag{18.7-3}$$

The force coefficients, C_{mFD} and C_{mFV}, shall be determined from Tables 18.7-1 and 18.7-2, respectively, using values of effective damping determined in accordance with the following requirements:

For fundamental-mode response (m = 1) in the direction of interest, the coefficients, C_{1FD} and C_{1FV}, shall be based on the velocity exponent, α, that

Table 18.7-1 Force Coefficient, C_{mFD}[a,b]

	μ ≤ 1.0				
Effective Damping	α ≤ 0.25	α = 0.5	α = 0.75	α ≥ 1.0	C_{mFD} = 1.0[c]
≤0.05	1.00	1.00	1.00	1.00	μ ≥ 1.0
0.1	1.00	1.00	1.00	1.00	μ ≥ 1.0
0.2	1.00	0.95	0.94	0.93	μ ≥ 1.1
0.3	1.00	0.92	0.88	0.86	μ ≥ 1.2
0.4	1.00	0.88	0.81	0.78	μ ≥ 1.3
0.5	1.00	0.84	0.73	0.71	μ ≥ 1.4
0.6	1.00	0.79	0.64	0.64	μ ≥ 1.6
0.7	1.00	0.75	0.55	0.58	μ ≥ 1.7
0.8	1.00	0.70	0.50	0.53	μ ≥ 1.9
0.9	1.00	0.66	0.50	0.50	μ ≥ 2.1
≥1.0	1.00	0.62	0.50	0.50	μ ≥ 2.2

[a]Unless analysis or test data support other values, the force coefficient C_{mFD} for viscoelastic systems shall be taken as 1.0.
[b]Interpolation shall be used for intermediate values of velocity exponent, α, and ductility demand, μ.
[c]C_{mFD} shall be taken as equal to 1.0 for values of ductility demand, μ, greater than or equal to the values shown.

Table 18.7-2 Force Coefficient, C_{mFV}[a,b]

Effective Damping	$\alpha \le 0.25$	$\alpha = 0.5$	$\alpha = 0.75$	$\alpha \ge 1.0$
≤0.05	1.00	0.35	0.20	0.10
0.1	1.00	0.44	0.31	0.20
0.2	1.00	0.56	0.46	0.37
0.3	1.00	0.64	0.58	0.51
0.4	1.00	0.70	0.69	0.62
0.5	1.00	0.75	0.77	0.71
0.6	1.00	0.80	0.84	0.77
0.7	1.00	0.83	0.90	0.81
0.8	1.00	0.90	0.94	0.90
0.9	1.00	1.00	1.00	1.00
≥1.0	1.00	1.00	1.00	1.00

[a]Unless analysis or test data support other values, the force coefficient C_{mFD} for viscoelastic systems shall be taken as 1.0.
[b]Interpolation shall be used for intermediate values of velocity exponent, α.

relates device force to damping device velocity. The effective fundamental-mode damping shall be taken as equal to the total effective damping of the fundamental mode less the hysteretic component of damping ($\beta_{1D} - \beta_{HD}$ or $\beta_{1M} - \beta_{HM}$) at the response level of interest ($\mu = \mu_D$ or $\mu = \mu_M$).

For higher-mode ($m > 1$) or residual-mode response in the direction of interest, the coefficients, C_{mFD} and C_{mFV}, shall be based on a value of α equal to 1.0. The effective modal damping shall be taken as equal to the total effective damping of the mode of interest (β_{mD} or β_{mM}). For determination of the coefficient C_{mFD}, the ductility demand shall be taken as equal to that of the fundamental mode ($\mu = \mu_D$ or $\mu = \mu_M$).

18.7.2.6 Inelastic Response Limits

Elements of the damping system are permitted to exceed strength limits for design loads provided it is shown by analysis or test that

1. Inelastic response does not adversely affect damping system function.
2. Element forces calculated in accordance with Section 18.7.2.5, using a value of Ω_0 taken as equal to 1.0, do not exceed the strength required to satisfy the load combinations of Section 12.4.

18.8 DESIGN REVIEW

A design review of the damping system and related test programs shall be performed by an independent team of registered design professionals in the appropriate disciplines and others experienced in seismic analysis methods and the theory and application of energy dissipation systems.

The design review shall include, but need not be limited to, the following:

1. Review of site-specific seismic criteria including the development of the site-specific spectra and ground motion histories and all other project-specific design criteria.
2. Review of the preliminary design of the seismic force-resisting system and the damping system, including design parameters of damping devices.
3. Review of the final design of the seismic force-resisting system and the damping system and all supporting analyses.
4. Review of damping device test requirements, device manufacturing quality control and assurance, and scheduled maintenance and inspection requirements.

18.9 TESTING

The force-velocity displacement and damping properties used for the design of the damping system shall be based on the prototype tests specified in this section.

The fabrication and quality control procedures used for all prototype and production damping devices shall be identical.

18.9.1 Prototype Tests

The following tests shall be performed separately on two full-size damping devices of each type and size used in the design, in the order listed as follows.

Representative sizes of each type of device are permitted to be used for prototype testing, provided both of the following conditions are met:

1. Fabrication and quality control procedures are identical for each type and size of device used in the structure.
2. Prototype testing of representative sizes is accepted by the registered design professional responsible for design of the structure.

Test specimens shall not be used for construction, unless they are accepted by the registered design professional responsible for design of the structure and meet the requirements for prototype and production tests.

18.9.1.1 Data Recording

The force-deflection relationship for each cycle of each test shall be recorded.

18.9.1.2 Sequence and Cycles of Testing

For the following test sequences, each damping device shall be subjected to gravity load effects and thermal environments representative of the installed condition. For seismic testing, the displacement in the devices calculated for the maximum considered earthquake ground motions, termed herein as the maximum device displacement, shall be used.

1. Each damping device shall be subjected to the number of cycles expected in the design windstorm, but not less than 2,000 continuous fully reversed cycles of wind load. Wind load shall be at amplitudes expected in the design windstorm and shall be applied at a frequency equal to the inverse of the fundamental period of the structure ($f_1 = 1/T_1$).

EXCEPTION: Damping devices need not be subjected to these tests if they are not subject to wind-induced forces or displacements or if the design wind force is less than the device yield or slip force.

2. Each damping device shall be loaded with five fully reversed, sinusoidal cycles at the maximum earthquake device displacement at a frequency equal to $1/T_{1M}$ as calculated in Section 18.4.2.5. Where the damping device characteristics vary with operating temperature, these tests shall be conducted at a minimum of three temperatures (minimum, ambient, and maximum) that bracket the range of operating temperatures.

EXCEPTION: Damping devices are permitted to be tested by alternative methods provided all of the following conditions are met:

a. Alternative methods of testing are equivalent to the cyclic testing requirements of this section.
b. Alternative methods capture the dependence of the damping device response on ambient temperature, frequency of loading, and temperature rise during testing.
c. Alternative methods are accepted by the registered design professional responsible for the design of the structure.

3. If the force-deformation properties of the damping device at any displacement less than or equal to the maximum device displacement change by more than 15 percent for changes in testing frequency from $1/T_{1M}$ to $2.5/T_1$, then the preceding tests shall also be performed at frequencies equal to $1/T_1$ and $2.5/T_1$.

If reduced-scale prototypes are used to qualify the rate-dependent properties of damping devices, the reduced-scale prototypes should be of the same type and materials, and manufactured with the same processes and quality control procedures, as full-scale prototypes, and tested at a similitude-scaled frequency that represents the full-scale loading rates.

18.9.1.3 Testing Similar Devices

Damping devices need not be prototype tested provided that both of the following conditions are met:

1. All pertinent testing and other damping device data are made available to and are accepted by the registered design professional responsible for the design of the structure.
2. The registered design professional substantiates the similarity of the damping device to previously tested devices.

18.9.1.4 Determination of Force-Velocity-Displacement Characteristics

The force-velocity-displacement characteristics of a damping device shall be based on the cyclic load and displacement tests of prototype devices specified in the preceding text. Effective stiffness of a damping device shall be calculated for each cycle of deformation using Eq. 17.8-1.

18.9.1.5 Device Adequacy

The performance of a prototype damping device shall be deemed adequate if all of the conditions listed below are satisfied. The 15 percent limits specified in the following text are permitted to be increased by the registered design professional responsible for the design of the structure provided that the increased limit has been demonstrated by analysis not to have a deleterious effect on the response of the structure.

18.9.1.5.1 Displacement-Dependent Damping Devices The performance of the prototype displacement-dependent damping devices shall be deemed adequate if the following conditions, based on tests specified in Section 18.9.1.2, are satisfied:

1. For Test 1, no signs of damage including leakage, yielding, or breakage.
2. For Tests 2 and 3, the maximum force and minimum force at zero displacement for a damping device for any one cycle does not differ by more

than 15 percent from the average maximum and minimum forces at zero displacement as calculated from all cycles in that test at a specific frequency and temperature.

3. For Tests 2 and 3, the maximum force and minimum force at maximum device displacement for a damping device for any one cycle does not differ by more than 15 percent from the average maximum and minimum forces at the maximum device displacement as calculated from all cycles in that test at a specific frequency and temperature.
4. For Tests 2 and 3, the area of hysteresis loop (E_{loop}) of a damping device for any one cycle does not differ by more than 15 percent from the average area of the hysteresis loop as calculated from all cycles in that test at a specific frequency and temperature.
5. The average maximum and minimum forces at zero displacement and maximum displacement, and the average area of the hysteresis loop (E_{loop}), calculated for each test in the sequence of Tests 2 and 3, shall not differ by more than 15 percent from the target values specified by the registered design professional responsible for the design of the structure.

18.9.1.5.2 Velocity-Dependent Damping Devices The performance of the prototype velocity-dependent damping devices shall be deemed adequate if the following conditions, based on tests specified in Section 18.9.1.2, are satisfied:

1. For Test 1, no signs of damage including leakage, yielding, or breakage.
2. For velocity-dependent damping devices with stiffness, the effective stiffness of a damping device in any one cycle of Tests 2 and 3 does not differ by more than 15 percent from the average effective stiffness as calculated from all cycles in that test at a specific frequency and temperature.
3. For Tests 2 and 3, the maximum force and minimum force at zero displacement for a damping device for any one cycle does not differ by more than 15 percent from the average maximum and minimum forces at zero displacement as calculated from all cycles in that test at a specific frequency and temperature.
4. For Tests 2 and 3, the area of hysteresis loop (E_{loop}) of a damping device for any one cycle does not differ by more than 15 percent from the average area of the hysteresis loop as calculated from all cycles in that test at a specific frequency and temperature.
5. The average maximum and minimum forces at zero displacement, effective stiffness (for damping devices with stiffness only), and average area of the hysteresis loop (E_{loop}) calculated for each test in the sequence of Tests 2 and 3, does not differ by more than 15 percent from the target values specified by the registered design professional responsible for the design of the structure.

18.9.2 Production Testing

Prior to installation in a building, damping devices shall be tested to ensure that their force-velocity-displacement characteristics fall within the limits set by the registered design professional responsible for the design of the structure. The scope and frequency of the production-testing program shall be determined by the registered design professional responsible for the design of the structure.

Chapter 19
SOIL–STRUCTURE INTERACTION FOR SEISMIC DESIGN

19.1 GENERAL

If the option to incorporate the effects of soil–structure interaction is exercised, the requirements of this section are permitted to be used in the determination of the design earthquake forces and the corresponding displacements of the structure if the model used for structural response analysis does not directly incorporate the effects of foundation flexibility (i.e., the model corresponds to a fixed-based condition with no foundation springs). The provisions in this section shall not be used if a flexible-base foundation is included in the structural response model.

The provisions for use with the equivalent lateral force procedure are given in Section 19.2, and those for use with the modal analysis procedure are given in Section 19.3.

19.2 EQUIVALENT LATERAL FORCE PROCEDURE

The following requirements are supplementary to those presented in Section 12.8.

19.2.1 Base Shear

To account for the effects of soil–structure interaction, the base shear (V) determined from Eq. 12.8-1 shall be reduced to

$$\tilde{V} = V - \Delta V \qquad (19.2\text{-}1)$$

The reduction (ΔV) shall be computed as follows and shall not exceed 0.3V:

$$\Delta V = \left[C_s - \tilde{C}_s \left(\frac{0.05}{\tilde{\beta}} \right)^{0.4} \right] \bar{W} \le 0.3V \qquad (19.2\text{-}2)$$

where

C_s = the seismic design coefficient computed from Eqs. 12.8-2, 12.8-3, and through 12.8-4 using the fundamental natural period of the fixed-base structure (T or T_a) as specified in Section 12.8.2

$\tilde{C}$ = the value of C_s computed from Eqs. 12.8-2, 12.8-3, and through 12.8-4 using the fundamental natural period of the flexibly supported structure ($\tilde{T}$) defined in Section 19.2.1.1

$\tilde{\beta}$ = the fraction of critical damping for the structure-foundation system determined in Section 19.2.1.2

$\bar{W}$ = the effective seismic weight of the structure, which shall be taken as 0.7W, except for structures where the effective seismic weight is concentrated at a single level, it shall be taken as equal to W

19.2.1.1 Effective Building Period

The effective period ($\tilde{T}$) shall be determined as follows:

$$\tilde{T} = T\sqrt{1 + \frac{\bar{k}}{K_y}\left(1 + \frac{K_y \bar{h}^2}{K_\theta}\right)} \qquad (19.2\text{-}3)$$

where

T = the fundamental period of the structure as determined in Section 12.8.2

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

$$\bar{k} = 4\pi^2 \left(\frac{\bar{W}}{gT^2} \right) \qquad (19.2\text{-}4)$$

where

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

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

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

g = the acceleration of gravity

The foundation stiffnesses (K_y and K_θ) shall be computed by established principles of foundation mechanics using soil properties that are compatible

Table 19.2-1 Values of G/G_o and v_s/v_{so}

	Value of v_s/v_{so}			Value of G/G_o		
	$S_{DS}/2.5$			$S_{DS}/2.5$		
Site Class	≤0.1	0.4	≥0.8	≤0.1	0.4	≥0.8
A	1.00	1.00	1.00	1.00	1.00	1.00
B	1.00	0.97	0.95	1.00	0.95	0.90
C	0.97	0.87	0.77	0.95	0.75	0.60
D	0.95	0.71	0.32	0.90	0.50	0.10
E	0.77	0.22	[a]	0.60	0.05	[a]
F	[a]	[a]	[a]	[a]	[a]	[a]

Note: Use straight-line interpolation for intermediate values of $S_{DS}/2.5$.
[a]Should be evaluated from site specific analysis

Table 19.2-2 Values of α_θ

r_m/v_sT	α_θ
<0.05	1.0
0.15	0.85
0.35	0.7
0.5	0.6

with the soil strain levels associated with the design earthquake motion. The average shear modulus (G) for the soils beneath the foundation at large strain levels and the associated shear wave velocity (v_s) needed in these computations shall be determined from Table 19.2-1 where

v_{so} = the average shear wave velocity for the soils beneath the foundation at small strain levels (10^{-3} percent or less)
$G_o = \gamma v^2_{so}/g$ = the average shear modulus for the soils beneath the foundation at small strain levels
γ = the average unit weight of the soils

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

$$\tilde{T} = T\sqrt{1+\frac{25\alpha r_a \bar{h}}{v_s^2 T^2}\left(1+\frac{1.12 r_a \bar{h}^2}{\alpha_\theta r_m^3}\right)} \quad (19.2\text{-}5)$$

where

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

$$\alpha = \frac{\bar{W}}{\gamma A_o \bar{h}} \quad (19.2\text{-}6)$$

r_a and r_m = characteristic foundation lengths defined by

$$r_a = \sqrt{\frac{A_o}{\pi}} \quad (19.2\text{-}7)$$

and

$$r_m = 4\sqrt{\frac{4I_o}{\pi}} \quad (19.2\text{-}8)$$

where

A_o = the area of the load-carrying foundation
I_o = the static moment of inertia of the load-carrying foundation about a horizontal centroidal axis normal to the direction in which the structure is analyzed
α_θ = dynamic foundation stiffness modifier for rocking as determined from Table 19.2-2
v_s = shear wave velocity
T = fundamental period as determined in Section 12.8.2

19.2.1.2 Effective Damping

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

$$\tilde{\beta} = \beta_o \frac{0.05}{\left(\frac{\tilde{T}}{T}\right)^3} \quad (19.2\text{-}9)$$

where

β_o = the foundation damping factor as specified in Fig. 19.2-1

For values of $\frac{S_{DS}}{2.5}$ between 0.10 and 0.20 the values of β_o shall be determined by linear interpolation vbetween the solid lines and the dashed lines of Fig. 19.2-1.

The quantity r in Fig. 19.2-1 is a characteristic foundation length that shall be determined as follows:

$$\text{For } \frac{\bar{h}}{L_0} \le 0.5,\ r = r_a \quad (19.2\text{-}10)$$

$$\text{For } \frac{\bar{h}}{L_0} \ge 1,\ r = r_m \quad (19.2\text{-}11)$$

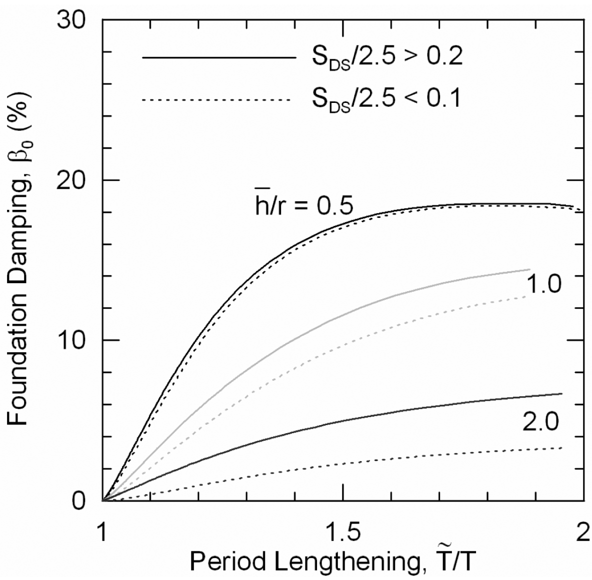


FIGURE 19.2-1 Foundation Damping Factor

where

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

r_a and r_m = characteristic foundation lengths defined in Eqs. 19.2-7 and 19.2-8, respectively

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

EXCEPTION: For structures supported on point-bearing piles and in all other cases where the foundation soil consists of a soft stratum of reasonably uniform properties underlain by a much stiffer, rock-like deposit with an abrupt increase in stiffness, the factor β_o in Eq. 19.2-9 shall be replaced by β'_o if $\dfrac{4D_s}{v_s\tilde{T}} < 1$ where D_s is the total depth of the stratum. β'_o shall be determined as follows:

$$\beta'_o = \left(\frac{4D_s}{v_s\tilde{T}}\right)^2 \beta_o \qquad (19.2\text{-}12)$$

The value of $\tilde{\beta}$ computed from Eq. 19.2-9, both with or without the adjustment represented by Eq. 19.2-12, shall in no case be taken as less than $\tilde{\beta}$ = 0.05 or greater than $\tilde{\beta}$ = 0.20.

19.2.2 Vertical Distribution of Seismic Forces

The distribution over the height of the structure of the reduced total seismic force ($\tilde{V}$) shall be considered to be the same as for the structure without interaction.

19.2.3 Other Effects

The modified story shears, overturning moments, and torsional effects about a vertical axis shall be determined as for structures without interaction using the reduced lateral forces.

The modified deflections ($\tilde{\delta}$) shall be determined as follows:

$$\tilde{\delta}_x = \frac{\tilde{V}}{V}\left[\frac{M_o h_x}{K_\theta} + \delta_x\right] \qquad (19.2\text{-}13)$$

where

M_o = the overturning moment at the base using the unmodified seismic forces and not including the reduction permitted in the design of the foundation

h_x = the height above the base to the level under consideration

δ_x = the deflections of the fixed-base structure as determined in Section 12.8.6 using the unmodified seismic forces

The modified story drifts and P-delta effects shall be evaluated in accordance with the provisions of Sections 12.8.6 and 12.8.7 using the modified story shears and deflections determined in this section.

19.3 MODAL ANALYSIS PROCEDURE

The following provisions are supplementary to those presented in Section 12.9.

19.3.1 Modal Base Shears

To account for the effects of soil–structure interaction, the base shear corresponding to the fundamental mode of vibration (V_1) shall be reduced to

$$\tilde{V}_1 = V_1 - \Delta V_1 \qquad (19.3\text{-}1)$$

The reduction (ΔV_1) shall be computed in accordance with Eq. 19.2-2 with $\bar{W}$ taken as equal to the effective seismic weight of the fundamental period of vibration, $\bar{W}$, and C_s computed in accordance with Eq. 12.8-1, except that S_{DS} shall be replaced by design spectral response acceleration of the design response spectra at the fundamental period of the fixed-base structure (T_1).

The period $\tilde{T}$ shall be determined from Eq. 19.2-3 or from Eq. 19.2-5 where applicable, taking $T = T_1$, evaluating $\bar{k}$ from Eq. 19.2-4 with $\bar{W} = \bar{W}_1$, and computing $\bar{h}$ as follows:

$$\bar{h} = \frac{\sum_{i=1}^{n} w_i \varphi_{i1} h_i}{\sum_{i=1}^{n} w_i \varphi_{i1}} \qquad (19.3\text{-}2)$$

where

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

φ_{i1} = the displacement amplitude at the i^{th} level of the structure when vibrating in its fundamental mode

h_i = the height above the base to Level i

The preceding designated values of $\bar{W}$, $\bar{h}$, T, and $\tilde{T}$ also shall be used to evaluate the factor α from Eq. 19.2-6 and the factor β_o from Fig. 19.2-1. No reduction shall be made in the shear components contributed by the higher modes of vibration. The reduced base shear ($\tilde{V}_1$) shall in no case be taken less than $0.7V_1$.

19.3.2 Other Modal Effects

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

$$\tilde{\delta}_{x1} = \frac{\tilde{V}_1}{V_1}\left[\frac{M_{o1} h_x}{K_\theta} + \delta_{x1}\right] \qquad (19.3\text{-}3)$$

and

$$\tilde{\delta}_{xm} = \delta_{xm} \text{ for } m = 2, 3, \ldots \qquad (19.3\text{-}4)$$

where

M_{o1} = the overturning base moment for the fundamental mode of the fixed-base structure using the unmodified modal base shear V_1

δ_{xm} = the modal deflections at Level x of the fixed-base structure using the unmodified modal shears, V_m

The modified modal drift in a story ($\tilde{\Delta}_m$) shall be computed as the difference of the deflections ($\tilde{\delta}_{xm}$) at the top and bottom of the story under consideration.

19.3.3 Design Values

The design values of the modified shears, moments, deflections, and story drifts shall be determined as for structures without interaction by taking the square root of the sum of the squares (SRSS) of the respective modal contributions. In the design of the foundation, it is permitted to reduce the overturning moment at the foundation–soil interface determined in this manner by 10 percent as for structures without interaction.

The effects of torsion about a vertical axis shall be evaluated in accordance with the provisions of Section 12.8.4, and the P-delta effects shall be evaluated in accordance with the provisions of Section 12.8.7 using the story shears and drifts determined in Section 19.3.2.

Chapter 20

SITE CLASSIFICATION PROCEDURE FOR SEISMIC DESIGN

20.1 SITE CLASSIFICATION

The site soil shall be classified in accordance with Table 20.3-1 and Section 20.3 based on the upper 100 ft (30 m) of the site profile. Where site-specific data are not available to a depth of 100 ft (30 m), appropriate soil properties are permitted to be estimated by the registered design professional preparing the soil investigation report based on known geologic conditions. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the authority having jurisdiction or geotechnical data determine Site Class E or F soils are present at the site. Site Classes A and B shall not be assigned to a site if there is more than 10 ft (10.1 m) of soil between the rock surface and the bottom of the spread footing or mat foundation.

20.2 SITE RESPONSE ANALYSIS FOR SITE CLASS F SOIL

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

20.3 SITE CLASS DEFINITIONS

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

20.3.1 Site Class F

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

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

EXCEPTION: For structures having fundamental periods of vibration equal to or less than 0.5 s, site response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class is permitted to be determined in accordance with Section 20.3 and the corresponding values of F_a and F_v determined from Tables 11.4-1 and 11.4-2.

2. Peats and/or highly organic clays [$H > 10$ ft (3 m)] of peat and/or highly organic clay where H = thickness of soil.
3. Very high plasticity clays [$H > 25$ ft (7.6 m) with $PI > 75$].
4. Very thick soft/medium stiff clays [$H > 120$ ft (37 m)] with $s_u < 1{,}000$ psf (50 kPa).

20.3.2 Soft Clay Site Class E

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

20.3.3 Site Classes C, D, and E

The existence of Site Class C, D, and E soils shall be classified by using one of the following three methods with $\bar{v}_s$, $\bar{N}$, and $\bar{s}_u$ computed in all cases as specified in Section 20.4:

1. $\bar{v}_s$ for the top 100 ft (30 m) ($\bar{v}_s$ method).
2. $\bar{N}$ for the top 100 ft (30 m) ($\bar{N}$ method).
3. $\bar{N}_{ch}$ for cohesionless soil layers ($PI < 20$) in the top 100 ft (30 m) and $\bar{s}_u$ for cohesive soil layers ($PI > 20$) in the top 100 ft (30 m) ($\bar{s}_u$ method). Where the $\bar{N}_{ch}$ and $\bar{s}_u$ criteria differ, the site shall be assigned to the category with the softer soil.

20.3.4 Shear Wave Velocity for Site Class B

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

20.3.5 Shear Wave Velocity for Site Class A

The hard rock, Site Class A, category shall be supported by shear wave velocity measurement either

Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the following characteristics: —Plasticity index $PI > 20$, —Moisture content $w \geq 40\%$, —Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1 ft/s = 0.3048 m/s; 1 lb/ft^2 = 0.0479 kN/m^2.

on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 ft (30 m), surficial shear wave velocity measurements are permitted to be extrapolated to assess $\bar{v}_s$.

20.4 DEFINITIONS OF SITE CLASS PARAMETERS

The definitions presented in this section shall apply to the upper 100 ft (30 m) of the site profile. Profiles containing distinct soil and rock layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there are a total of n distinct layers in the upper 100 ft (30 m). Where some of the n layers are cohesive and others are not, k is the number of cohesive layers and m is the number of cohesionless layers. The symbol i refers to any one of the layers between 1 and n.

20.4.1 $\bar{v}_s$, Average Shear Wave Velocity

$\bar{v}_s$ shall be determined in accordance with the following formula:

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \tag{20.4-1}$$

where

d_i = the thickness of any layer between 0 and 100 ft (30 m)

v_{si} = the shear wave velocity in ft/s (m/s)

$\sum_{i=1}^{n} d_i$ = 100 ft (30 m)

20.4.2 $\bar{N}$, Average Field Standard Penetration Resistance and $\bar{N}_{ch}$, Average Standard Penetration Resistance for Cohesionless Soil Layers

$\bar{N}$ and $\bar{N}_{ch}$ shall be determined in accordance with the following formulas:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \tag{20.4-2}$$

where N_i and d_i in Eq. 20.4-2 are for cohesionless soil, cohesive soil, and rock layers.

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \tag{20.4-3}$$

where N_i and d_i in Eq. 20.4-3 are for cohesionless soil layers only and $\sum_{i=1}^{m} d_i = d_s$ where d_s is the total thickness of cohesionless soil layers in the top 100 ft (30 m). N_i is the standard penetration resistance (ASTM D1586) not to exceed 100 blows/ft (305 blows/m) as directly measured in the field without corrections. Where refusal is met for a rock layer, N_i shall be taken as 100 blows/ft (305 blows/m).

20.4.3 $\bar{s}_u$, Average Undrained Shear Strength

$\bar{s}_u$ shall be determined in accordance with the following formula:

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \tag{20.4-4}$$

where

$\sum_{i=1}^{k} d_i = d_c$

d_c = the total thickness of cohesive soil layers in the top 100 ft (30 m)

PI = the plasticity index as determined in accordance with ASTM D4318

w = the moisture content in percent as determined in accordance with ASTM D2216

s_{ui} = the undrained shear strength in psf (kPa), not to exceed 5,000 psf (240 kPa) as determined in accordance with ASTM D2166 or ASTM D2850

Chapter 21
SITE-SPECIFIC GROUND MOTION PROCEDURES FOR SEISMIC DESIGN

21.1 SITE RESPONSE ANALYSIS

The requirements of Section 21.1 shall be satisfied where site response analysis is performed or required by Section 11.4.7. The analysis shall be documented in a report.

21.1.1 Base Ground Motions

A MCE_R response spectrum shall be developed for bedrock, using the procedure of Sections 11.4.6 or 21.2. Unless a site-specific ground motion hazard analysis described in Section 21.2 is carried out, the MCE_R rock response spectrum shall be developed using the procedure of Section 11.4.6 assuming Site Class B. If bedrock consists of Site Class A, the spectrum shall be adjusted using the site coefficients in Section 11.4.3 unless other site coefficients can be justified. At least five recorded or simulated horizontal ground motion acceleration time histories shall be selected from events having magnitudes and fault distances that are consistent with those that control the MCE_R ground motion. Each selected time history shall be scaled so that its response spectrum is, on average, approximately at the level of the MCE_R rock response spectrum over the period range of significance to structural response.

21.1.2 Site Condition Modeling

A site response model based on low-strain shear wave velocities, nonlinear or equivalent linear shear stress–strain relationships, and unit weights shall be developed. Low-strain shear wave velocities shall be determined from field measurements at the site or from measurements from similar soils in the site vicinity. Nonlinear or equivalent linear shear stress–strain relationships and unit weights shall be selected on the basis of laboratory tests or published relationships for similar soils. The uncertainties in soil properties shall be estimated. Where very deep soil profiles make the development of a soil model to bedrock impractical, the model is permitted to be terminated where the soil stiffness is at least as great as the values used to define Site Class D in Chapter 20. In such cases, the MCE_R response spectrum and acceleration time histories of the base motion developed in Section 21.1.1 shall be adjusted upward using site coefficients in Section 11.4.3 consistent with the classification of the soils at the profile base.

21.1.3 Site Response Analysis and Computed Results

Base ground motion time histories shall be input to the soil profile as outcropping motions. Using appropriate computational techniques that treat nonlinear soil properties in a nonlinear or equivalent-linear manner, the response of the soil profile shall be determined and surface ground motion time histories shall be calculated. Ratios of 5 percent damped response spectra of surface ground motions to input base ground motions shall be calculated. The recommended surface MCE_R ground motion response spectrum shall not be lower than the MCE_R response spectrum of the base motion multiplied by the average surface-to-base response spectral ratios (calculated period by period) obtained from the site response analyses. The recommended surface ground motions that result from the analysis shall reflect consideration of sensitivity of response to uncertainty in soil properties, depth of soil model, and input motions.

21.2 RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION HAZARD ANALYSIS

The requirements of Section 21.2 shall be satisfied where a ground motion hazard analysis is performed or required by Section 11.4.7. The ground motion hazard analysis shall account for the regional tectonic setting, geology, and seismicity, the expected recurrence rates and maximum magnitudes of earthquakes on known faults and source zones, the characteristics of ground motion attenuation, near source effects, if any, on ground motions, and the effects of subsurface site conditions on ground motions. The characteristics of subsurface site conditions shall be considered either using attenuation relations that represent regional and local geology or in accordance with Section 21.1. The analysis shall incorporate current seismic interpretations, including uncertainties for models and parameter values for seismic sources and ground motions. The analysis shall be documented in a report.

21.2.1 Probabilistic (MCE_R) Ground Motions

The probabilistic spectral response accelerations shall be taken as the spectral response accelerations in the direction of maximum horizontal response represented by a 5 percent damped acceleration response spectrum that is expected to achieve a 1 percent probability of collapse within a 50-year period. For the purpose of this standard, ordinates of the probabilistic ground motion response spectrum shall be determined by either Method 1 of Section 21.2.1.1 or Method 2 of Section 21.2.1.2.

21.2.1.1 Method 1

At each spectral response period for which the acceleration is computed, ordinates of the probabilistic ground motion response spectrum shall be determined as the product of the risk coefficient, C_R, and the spectral response acceleration from a 5 percent damped acceleration response spectrum having a 2 percent probability of exceedance within a 50-year period. The value of the risk coefficient, C_R, shall be determined using values of C_{RS} and C_{R1} from Figs. 22-3 and 22-4, respectively. At spectral response periods less than or equal to 0.2 s, C_R shall be taken as equal to C_{RS}. At spectral response periods greater than or equal to 1.0 s, C_R shall be taken as equal to C_{R1}. At response spectral periods greater than 0.2 s and less than 1.0 s, C_R shall be based on linear interpolation of C_{RS} and C_{R1}.

21.2.1.2 Method 2

At each spectral response period for which the acceleration is computed, ordinates of the probabilistic ground motion response spectrum shall be determined from iterative integration of a site-specific hazard curve with a lognormal probability density function representing the collapse fragility (i.e., probability of collapse as a function of spectral response acceleration). The ordinate of the probabilistic ground motion response spectrum at each period shall achieve a 1 percent probability of collapse within a 50-year period for a collapse fragility having (i) a 10 percent probability of collapse at said ordinate of the probabilistic ground motion response spectrum and (ii) a logarithmic standard deviation value of 0.6.

21.2.2 Deterministic (MCE_R) Ground Motions

The deterministic spectral response acceleration at each period shall be calculated as an 84th-percentile 5 percent damped spectral response acceleration in the direction of maximum horizontal response computed at that period. The largest such acceleration calculated for the characteristic earthquakes on all known active faults within the region shall be used. For the purposes of this standard, the ordinates of the deterministic ground motion response spectrum shall not be taken as lower than the corresponding ordinates of the response spectrum determined in accordance with Fig. 21.2-1, where F_a and F_v are determined using Tables 11.4-1 and 11.4-2, respectively, with the value of S_S taken as 1.5 and the value of S_1 taken as 0.6.

21.2.3 Site-Specific MCE_R

The site-specific MCE_R spectral response acceleration at any period, S_{aM}, shall be taken as the lesser of the spectral response accelerations from the probabilistic ground motions of Section 21.2.1 and the deterministic ground motions of Section 21.2.2.

21.3 DESIGN RESPONSE SPECTRUM

The design spectral response acceleration at any period shall be determined from Eq. 21.3-1:

$$S_a = \frac{2}{3} S_{aM} \qquad (21.3\text{-}1)$$

where S_{aM} is the MCE_R spectral response acceleration obtained from Section 21.1 or 21.2. The design spectral response acceleration at any period shall not be taken as less than 80 percent of S_a determined in accordance with Section 11.4.5. For sites classified as Site Class F requiring site response analysis in accordance with Section 11.4.7, the design spectral response acceleration at any period shall not be taken as less than 80 percent of S_a determined for Site Class E in accordance with Section 11.4.5.

21.4 DESIGN ACCELERATION PARAMETERS

Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall be taken as the spectral acceleration, S_a, obtained from the site-specific spectra at a period of 0.2 s, except that it shall not be taken as less than 90 percent of the peak spectral acceleration, S_a, at any period larger than 0.2 s. The parameter S_{D1} shall be taken as the greater of the spectral acceleration, S_a, at a period of 1 s or two times the spectral acceleration, S_a, at a period of 2 s. The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The values so obtained shall not be less than 80 percent of the values determined in

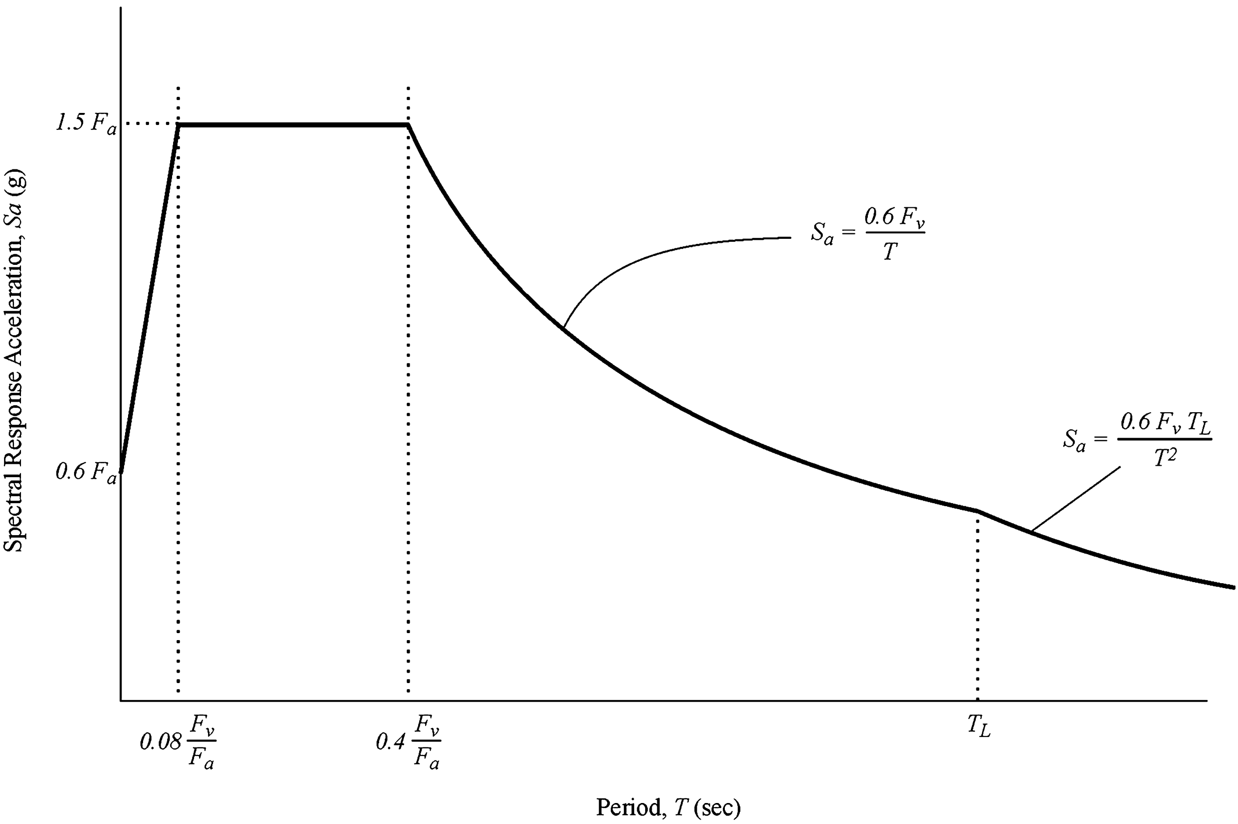


FIGURE 21.2-1 Deterministic Lower Limit on MCE_R Response Spectrum

accordance with Section 11.4.3 for S_{MS} and S_{M1} and Section 11.4.4 for S_{DS} and S_{D1}.

For use with the Equivalent Lateral Force Procedure, the site-specific spectral acceleration, S_a, at T shall be permitted to replace S_{D1}/T in Eq. 12.8-3 and $S_{D1}T_L/T^2$ in Eq. 12.8-4. The parameter S_{DS} calculated per this section shall be permitted to be used in Eqs. 12.8-2, 12.8-5, 15.4-1, and 15.4-3. The mapped value of S_1 shall be used in Eqs. 12.8-6, 15.4-2, and 15.4-4.

21.5 MAXIMUM CONSIDERED EARTHQUAKE GEOMETRIC MEAN (MCE_G) PEAK GROUND ACCELERATION

21.5.1 Probabilistic MCE_G Peak Ground Acceleration

The probabilistic geometric mean peak ground acceleration shall be taken as the geometric mean peak ground acceleration with a 2 percent probability of exceedance within a 50-year period.

21.5.2 Deterministic MCE_G Peak Ground Acceleration

The deterministic geometric mean peak ground acceleration shall be calculated as the largest 84^{th}-percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region. The deterministic geometric mean peak ground acceleration shall not be taken as lower than 0.5 F_{PGA}, where F_{PGA} is determined using Table 11.8-1 with the value of PGA taken as 0.5 g.

21.5.3 Site-Specific MCE_G Peak Ground Acceleration

The site-specific MCE_G peak ground acceleration, PGA_M, shall be taken as the lesser of the probabilistic geometric mean peak ground acceleration of Section 21.5.1 and the deterministic geometric mean peak ground acceleration of Section 21.5.2. The site-specific MCE_G peak ground acceleration shall not be taken as less than 80 percent of PGA_M determined from Eq. 11.8-1.

Chapter 22

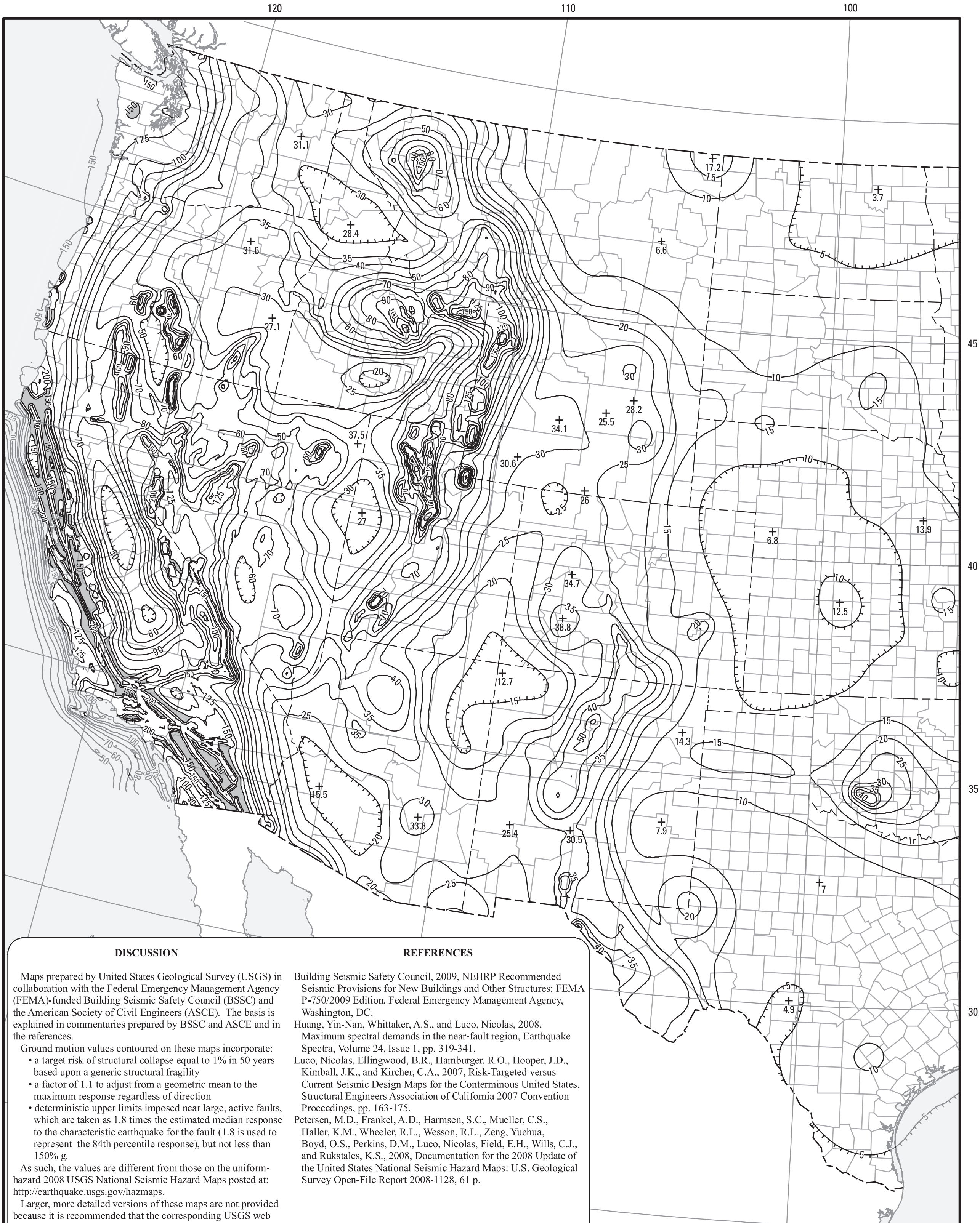
SEISMIC GROUND MOTION LONG-PERIOD TRANSITION AND RISK COEFFICIENT MAPS

Contained in this chapter are Figs. 22-1 through 22-6, which provide the risk-adjusted maximum considered earthquake (MCE_R) ground motion parameters S_S and S_1; Figs. 22-17 and 22-18, which provide the risk coefficients C_{RS} and C_{R1}; and Figs. 22-12 through 22-15, which provide the long-period transition periods T_L for use in applying the seismic provisions of this standard. S_S is the risk-adjusted MCE_R, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 11.4.1. S_1 is the mapped MCE_R ground motion, 5 percent damped, spectral response acceleration parameter at a period of 1 s as defined in Section 11.4.1. C_{RS} is the mapped risk coefficient at short periods used in Section 21.2.1.1. C_{R1} is the mapped risk coefficient at a period of 1 s used in Section 21.2.1.1. T_L is the mapped long-period transition period used in Section 11.4.5.

These maps were prepared by the United States Geological Survey (USGS) in collaboration with the Building Seismic Safety Council (BSSC) Seismic Design Procedures Reassessment Group and the American Society of Civil Engineers (ASCE) 7 Seismic Subcommittee and have been updated for the 2010 edition of this standard.

Maps of the MCE_R ground motion parameters, S_S and S_1, for Guam and American Samoa are not provided because parameters have not yet been developed for those islands. Therefore, as in the 2005 edition of this standard, the parameters S_S and S_1 shall be, respectively, 1.5 and 0.6 for Guam and 1.0 and 0.4 for American Samoa. Maps of the mapped risk coefficients, C_{RS} and C_{R1}, are also not provided.

Also contained in this chapter are Figs. 22-7 through 22-11, which provide the maximum considered earthquake geometric mean (MCE_G) peak ground accelerations as a percentage of g for Site Class B.



DISCUSSION

Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC) and the American Society of Civil Engineers (ASCE). The basis is explained in commentaries prepared by BSSC and ASCE and in the references.

Ground motion values contoured on these maps incorporate:

- a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
- a factor of 1.1 to adjust from a geometric mean to the maximum response regardless of direction
- deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the fault (1.8 is used to represent the 84th percentile response), but not less than 150% g.

As such, the values are different from those on the uniform-hazard 2008 USGS National Seismic Hazard Maps posted at: http://earthquake.usgs.gov/hazmaps.

Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps or http://content.seinstitute.org) be used to determine the mapped value for a specified location.

REFERENCES

Building Seismic Safety Council, 2009, NEHRP Recommended Seismic Provisions for New Buildings and Other Structures: FEMA P-750/2009 Edition, Federal Emergency Management Agency, Washington, DC.

Huang, Yin-Nan, Whittaker, A.S., and Luco, Nicolas, 2008, Maximum spectral demands in the near-fault region, Earthquake Spectra, Volume 24, Issue 1, pp. 319-341.

Luco, Nicolas, Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.

Petersen, M.D., Frankel, A.D., Harmsen, S.C., Mueller, C.S., Haller, K.M., Wheeler, R.L., Wesson, R.L., Zeng, Yuehua, Boyd, O.S., Perkins, D.M., Luco, Nicolas, Field, E.H., Wills, C.J., and Rukstales, K.S., 2008, Documentation for the 2008 Update of the United States National Seismic Hazard Maps: U.S. Geological Survey Open-File Report 2008-1128, 61 p.

FIGURE 22-1 S_S Risk-Adjusted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.

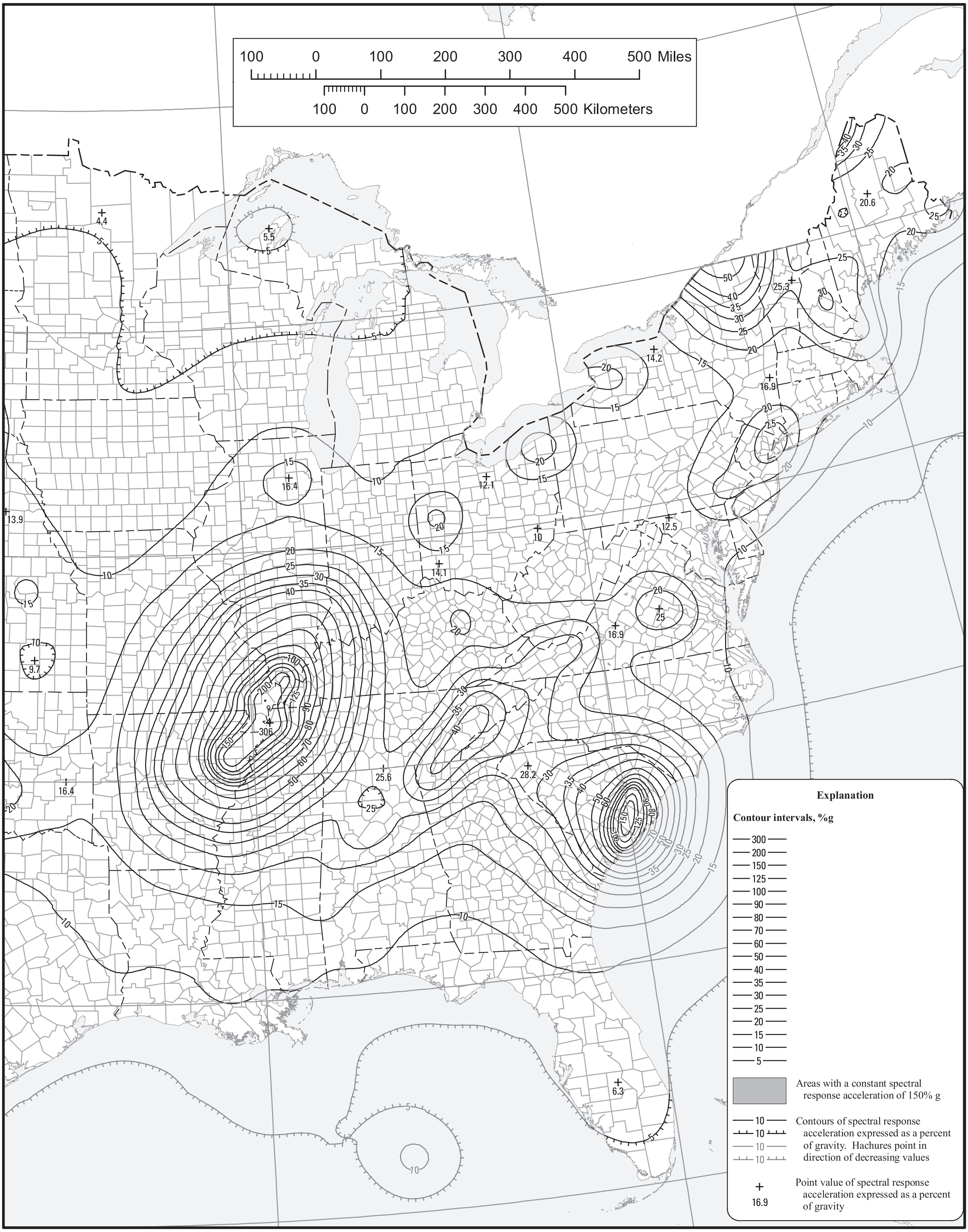
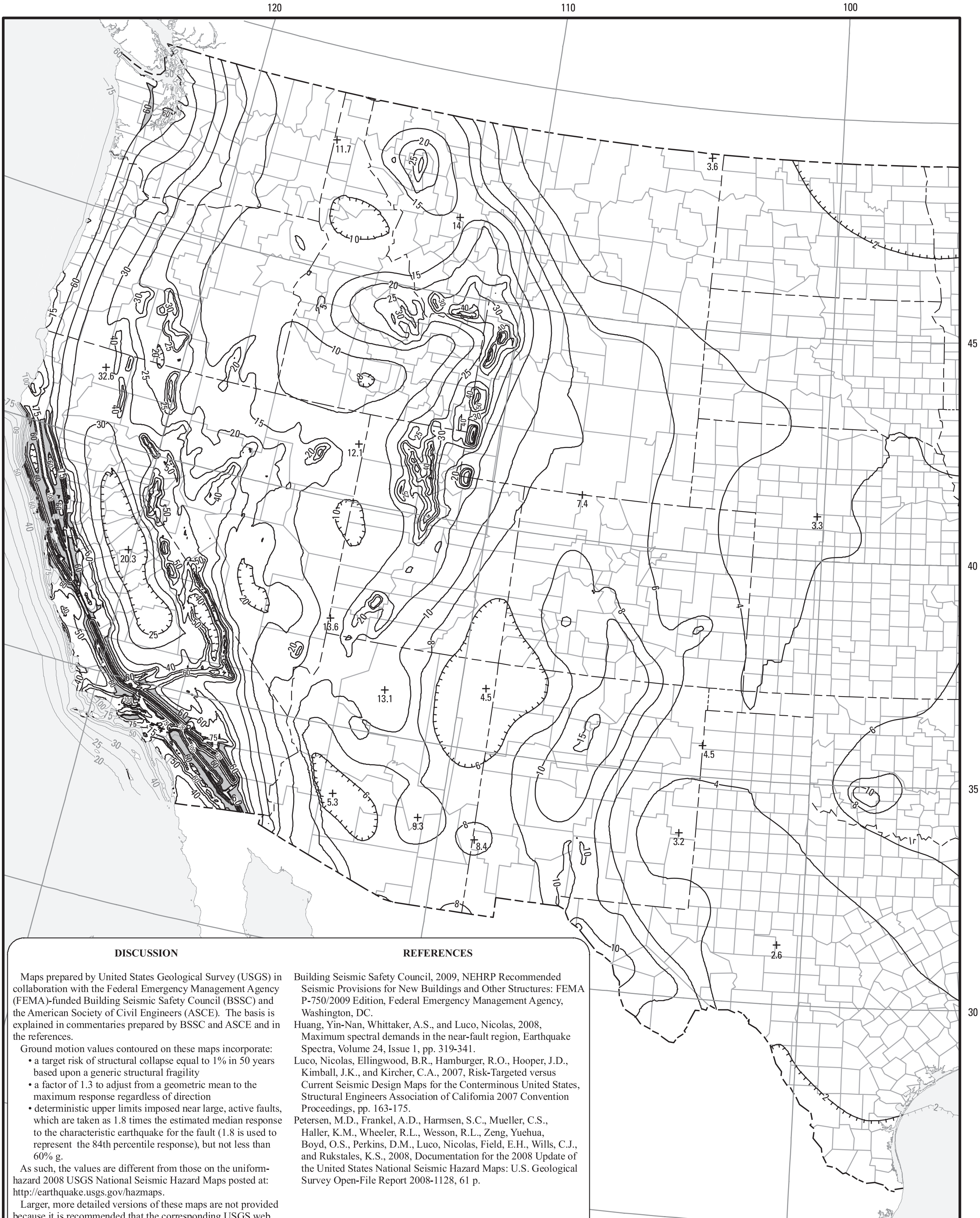


FIGURE 22-1 (*Continued*)



DISCUSSION

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Ground motion values contoured on these maps incorporate:

- a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
- a factor of 1.3 to adjust from a geometric mean to the maximum response regardless of direction
- deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the fault (1.8 is used to represent the 84th percentile response), but not less than 60% g.

As such, the values are different from those on the uniform-hazard 2008 USGS National Seismic Hazard Maps posted at: http://earthquake.usgs.gov/hazmaps.

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Luco, Nicolas, Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.

Petersen, M.D., Frankel, A.D., Harmsen, S.C., Mueller, C.S., Haller, K.M., Wheeler, R.L., Wesson, R.L., Zeng, Yuehua, Boyd, O.S., Perkins, D.M., Luco, Nicolas, Field, E.H., Wills, C.J., and Rukstales, K.S., 2008, Documentation for the 2008 Update of the United States National Seismic Hazard Maps: U.S. Geological Survey Open-File Report 2008-1128, 61 p.

FIGURE 22-2 S_1 Risk-Adjusted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for 1 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.

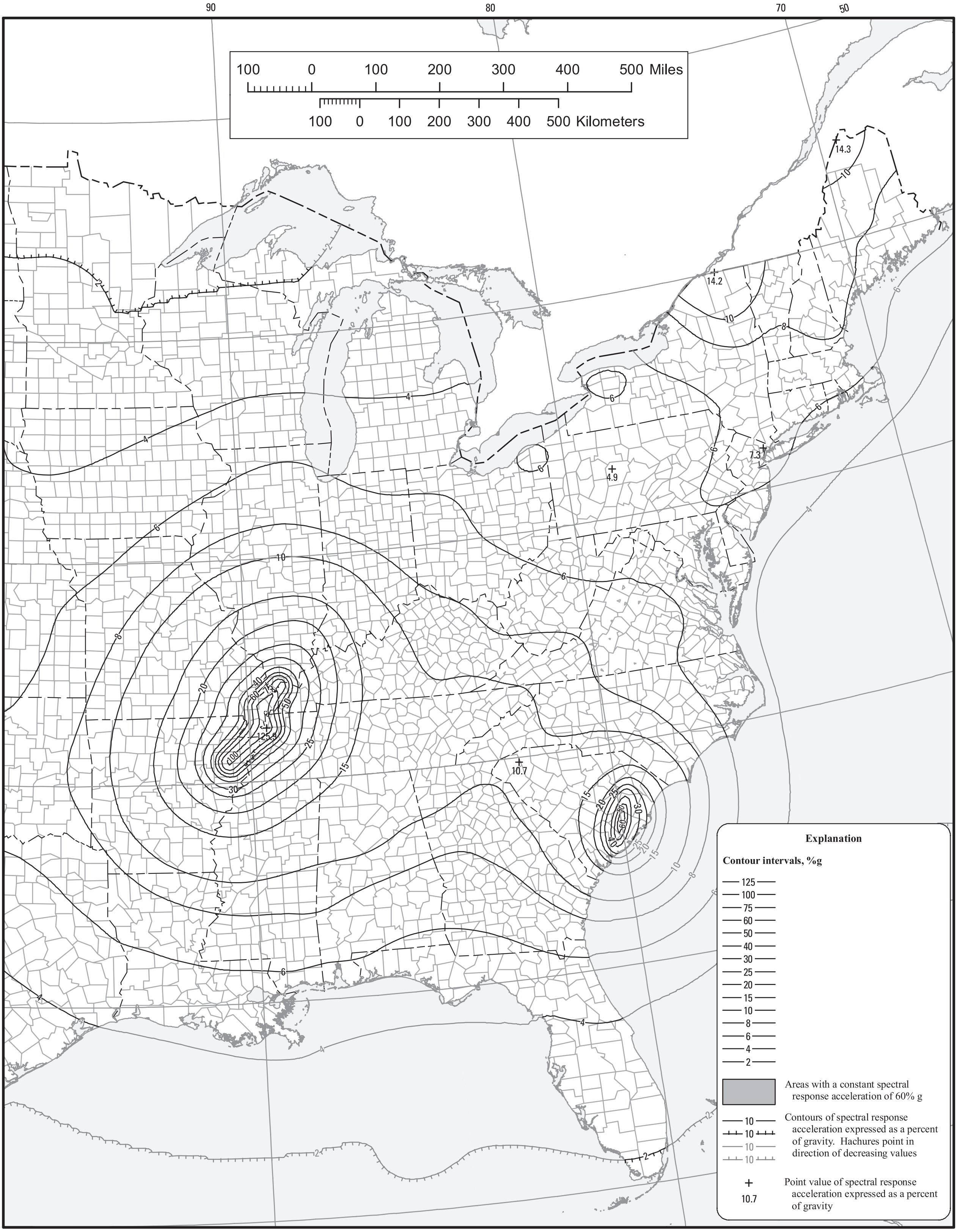
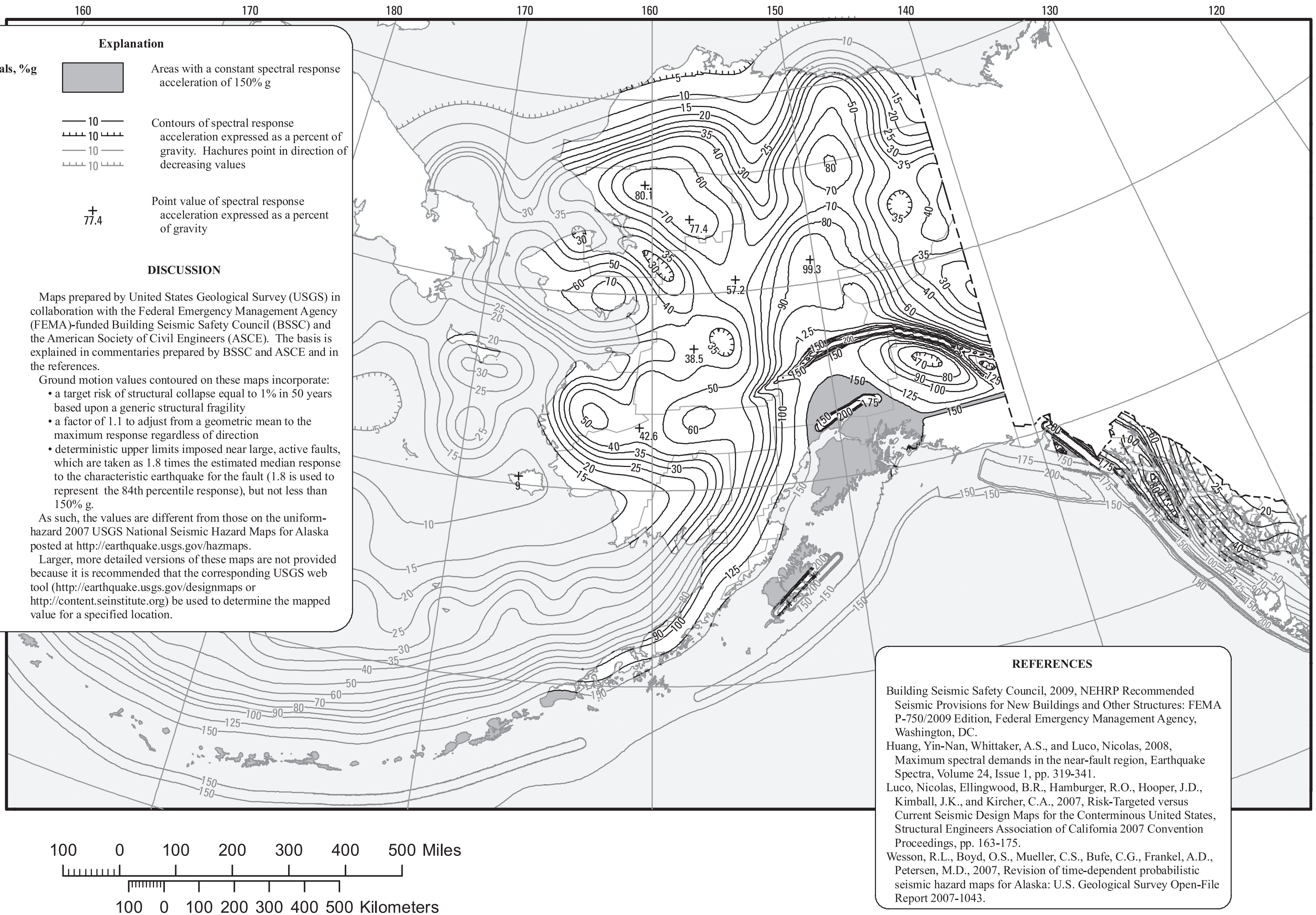


FIGURE 22-2 (*Continued*)



Explanation

Contour intervals, %g

200
175
150
125
100
90
80
70
60
50
40
35
30
25
20
15
10
5

Areas with a constant spectral response acceleration of 150% g

Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values

77.4 Point value of spectral response acceleration expressed as a percent of gravity

DISCUSSION

Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC) and the American Society of Civil Engineers (ASCE). The basis is explained in commentaries prepared by BSSC and ASCE and in the references.

Ground motion values contoured on these maps incorporate:

- a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
- a factor of 1.1 to adjust from a geometric mean to the maximum response regardless of direction
- deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the fault (1.8 is used to represent the 84th percentile response), but not less than 150% g.

As such, the values are different from those on the uniform-hazard 2007 USGS National Seismic Hazard Maps for Alaska posted at http://earthquake.usgs.gov/hazmaps.

Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps or http://content.seinstitute.org) be used to determine the mapped value for a specified location.

REFERENCES

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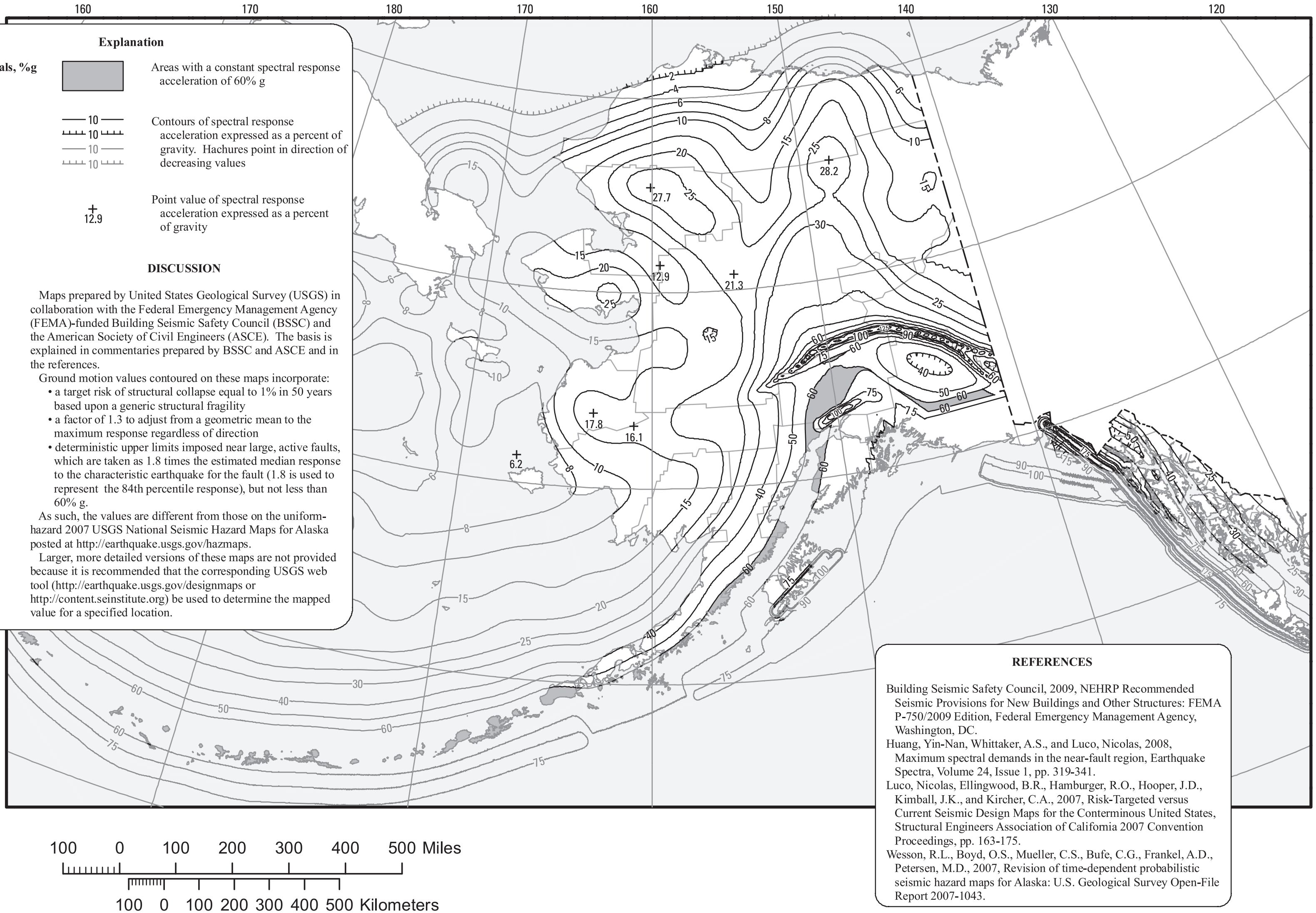
Huang, Yin-Nan, Whittaker, A.S., and Luco, Nicolas, 2008, Maximum spectral demands in the near-fault region, Earthquake Spectra, Volume 24, Issue 1, pp. 319-341.
Luco, Nicolas, Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.
Wesson, R.L., Boyd, O.S., Mueller, C.S., Bufe, C.G., Frankel, A.D., Petersen, M.D., 2007, Revision of time-dependent probabilistic seismic hazard maps for Alaska: U.S. Geological Survey Open-File Report 2007-1043.

FIGURE 22-3 S_S Risk-Adjusted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for Alaska for 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.



Explanation

Contour intervals, %g

175, 150, 125, 100, 90, 75, 60, 50, 40, 30, 25, 20, 15, 10, 8, 6, 4, 2

Areas with a constant spectral response acceleration of 60% g

Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values

Point value of spectral response acceleration expressed as a percent of gravity (12.9)

DISCUSSION

Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC) and the American Society of Civil Engineers (ASCE). The basis is explained in commentaries prepared by BSSC and ASCE and in the references.

Ground motion values contoured on these maps incorporate:

- a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
- a factor of 1.3 to adjust from a geometric mean to the maximum response regardless of direction
- deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the fault (1.8 is used to represent the 84th percentile response), but not less than 60% g.

As such, the values are different from those on the uniform-hazard 2007 USGS National Seismic Hazard Maps for Alaska posted at http://earthquake.usgs.gov/hazmaps.

Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps or http://content.seinstitute.org) be used to determine the mapped value for a specified location.

REFERENCES

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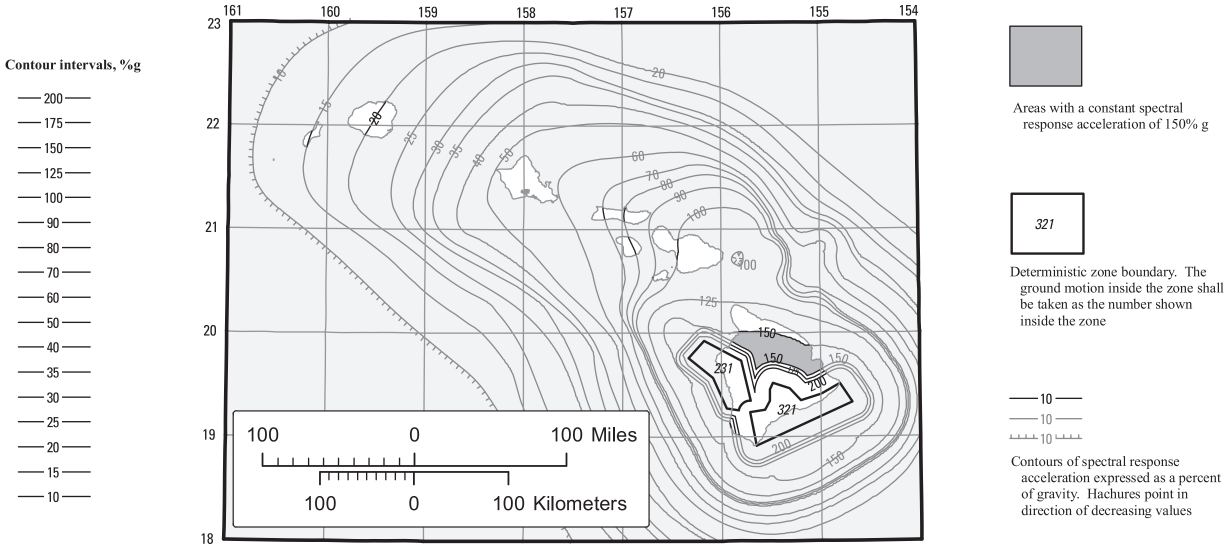
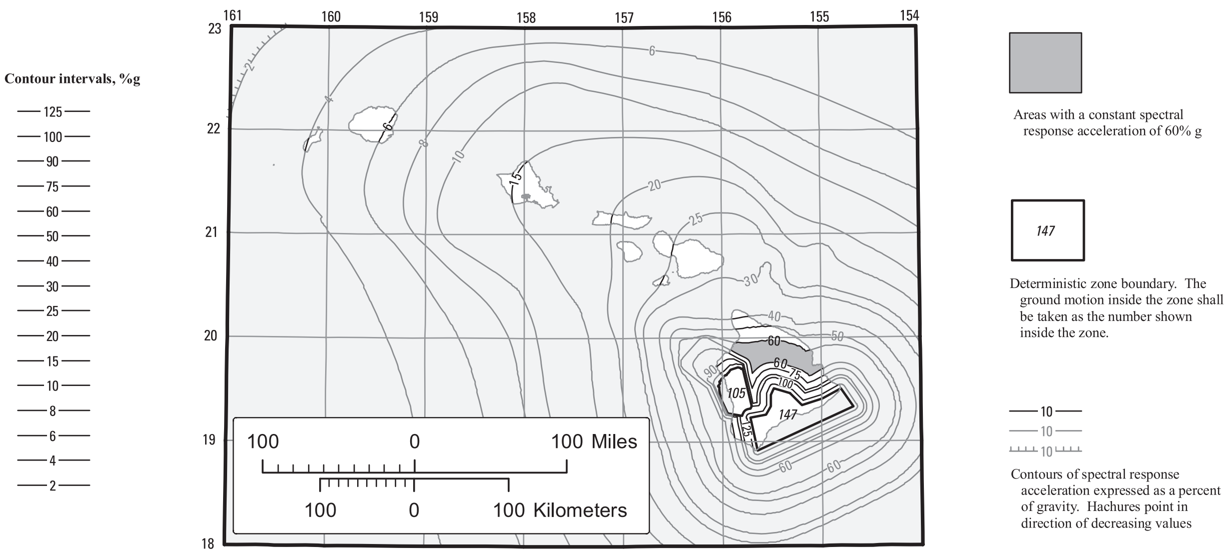
Luco, Nicolas, Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.

Wesson, R.L., Boyd, O.S., Mueller, C.S., Bufe, C.G., Frankel, A.D., Petersen, M.D., 2007, Revision of time-dependent probabilistic seismic hazard maps for Alaska: U.S. Geological Survey Open-File Report 2007-1043.

FIGURE 22-4 S_1 **Risk-Adjusted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for Alaska for 1.0s Spectral Response Acceleration (5% of Critical Damping), Site Class B.**



0.2 Second Spectral Response Acceleration (5% of Critical Damping)



1.0 Second Spectral Response Acceleration (5% of Critical Damping)

DISCUSSION

Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC) and the American Society of Civil Engineers (ASCE). The basis is explained in commentaries prepared by BSSC and ASCE and in the references.

Ground motion values contoured on these maps incorporate:

- a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
- deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the fault (1.8 is used to represent the 84th percentile response), but not less than 150% and 60% g for 0.2 and 1.0 sec, respectively.

As such, the values are different from those on the uniform-hazard 1998 USGS National Seismic Hazard Maps for Hawaii posted at http://earthquake.usgs.gov/hazmaps.

Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps or http://content.seinstitute.org) be used to determine the mapped value for a specified location.

REFERENCES

Building Seismic Safety Council, 2009, NEHRP Recommended Seismic Provisions for New Buildings and Other Structures: FEMA P-750/2009 Edition, Federal Emergency Management Agency, Washington, DC.

Huang, Yin-Nan, Whittaker, A.S., and Luco, Nicolas, 2008, Maximum spectral demands in the near-fault region, Earthquake Spectra, Volume 24, Issue 1, pp. 319-341.

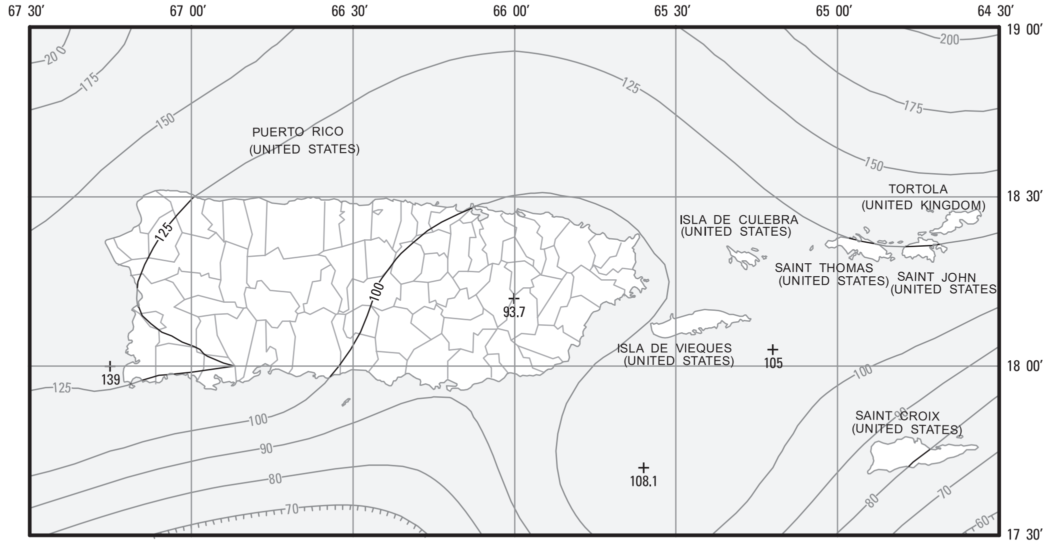
Klein, F., Frankel, A.D., Mueller, C.S., Wesson, R.L., and Okubo, P., 2001, Seismic hazard in Hawaii: high rate of large earthquakes and probabilistic ground-motion maps, Bulletin of the Seismological Society of America, Volume 91, pp. 479-498.

Luco, Nicolas, Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.

FIGURE 22-5 S_S and S_1 Risk-Adjusted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for Hawaii for 0.2 and 1.0 Spectral Response Acceleration (5% of Critical Damping), Site Class B.

Contour intervals, %g

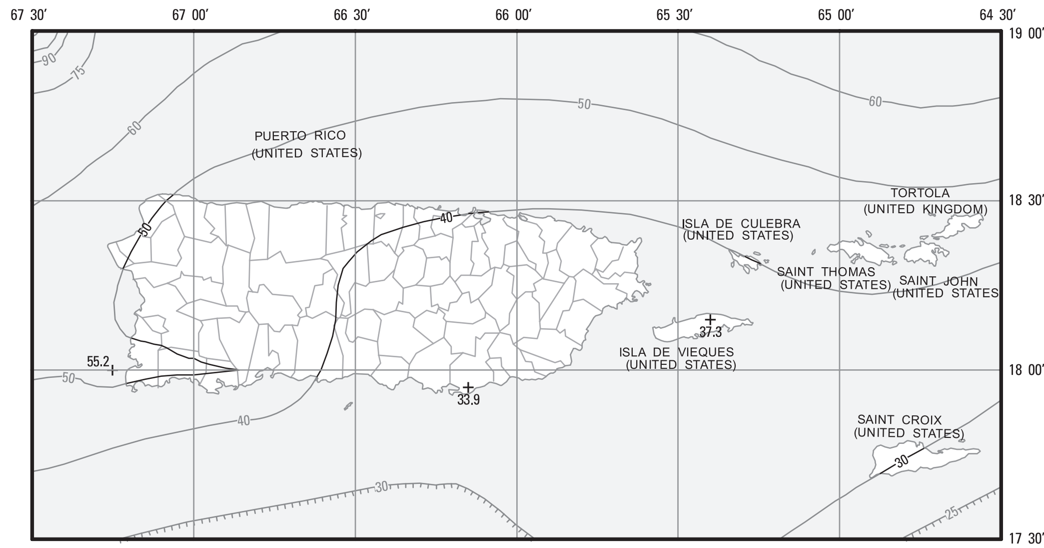
200
175
150
125
100
90
80
70
60



0.2 Second Spectral Response Acceleration (5% of Critical Damping)

Contour intervals, %g

125
100
90
75
60
50
40
30
25



1.0 Second Spectral Response Acceleration (5% of Critical Damping)

Explanation

10
10
10

Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values

93.7

Point value of spectral response acceleration expressed as a percent of gravity

DISCUSSION

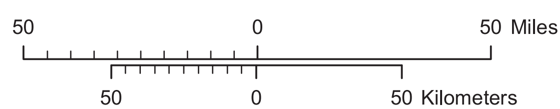
Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC) and the American Society of Civil Engineers (ASCE). The basis is explained in commentaries prepared by BSSC and ASCE and in the references.

Ground motion values contoured on these maps incorporate:

- a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
- a factor of 1.1 and 1.3 for 0.2 and 1.0 sec, respectively, to adjust from a geometric mean to the maximum response regardless of direction
- deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the fault (1.8 is used to represent the 84th percentile response), but not less than 150% and 60% g for 0.2 and 1.0 sec, respectively.

As such, the values are different from those on the uniform-hazard 2003 USGS National Seismic Hazard Maps for Puerto Rico and the U.S. Virgin Islands posted at http://earthquake.usgs.gov/hazmaps.

Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps or http://content.seinstitute.org) be used to determine the mapped value for a specified location.



REFERENCES

Building Seismic Safety Council, 2009, NEHRP Recommended Seismic Provisions for New Buildings and Other Structures: FEMA P-750/2009 Edition, Federal Emergency Management Agency, Washington, DC.

Huang, Yin-Nan, Whittaker, A.S., and Luco, Nicolas, 2008, Maximum spectral demands in the near-fault region, Earthquake Spectra, Volume 24, Issue 1, pp. 319-341.

Luco, Nicolas, Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.

Mueller, C.S., Frankel, A.D., Petersen, M.D., and Leyendecker, E.V., 2003, Documentation for the 2003 USGS Seismic Hazard Maps for Puerto Rico and the U.S. Virgin Islands: U.S. Geological Survey Open-File Report 03-379.

FIGURE 22-6 S_S and S_1 Risk-Adjusted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for Puerto Rico and the United States Virgin Islands for 0.2 and 1.0 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.

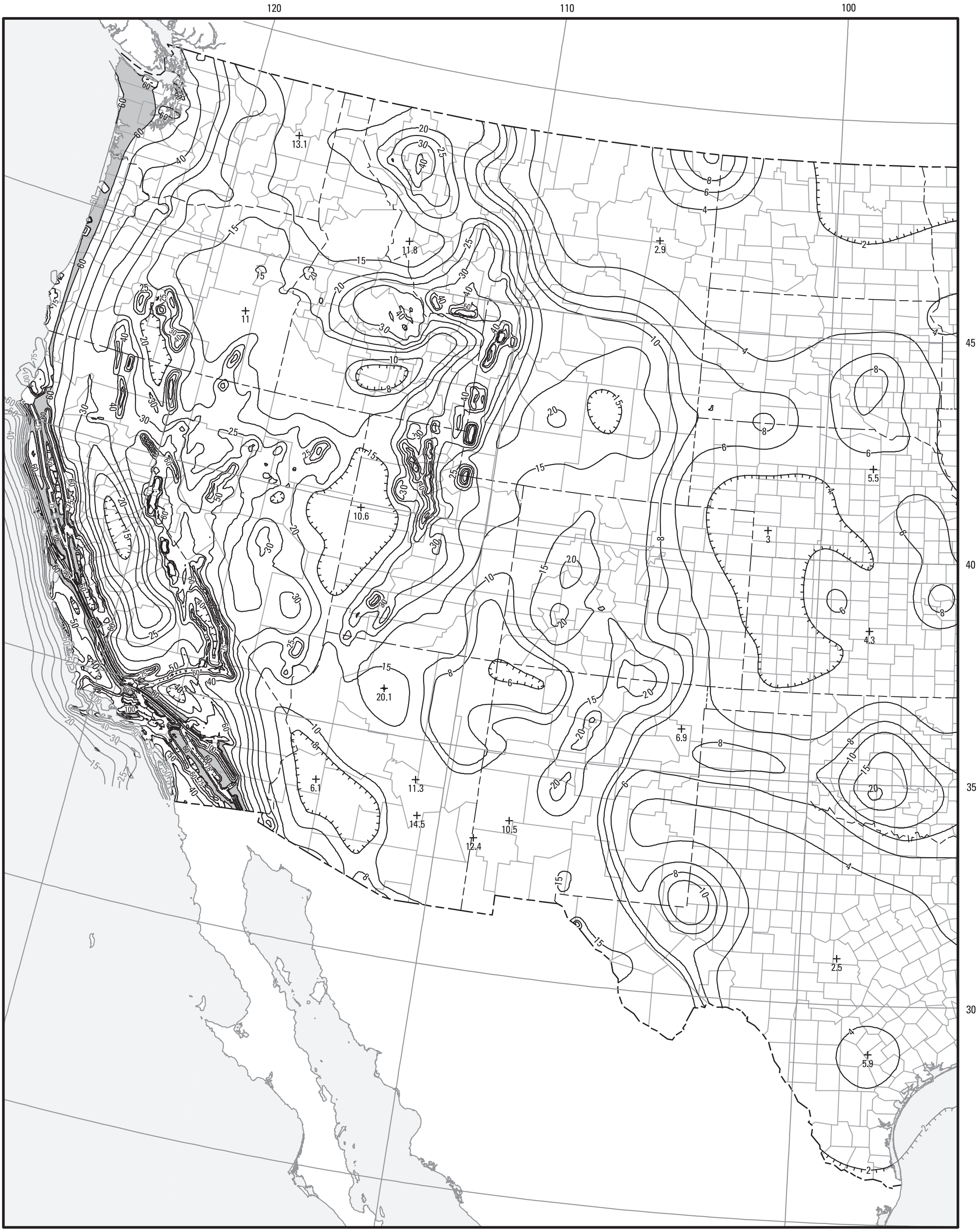


FIGURE 22-7 Maximum Considered Earthquake Geometric Mean (MCE_G) PGA, %*g*, Site Class B for the Conterminous United States.

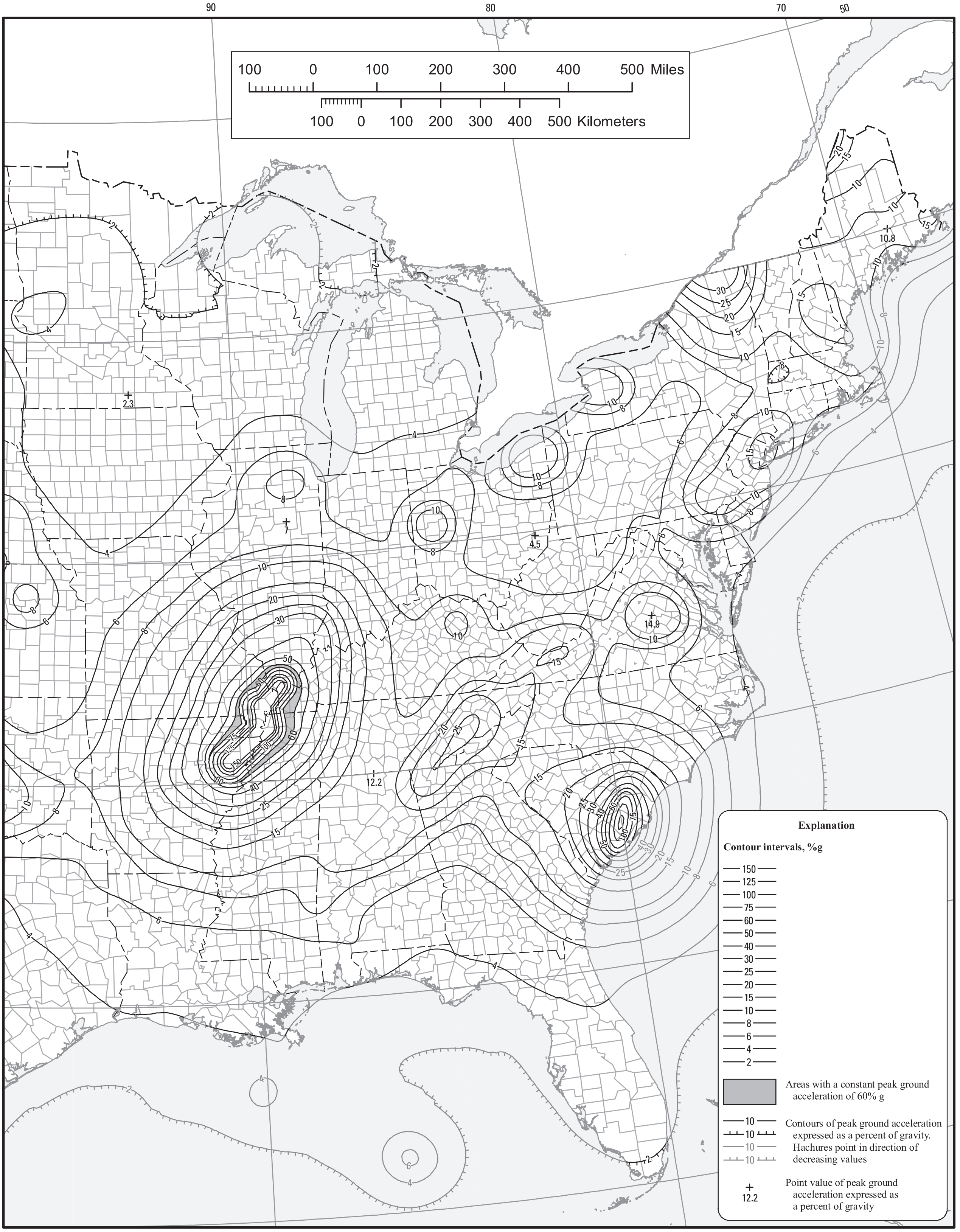


FIGURE 22-7 (*Continued*)

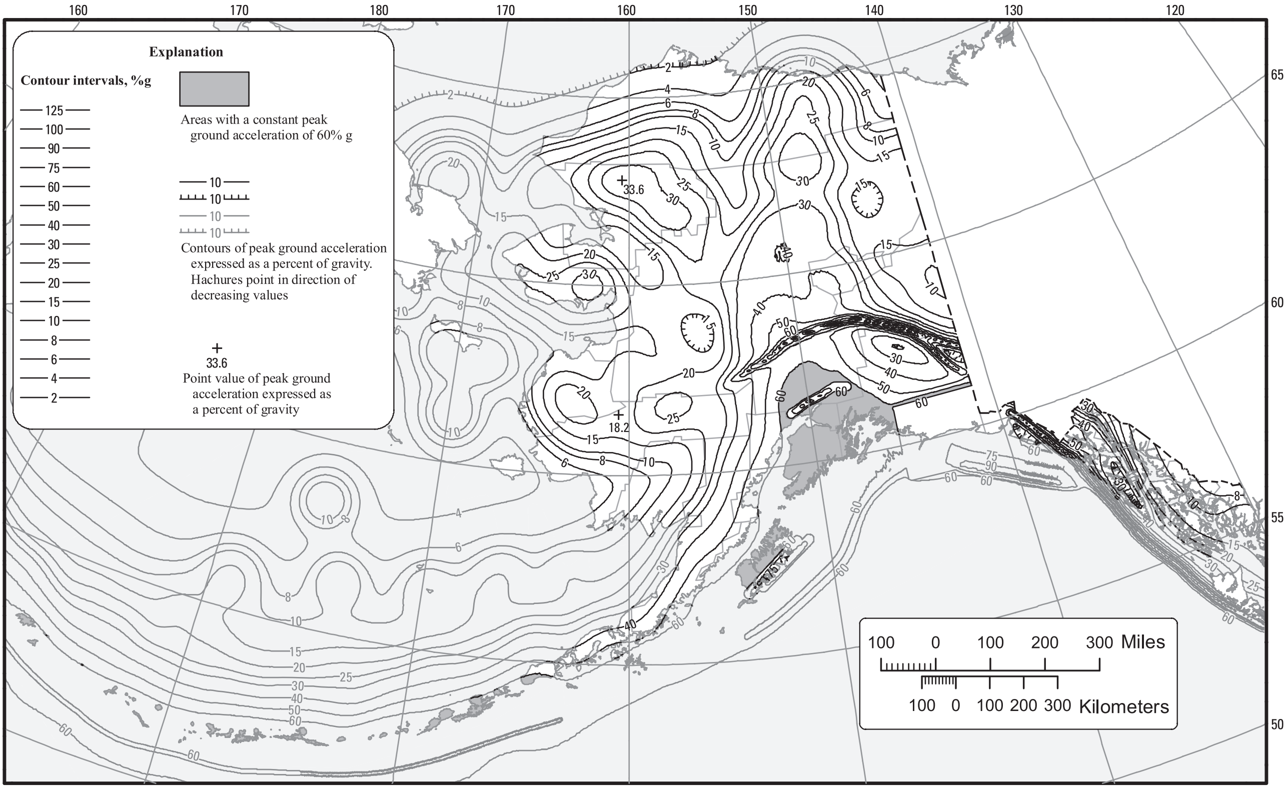


FIGURE 22-8 Maximum Considered Earthquake Geometric Mean (MCE_G) PGA, %*g*, Site Class B for Alaska.

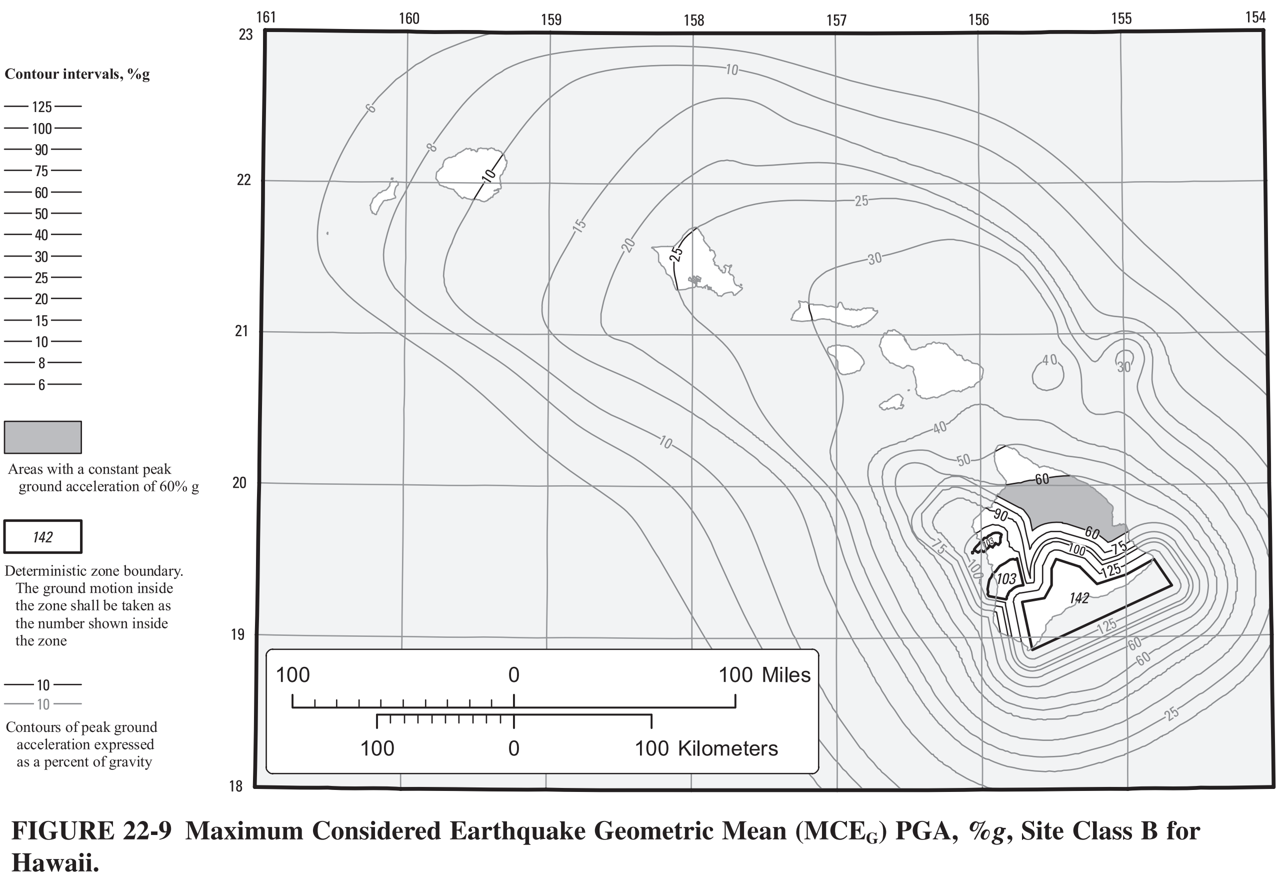


FIGURE 22-9 Maximum Considered Earthquake Geometric Mean (MCE_G) PGA, %*g*, Site Class B for Hawaii.

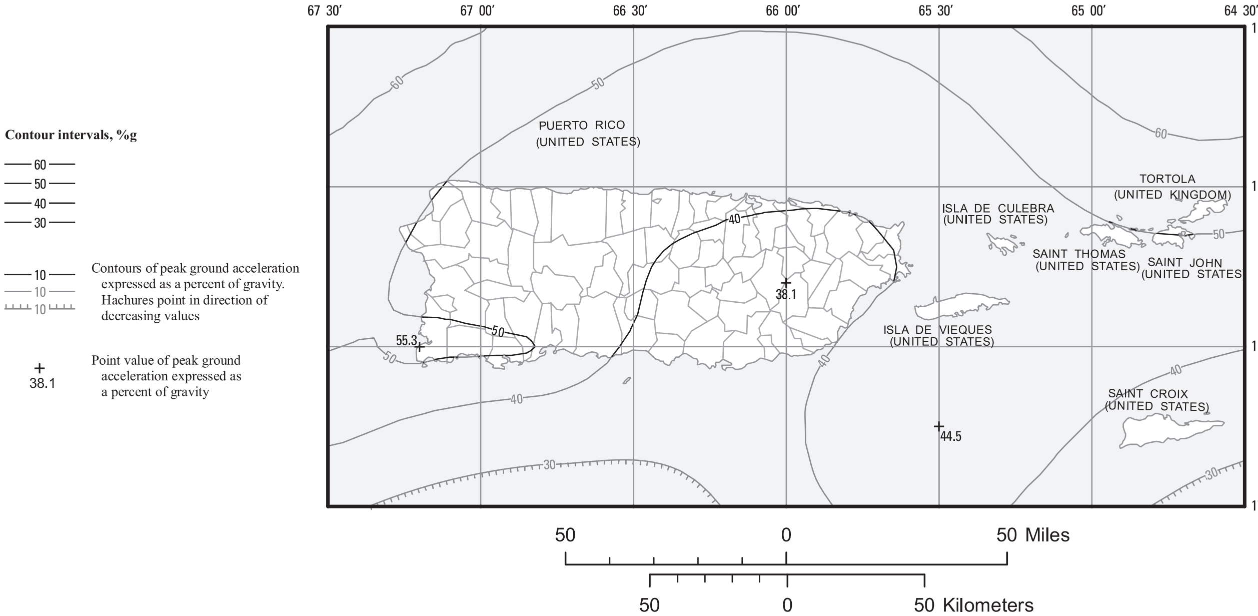


FIGURE 22-10 Maximum Considered Earthquake Geometric Mean (MCE_G) PGA, %*g*, Site Class B for Puerto Rico and the United States Virgin Islands.

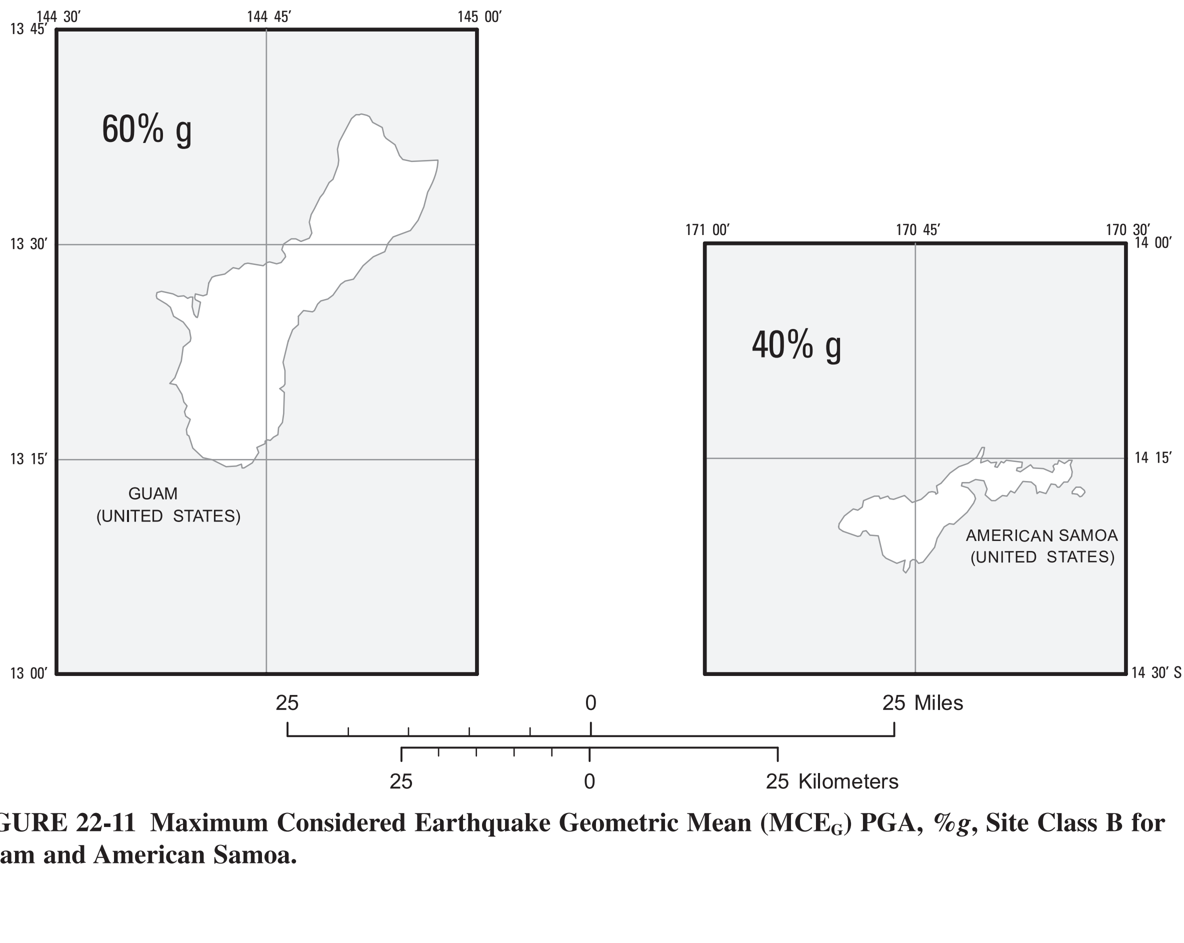


FIGURE 22-11 Maximum Considered Earthquake Geometric Mean (MCE_G) PGA, %*g*, Site Class B for Guam and American Samoa.

FIGURE 22-12 Mapped Long-Period Transition Period, T_L (s), for the Conterminous United States.

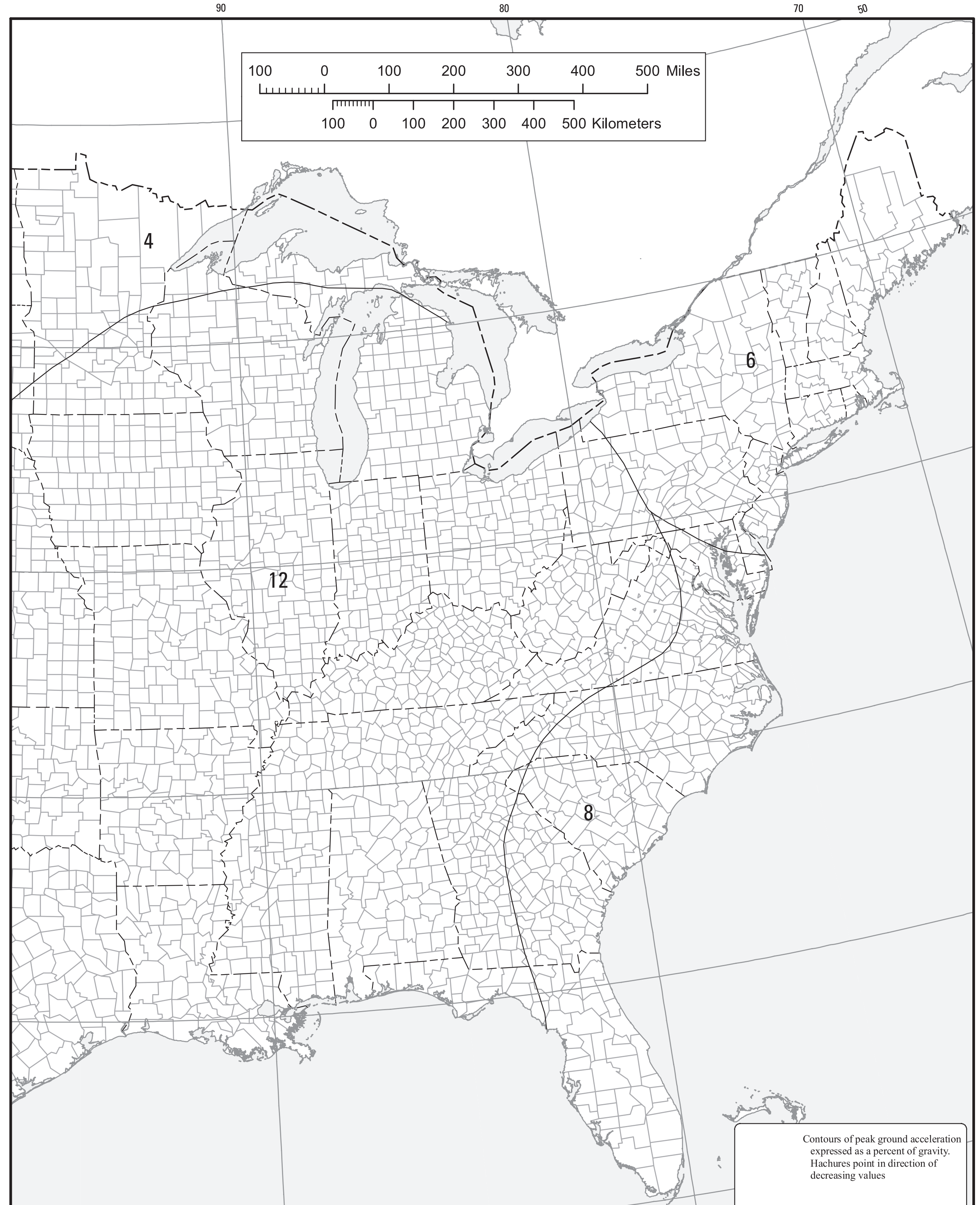


FIGURE 22-12 (*Continued*)

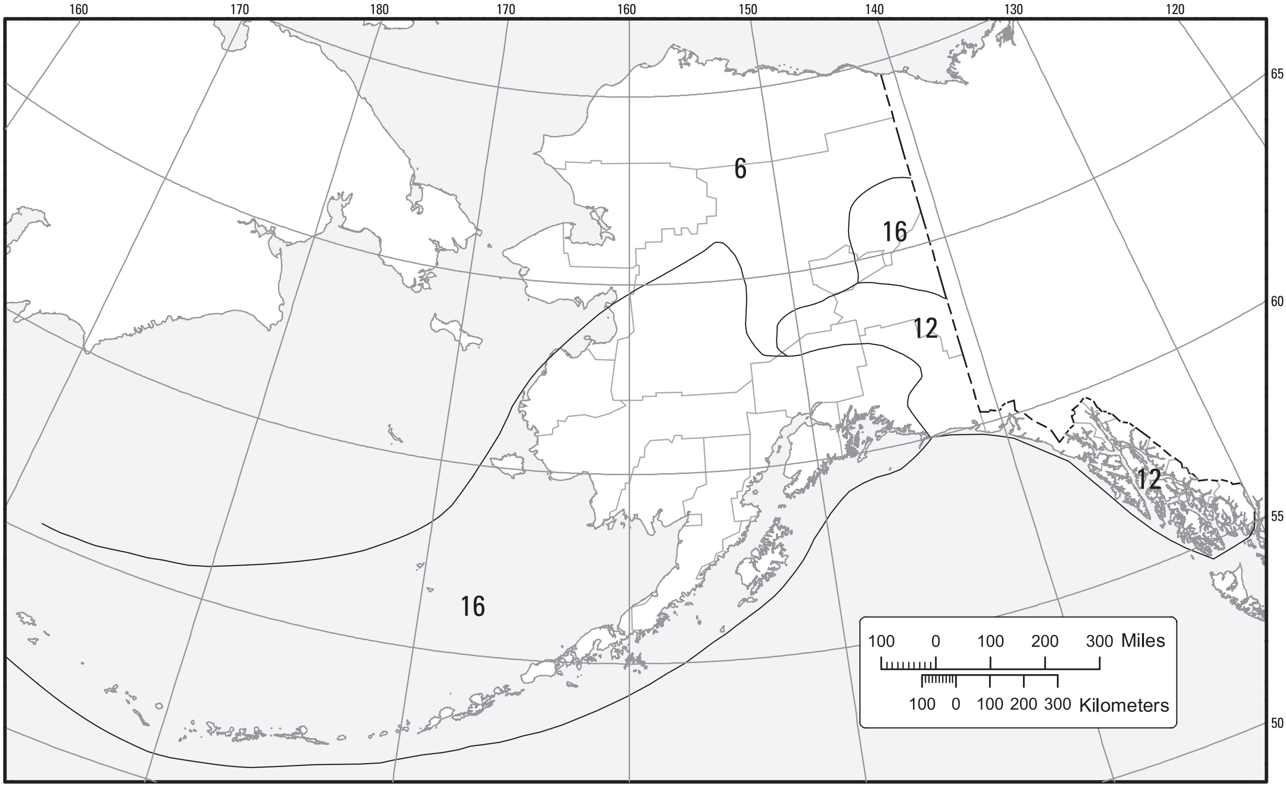


FIGURE 22-13 Mapped Long-Period Transition Period, T_L (s), for Alaska.

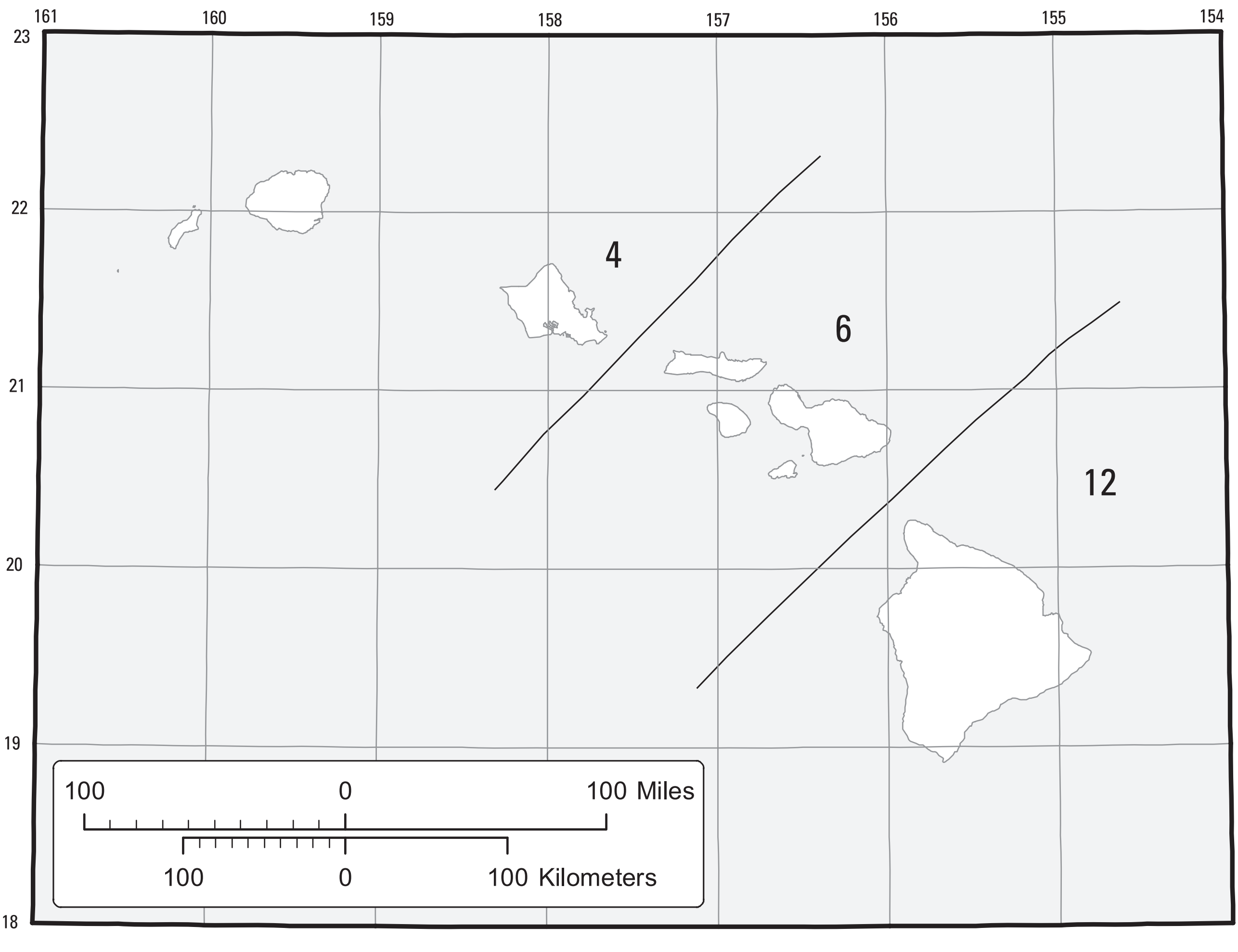


FIGURE 22-14 Mapped Long-Period Transition Period, T_L (s), for the Hawaii.

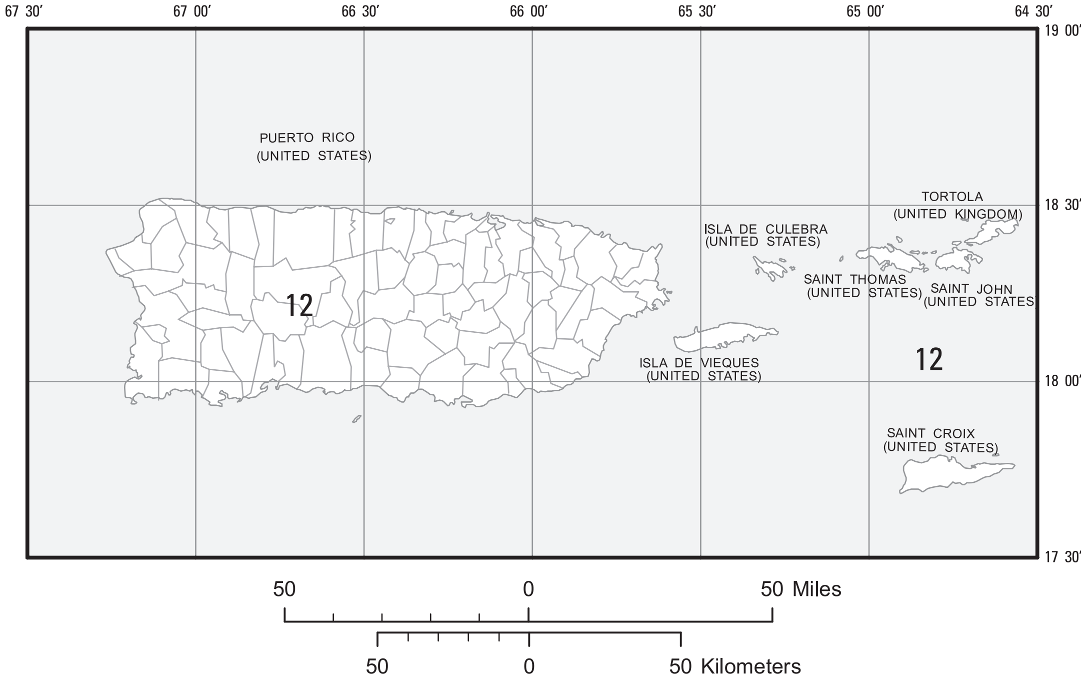


FIGURE 22-15 Mapped Long-Period Transition Period, T_L (s), for Puerto Rico and the United States Virgin Islands.

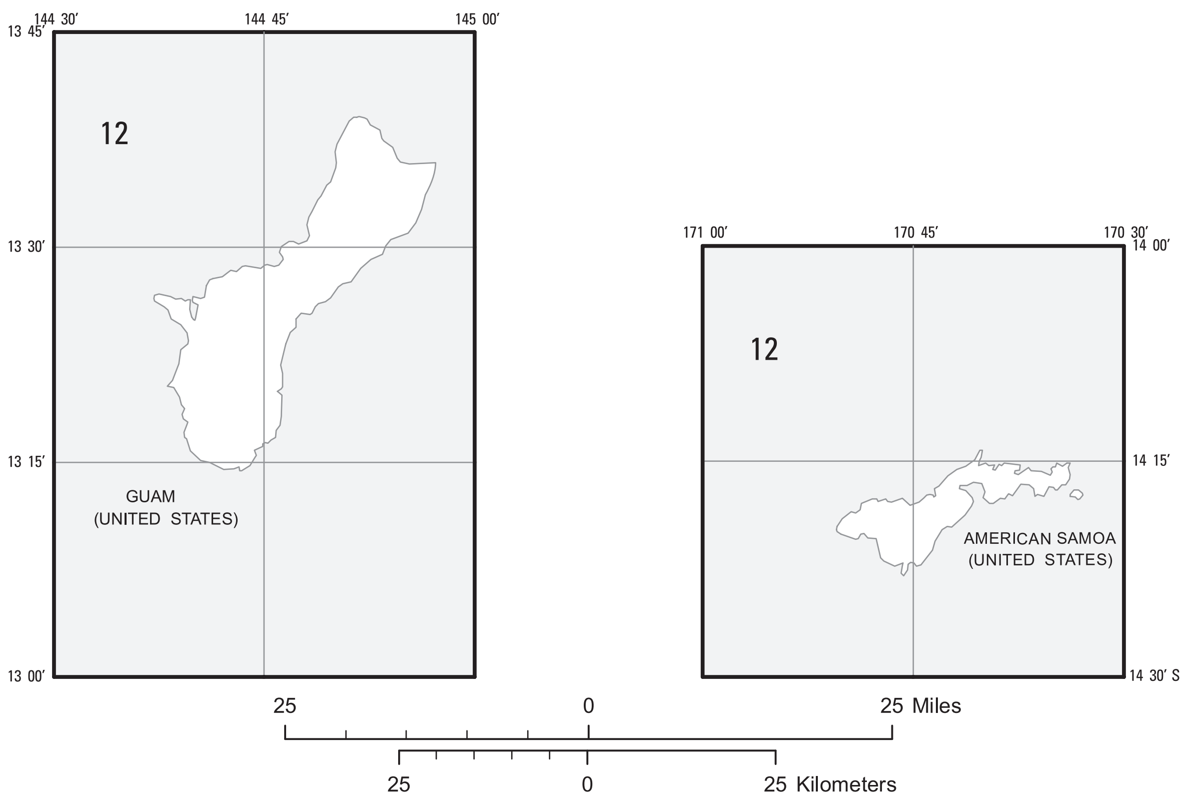


FIGURE 22-16 Mapped Long-Period Transition Period, T_L (s), for Puerto Guam and American Samoa.

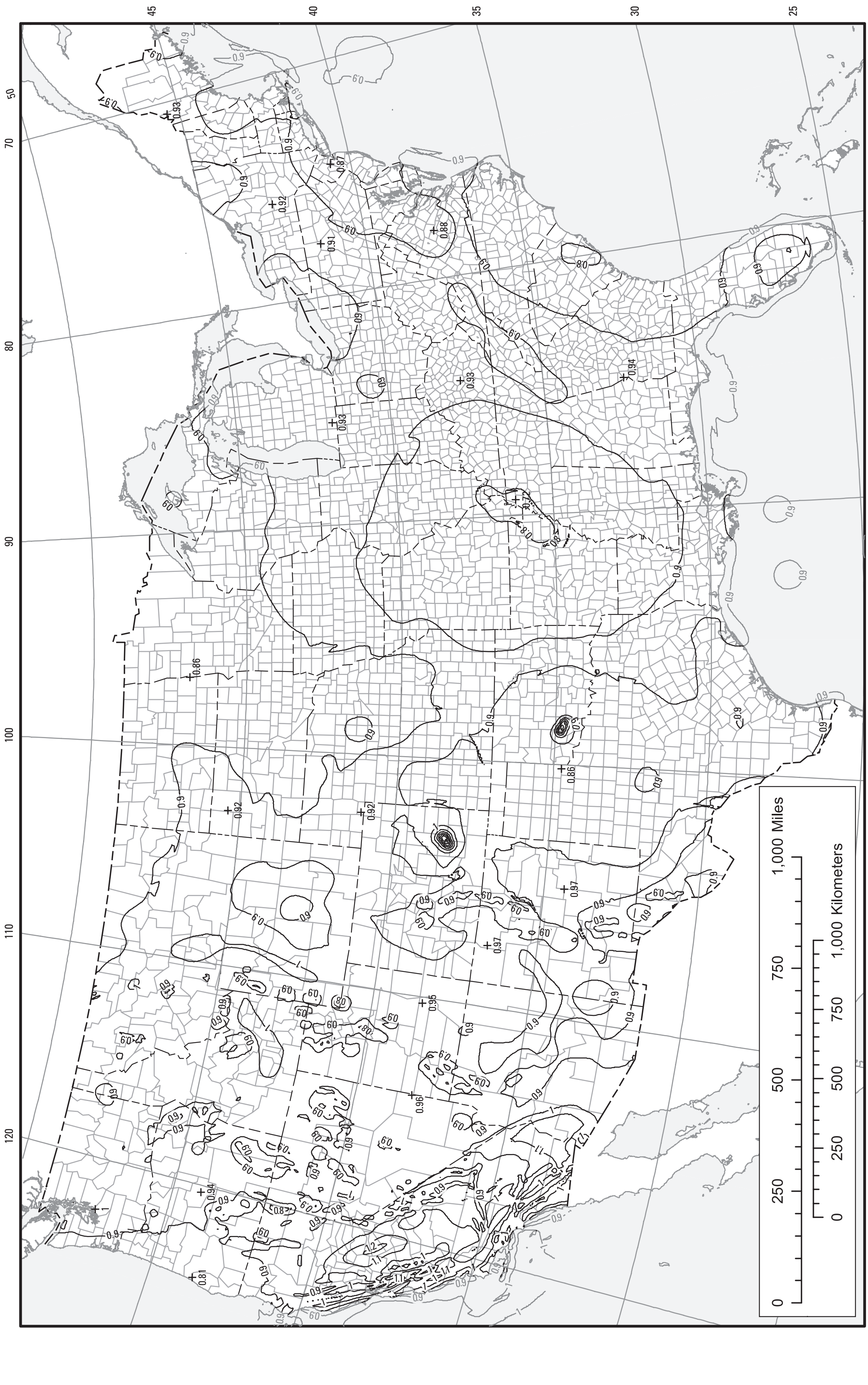
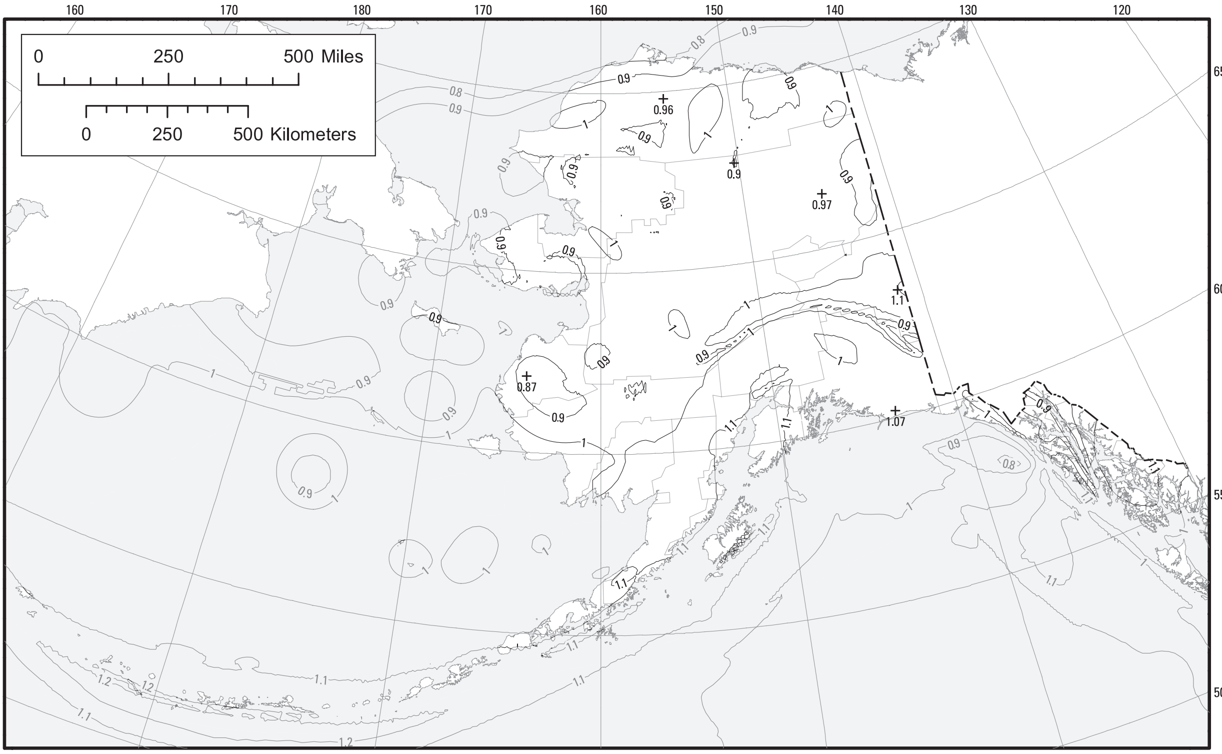
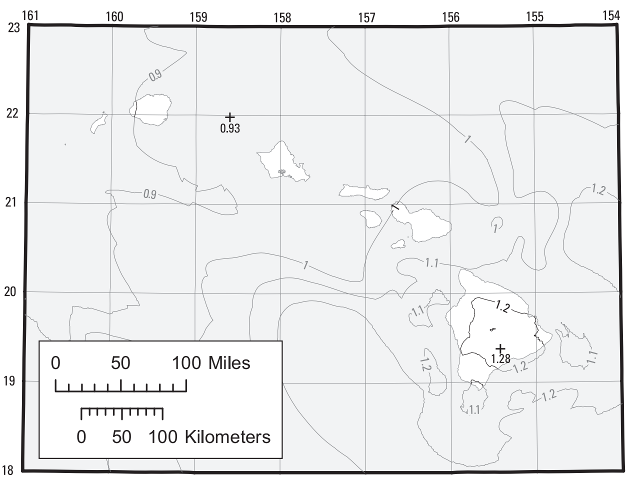
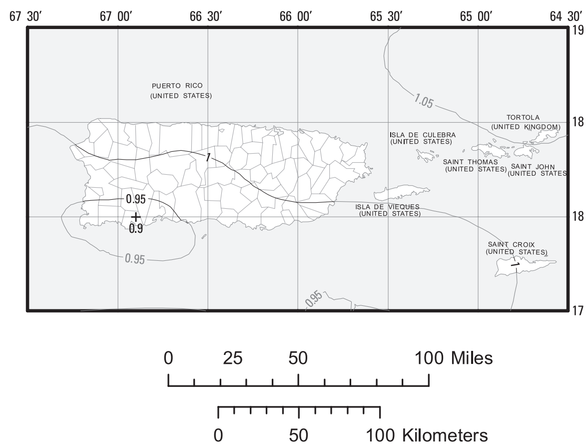


FIGURE 22-17 Mapped Risk Coefficient at 0.2 s Spectral Response Period, C_{RS}.







Notes:
- Maps prepared by United States Geological Survey (USGS).
- Larger, more detailed versions of these maps are not included because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps/) be used to determine the mapped value for a specified location.

FIGURE 22-17 (*Continued*)

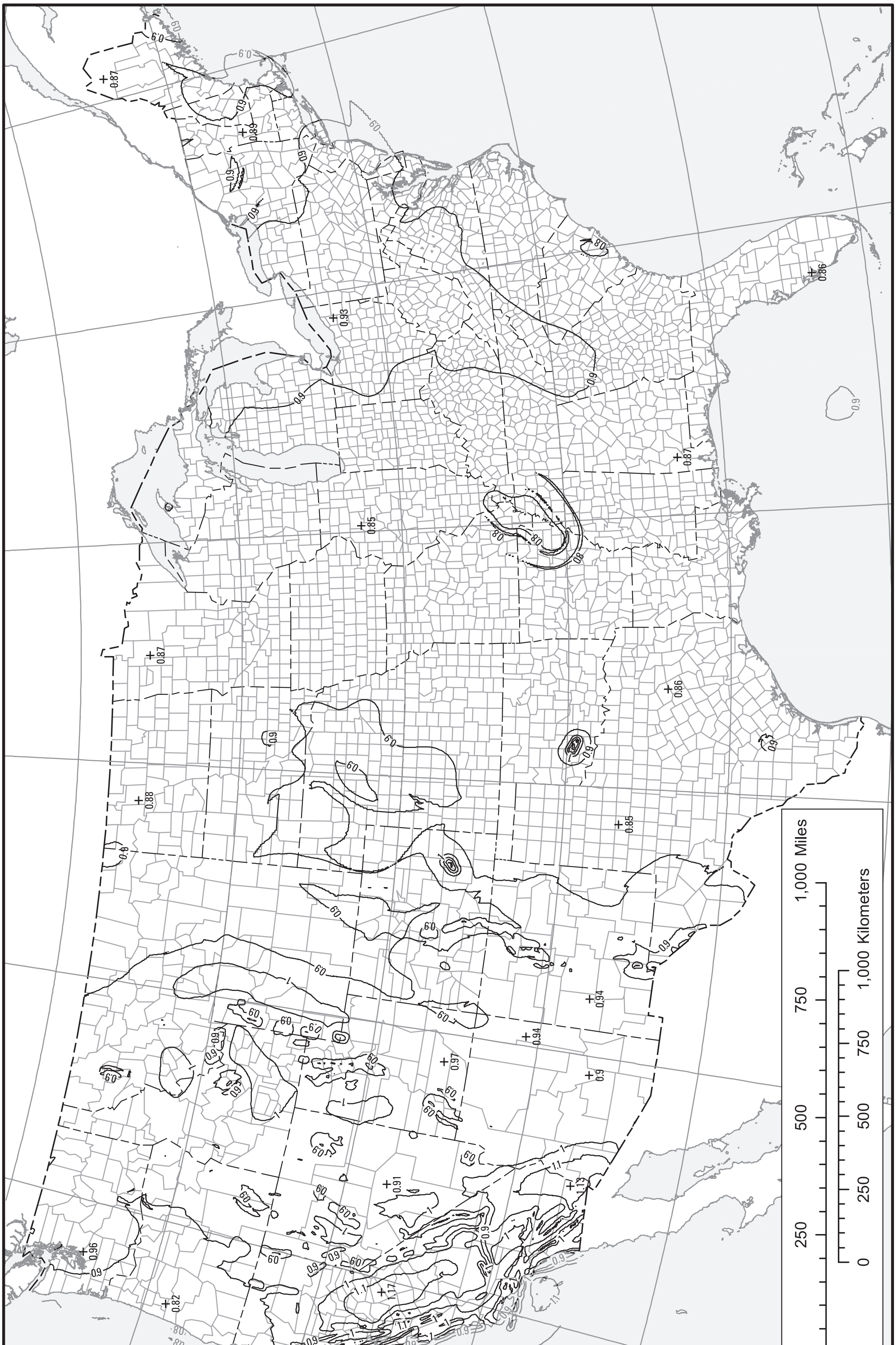
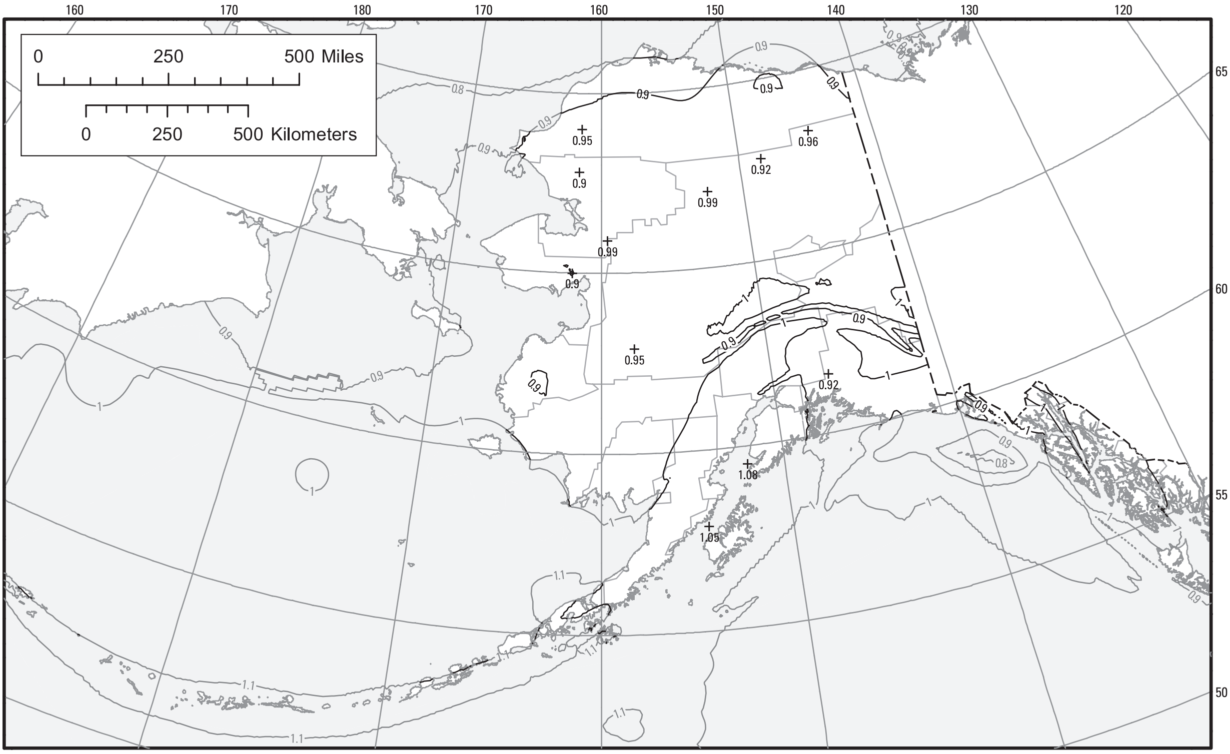
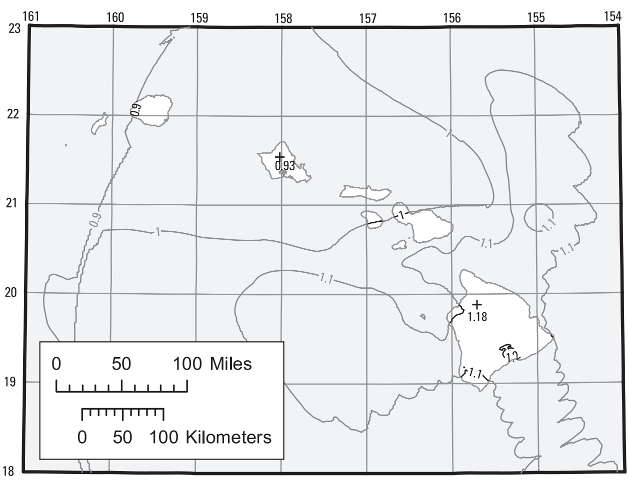
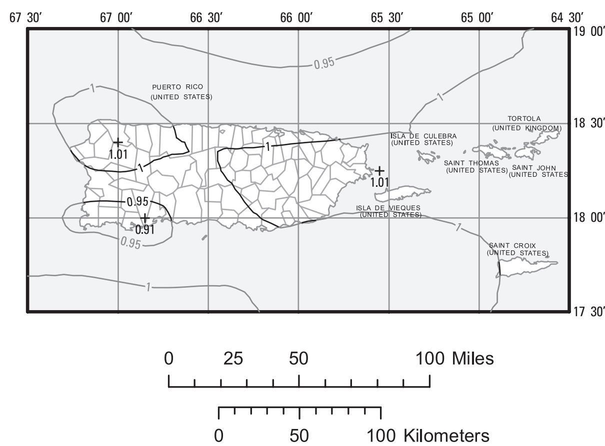


FIGURE 22-18 Mapped Risk Coefficient at 1.0 s Spectral Response Period, C_{R1}.







Notes:
- Maps prepared by United States Geological Survey (USGS).
- Larger, more detailed versions of these maps are not included because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps/) be used to determine the mapped value for a specified location.

FIGURE 22-18 (*Continued*)

Chapter 23
SEISMIC DESIGN REFERENCE DOCUMENTS

23.1 CONSENSUS STANDARDS AND OTHER REFERENCE DOCUMENTS

This section lists the reference documents that are referenced in Chapters 11 through 22. The reference documents are listed herein by the promulgating agency of the reference document, the reference document identification, the section(s), and tables of ASCE 7 that cite the reference document, the title, and effective date. Unless identified by an asterisk, the following reference documents are consensus standards and are to be considered part of this standard to the extent referenced in the specified section. Those reference documents identified by an asterisk (*) are documents developed within the industry and represent acceptable procedures for design and construction to the extent referred to in the specified section.

AAMA
American Architectural Manufacturers Association
1827 Waldon Office Square
Suite 104
Schaumburg, IL 60173

***AAMA 501.6**
Section 13.5.9.2
Recommended Dynamic Test Method for Determining the Seismic Drift Causing Glass Fallout from a Wall System, 2001

ACI
American Concrete Institute
P.O. Box 9094
Farmington Hills, MI 48333-9094

ACI 318
Sections 14.2.2, 14.2.2.1, 14.2.2.2, 14.2.2.3, 14.2.2.4, 14.2.2.5, 14.2.2.6, 14.2.2.7, 14.2.2.8, 14.2.2.9, 14.2.3, 14.2.3.1.1, 14.2.3.2.1, 14.2.3.2.2, 14.2.3.2.3, 14.2.3.2.5, 14.2.3.2.6, 14.3.1, 14.4.4.2.2, 14.4.5.2
Building Code Requirements for Structural Concrete and Commentary, 2008

ACI 355.2
Section 13.4.2
Qualification of Post-Installed Mechanical Anchors in Concrete and Commentary, 2007

ACI 530
Sections 14.4.1, 14.4.2, 14.4.3, 14.4.3.1, 14.4.4, 14.4.4.1, 14.4.4.2.2, 14.4.5, 14.4.5.1, 14.4.5.2, 14.4.5.3, 14.4.5.4, 14.4.5.5, 14.4.5.6, 14.4.6, 14.4.6.1, 15.4.9.2
Building Code Requirements for Masonry Structures, 2008

ACI 530.1
Sections 14.4.1, 14.4.2, 14.4.7, 14.4.7.1
Specification for Masonry Structures, 2008

ACI 313
Sections 15.7.9.3.3, 15.7.9.6, 15.7.9.7
Standard Practice for the Design and Construction of Concrete Silos and Stacking Tubes for Storing Granular Materials, 1997

***ACI 371R**
Section 15.7.10.7
Guide to the Analysis, Design, and Construction of Concrete-Pedestal Water Towers, 1998

ACI 350.3
Sections 15.7.6.1.1, 15.7.7.3
Standard Practice for the Seismic Design of Liquid-Containing Concrete Structures, 2006

AF&PA
American Forest and Paper Association
1111 19th Street NW
Suite 800
Washington, DC 20036

AF&PA NDS
Sections 12.4.3.3, 12.14.2.2.2.3, 14.5.1
National Design Specification for Wood Construction, Including Supplements, AF&PA NDS-05, 2005

AF&PA SDPWS
Sections 12.14.6.2, 14.5.1, 14.5.3, 14.5.3.1
AF&PA Special Design Provisions for Wind and Seismic, 2008

AISC
American Institute of Steel Construction
One East Wacker Drive
Suite 700
Chicago, IL 60601-2001

ANSI/AISC 360
Sections 14.1.1, 14.1.2.1, 14.1.2.2, 14.3.1, 14.3.2, 11A.1.3.6.2
Specification for Structural Steel Buildings, 2010

ANSI/AISC 341
Sections 14.1.1, 14.1.2.2, 14.3.1, 14.3.3, 11A1.3.6, 11A.2.4
Seismic Provisions for Structural Steel Buildings, 2010

AISI
American Iron and Steel Institute
1140 Connecticut Avenue
Suite 705
Washington, DC 20036

ANSI/AISI S100
Sections 14.1.1, 14.1.31, 14.1.3.2, 14.1.4.1, 14.1.5
North American Specification for the Design of Cold-Formed Steel Structural Members, 2007

ANSI/AISI S110
Sections 14.1.1, 14.1.3.2, 14.1.3.3, Table 12.2-1
Standard for Seismic Design of Cold-Formed Steel Structural Systems—Special Bolted Moment Frames, 2007

ANSI/AISI S230 with S2-08
Sections 14.1.1, 14.1.4.3
Standard for Cold-Formed Steel Framing—Prescriptive Method for One- and Two-Family Dwellings, 2007, with Supplement 2, 2008

ANSI/AISI S213 with S1-09
Sections 12.14.7.2, 14.1.1, 14.1.2, 14.1.4.2
North American Standard for Cold-Formed Steel Framing—Lateral Design, 2007, with Supplement 1, 2009

API
American Petroleum Institute
1220 L Street
Washington, DC 20005-4070

API 12B
Section 15.7.8.2
Bolted Tanks for Storage of Production Liquids, *Specification 12B,* 14th edition, 1995

API 620
Sections 15.4.1, 15.7.8.1, 15.7.13.1
Design and Construction of Large, Welded, Low Pressure Storage Tanks, 11th edition, Addendum 1, 2009

API 650
Sections 15.4.1, 15.7.8.1, 15.7.9.4
Welded Steel Tanks for Oil Storage, 11th Edition, Addendum 1, 2008

API 653
Section 15.7.6.1.9
Tank Inspection, Repair, Alteration, and Reconstruction, 3rd edition, 2001

ASCE/SEI
American Society of Civil Engineers
Structural Engineering Institute
1801 Alexander Bell Drive
Reston, VA 20191-4400

ASCE 4
Section 12.9.3
Seismic Analysis of Safety-Related Nuclear Structures, 1986

ASCE 5
Sections 14.4.1, 14.4.2, 14.4.3, 14.4.3.1, 14.4.4, 14.4.4.1, 14.4.4.2.2, 14.4.5, 14.4.5.1, 14.4.5.2, 14.4.5.3, 14.4.5.4, 14.4.5.5, 14.4.5.6, 14.4.6, 14.4.6.1, 15.4.9.2
Building Code Requirements for Masonry Structures, 2008

ASCE 6
Sections 14.4.1, 14.4.2, 14.4.7, 14.4.7.1
Specification for Masonry Structures, 2008

ASCE 8
Sections 14.1.1, 14.1.3.1, 14.13.2, 14.1.5
Specification for the Design of Cold-Formed Stainless Steel Structural Members, 2002

ASCE 19
Sections 14.1.1, 14.1.6
Structural Applications for Steel Cables for Buildings, 1996

ASME
American Society of Mechanical Engineers
Three Park Avenue
New York, NY 10016-5900

ASME A17.1
Sections 13.6.10, 13.6.10.3
Safety Code for Elevators and Escalators, 2004

ASME B31 (consists of the following listed standards)
Sections 13.6.5.1, 13.6.8.1, 13.6.8.4
Table 13.6-1
Power Piping, ASME B31.1, 2001
Process Piping, ASME B31.3, 2002

Liquid Transportation Systems for Hydrocarbons, Liquid Petroleum Gas, Anhydrous Ammonia, and Alcohols, ASME B31.4, 2002
Refrigeration Piping, ASME B31.5, 2001
Building Services Piping, ASME B31.9, 1996
Slurry Transportation Piping Systems, ASME B31.11, 2002
Gas Transmission and Distribution Piping Systems, ASME B31.8, 1999

ASME BPVC-01
Sections 13.6.9, 13.6.11, 15.7.11.2, 15.7.11.6, 15.7.12.2
Boiler and Pressure Vessel Code, 2004 excluding Section III, Nuclear Components, and Section XI, In-Service Inspection of Nuclear Components

ASTM
ASTM International
100 Barr Harbor Drive
West Conshohocken, PA 19428-2959

ASTM A421/A421M
Section 14.2.2.4
Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete, 2002

ASTM A435
Section 11A.2.5
Specification for Straight Beam Ultrasound Examination of Steel Plates, 2001

ASTM A615/A615M
Section 14.2.2.4
Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement, 2004b

ASTM A706/A706M
Sections 14.2.2.4, 14.4.9
Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement, 2004b

ASTM A722 /A722M
Section 14.2.2.4
Standard Specification for Uncoated High-Strength Steel Bars for Prestressing Concrete, 2003

ASTM A898/A898M
Section 11A.2.5
Specification for Straight Beam Ultrasound Examination of Rolled Steel Structural Shapes, 2001

ASTM C635
Section 13.5.6.2.2
Standard Specification for the Manufacture, Performance, and Testing of Metal Suspension Systems for Acoustical Tile and Lay-in Panel Ceilings, 2004

ASTM C636
Section 13.5.6.2.2
Standard Practice for Installation of Metal Ceiling Suspension Systems for Acoustical Tile and Lay-in Panels, 2004

ASTM D1586
Sections 11.3, 20.4.2
Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils, 2004

ASTM D2166
Sections 11.3, 20.4.3
Standard Test Method for Unconfined Compressive Strength of Cohesive Soil, 2000

ASTM D2216
Sections 11.3, 20.4.3
Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass, 1998

ASTM D2850
Sections 11.3, 20.4.3
Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils, 2003a

ASTM D4318
Sections 11.3, 20.4.3
Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils, 2000

AWWA
American Water Works Association
6666 West Quincy Avenue
Denver, CO 80235

AWWA D100
Sections 15.4.1, 15.7.7.1, 15.7.9.4, 15.7.10.6, 15.7.10.6.2
Welded Steel Tanks for Water Storage, 2006

AWWA D103
Sections 15.4.1, 15.7.7.2, 15.7.9.5
Factory-Coated Bolted Steel Tanks for Water Storage, 1997

AWWA D110
Section 15.7.7.3
Wire- and Strand-Wound Circular Prestressed Concrete Water Tanks, 2004

AWWA D115
Section 15.7.7.3
Tendon-Prestressed Concrete Water Tanks, 2006

ICC
International Code Council
5203 Leesburg Pike
Suite 600
Falls Church, VA 22041

*** IRC**
Section 11.1.2
2003 International Residential Code, 2003

ICC-ES
International Code Council Evaluation Service
5360 Workman Mill Road
Whittier, CA 90601

***ICC-ES AC 156-~~04~~ effective January 1, 2007**
Section 13.2.5
Acceptance Criteria for Seismic Qualification by Shake-Table Testing of Nonstructural Components and Systems, 2007

MSS
Manufacturers Standardization Society of the Valve and Fitting Industry
127 Park Street NE
Vienna, VA 22180

***MSS SP-58**
Section 13.6.5.1
Pipe Hangers and Supports—Materials, Design, and Manufacture, 2002

NFPA
National Fire Protection Association
1 Batterymarch Park
Quincy, MA 02269-9101

NFPA 13
Sections 13.4.6, 13.6.5.1, 13.6.8, 13.6.8.2
Standard for the Installation of Sprinkler Systems, 2007

NFPA 59A
Section 15.4.8
Production, Storage, and Handling of Liquefied Natural Gas (LNG), 2006

RMI
Rack Manufacturers Institute
8720 Red Oak Boulevard
Suite 201
Charlotte, NC 28217

ANSI/MH 16.1
Section 15.5.3
Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks, 2008

SJI
Steel Joist Institute
1173 B London Links Drive
Forest, VA 24551

ANSI/SJI-K-1.1
Section 14.1.1
Standard Specifications for Open Web Steel Joists, K-Series, 2005

ANSI/SJI-LH/DLH-1.1
Section 14.1.1
Standard Specifications for Longspan Steel Joists, LH-Series and Deep Longspan Steel Joists, DLH-Series, 2005

ANSI/SJI-JG-1.1
Section 14.1.1
Standard Specifications for Joist Girders, 2005

ANSI/SJI-CJ-1.0
Section 14.1.1
Standard Specifications for Composite Steel Joists, 2006

TMS
The Masonry Society
3970 Broadway
Unit 201-D
Boulder, CO 80304-1135

TMS 402
Sections 14.4.1, 14.4.2, 14.4.3, 14.4.3.1, 14.4.4, 14.4.4.1, 14.4.4.2.2, 14.4.5, 14.4.5.1, 14.4.5.2, 14.4.5.3, 14.4.5.4, 14.4.5.5, 14.4.5.6, 14.4.6, 14.4.6.1, 15.4.9.2
Building Code Requirements for Masonry Structures, 2008

TMS 602
Sections 14.4.1, 14.4.2, 14.4.7, 14.4.7.1
Specification for Masonry Structures, 2008

Chapter 24

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Chapter 25

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Chapter 26
WIND LOADS: GENERAL REQUIREMENTS

26.1 PROCEDURES

26.1.1 Scope

Buildings and other structures, including the Main Wind-Force Resisting System (MWFRS) and all components and cladding (C&C) thereof, shall be designed and constructed to resist the wind loads determined in accordance with Chapters 26 through 31. The provisions of this chapter define basic wind parameters for use with other provisions contained in this standard.

26.1.2 Permitted Procedures

The design wind loads for buildings and other structures, including the MWFRS and component and cladding elements thereof, shall be determined using one of the procedures as specified in this section. An outline of the overall process for the determination of the wind loads, including section references, is provided in Fig. 26.1-1.

26.1.2.1 Main Wind-Force Resisting System (MWFRS)

Wind loads for MWFRS shall be determined using one of the following procedures:

(1) Directional Procedure for buildings of all heights as specified in Chapter 27 for buildings meeting the requirements specified therein;
(2) Envelope Procedure for low-rise buildings as specified in Chapter 28 for buildings meeting the requirements specified therein;
(3) Directional Procedure for Building Appurtenances (rooftop structures and rooftop equipment) and Other Structures (such as solid freestanding walls and solid freestanding signs, chimneys, tanks, open signs, lattice frameworks, and trussed towers) as specified in Chapter 29;
(4) Wind Tunnel Procedure for all buildings and all other structures as specified in Chapter 31.

26.1.2.2 Components and Cladding

Wind loads on components and cladding on all buildings and other structures shall be designed using one of the following procedures:

(1) Analytical Procedures provided in Parts 1 through 6, as appropriate, of Chapter 30;
(2) Wind Tunnel Procedure as specified in Chapter 31.

26.2 DEFINITIONS

The following definitions apply to the provisions of Chapters 26 through 31:

APPROVED: Acceptable to the authority having jurisdiction.

BASIC WIND SPEED, *V*: Three-second gust speed at 33 ft (10 m) above the ground in Exposure C (see Section 26.7.3) as determined in accordance with Section 26.5.1.

BUILDING, ENCLOSED: A building that does not comply with the requirements for open or partially enclosed buildings.

BUILDING ENVELOPE: Cladding, roofing, exterior walls, glazing, door assemblies, window assemblies, skylight assemblies, and other components enclosing the building.

BUILDING AND OTHER STRUCTURE, FLEXIBLE: Slender buildings and other structures that have a fundamental natural frequency less than 1 Hz.

BUILDING, LOW-RISE: Enclosed or partially enclosed buildings that comply with the following conditions:

1. Mean roof height h less than or equal to 60 ft (18 m).
2. Mean roof height h does not exceed least horizontal dimension.

BUILDING, OPEN: A building having each wall at least 80 percent open. This condition is expressed for each wall by the equation $A_o \geq 0.8\ A_g$ where

A_o = total area of openings in a wall that receives positive external pressure, in ft^2 (m^2)
A_g = the gross area of that wall in which A_o is identified, in ft^2 (m^2)

BUILDING, PARTIALLY ENCLOSED: A building that complies with both of the following conditions:

1. The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10 percent.
2. The total area of openings in a wall that receives positive external pressure exceeds 4 ft^2 (0.37 m^2)

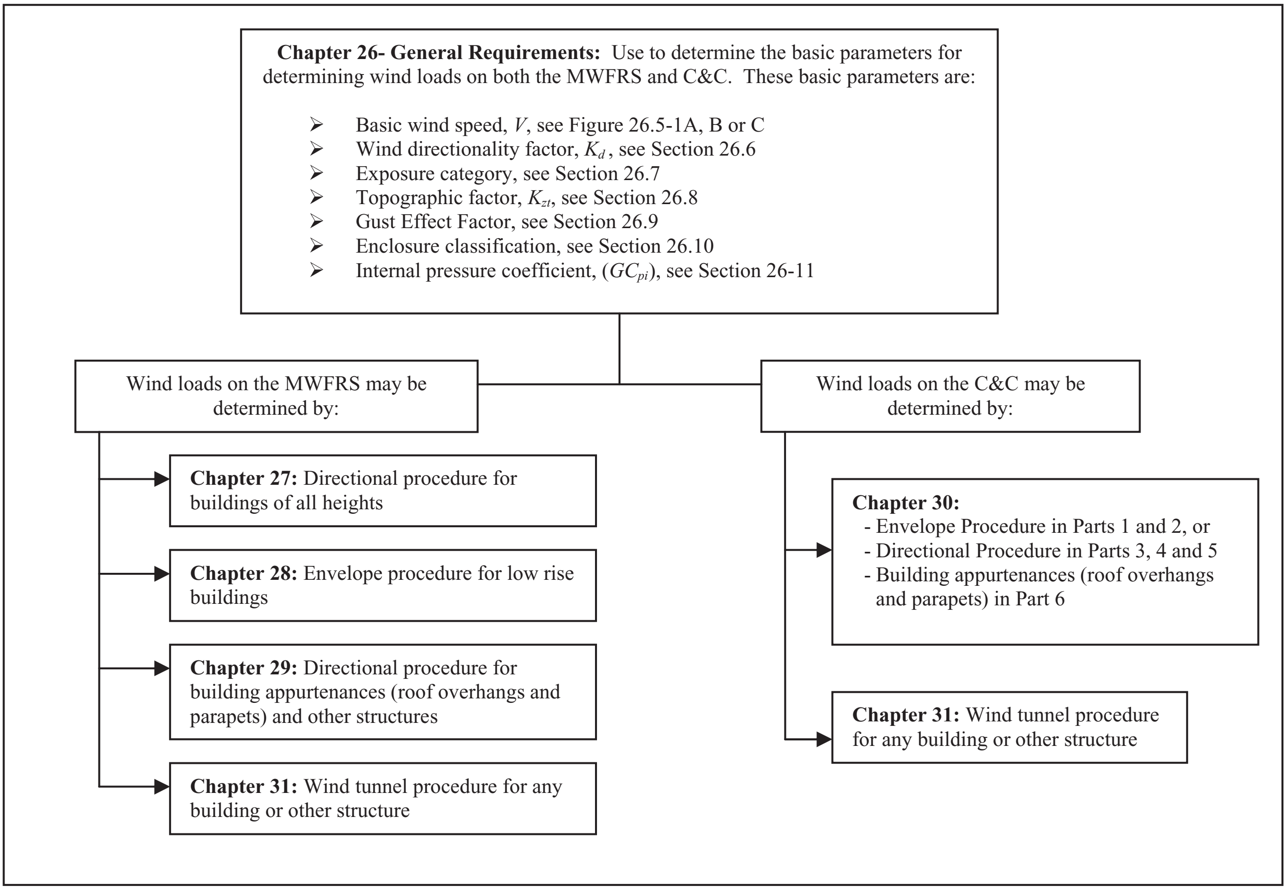

FIGURE 26.1-1 Outline of Process for Determining Wind Loads. Additional outlines and User Notes are provided at the beginning of each chapter for more detailed step-by-step procedures for determining the wind loads.

or 1 percent of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20 percent.

These conditions are expressed by the following equations:

1. $A_o > 1.10A_{oi}$
2. $A_o > 4$ ft^2 (0.37 m^2) or $> 0.01A_g$, whichever is smaller, and $A_{oi}/A_{gi} \leq 0.20$

where

A_o, A_g are as defined for Open Building

A_{oi} = the sum of the areas of openings in the building envelope (walls and roof) not including A_o, in ft^2 (m^2)

A_{gi} = the sum of the gross surface areas of the building envelope (walls and roof) not including A_g, in ft^2 (m^2)

BUILDING OR OTHER STRUCTURE, REGULAR-SHAPED: A building or other structure having no unusual geometrical irregularity in spatial form.

BUILDING OR OTHER STRUCTURES, RIGID: A building or other structure whose fundamental frequency is greater than or equal to 1 Hz.

BUILDING, SIMPLE DIAPHRAGM: A building in which both windward and leeward wind loads are transmitted by roof and vertically spanning wall assemblies, through continuous floor and roof diaphragms, to the MWFRS.

BUILDING, TORSIONALLY REGULAR UNDER WIND LOAD: A building with the MWFRS about each principal axis proportioned so that the maximum displacement at each story under Case 2, the torsional wind load case, of Fig. 27.4-8, does not exceed the maximum displacement at the same location under Case 1 of Fig. 27.4-8, the basic wind load case.

COMPONENTS AND CLADDING (C&C): Elements of the building envelope that do not qualify as part of the MWFRS.

DESIGN FORCE, *F*: Equivalent static force to be used in the determination of wind loads for other structures.

DESIGN PRESSURE, *p*: Equivalent static pressure to be used in the determination of wind loads for buildings.

DIAPHRAGM: Roof, floor, or other membrane or bracing system acting to transfer lateral forces to the vertical Main Wind-Force Resisting System. For analysis under wind loads, diaphragms constructed of untopped steel decks, concrete filled steel decks, and concrete slabs, each having a span-to-depth ratio of two or less, shall be permitted to be idealized as *rigid*. Diaphragms constructed of wood structural panels are permitted to be idealized as *flexible*.

DIRECTIONAL PROCEDURE: A procedure for determining wind loads on buildings and other structures for specific wind directions, in which the external pressure coefficients utilized are based on past wind tunnel testing of prototypical building models for the corresponding direction of wind.

EAVE HEIGHT, h_e: The distance from the ground surface adjacent to the building to the roof eave line at a particular wall. If the height of the eave varies along the wall, the average height shall be used.

EFFECTIVE WIND AREA, *A*: The area used to determine (GC_p). For component and cladding elements, the effective wind area in Figs. 30.4-1 through 30.4-7, 30.5-1. 30.6-1, and 30.8-1 through 30.8-3 is the span length multiplied by an effective width that need not be less than one-third the span length. For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener.

ENVELOPE PROCEDURE: A procedure for determining wind load cases on buildings, in which pseudo-external pressure coefficients are derived from past wind tunnel testing of prototypical building models successively rotated through 360 degrees, such that the pseudo-pressure cases produce key structural actions (uplift, horizontal shear, bending moments, etc.) that envelop their maximum values among all possible wind directions.

ESCARPMENT: Also known as scarp, with respect to topographic effects in Section 26.8, a cliff or steep slope generally separating two levels or gently sloping areas (see Fig. 26.8-1).

FREE ROOF: Roof with a configuration generally conforming to those shown in Figs. 27.4-4 through 27.4-6 (monoslope, pitched, or troughed) in an open building with no enclosing walls underneath the roof surface.

GLAZING: Glass or transparent or translucent plastic sheet used in windows, doors, skylights, or curtain walls.

GLAZING, IMPACT RESISTANT: Glazing that has been shown by testing to withstand the impact of test missiles. See Section 26.10.3.2.

HILL: With respect to topographic effects in Section 26.8, a land surface characterized by strong relief in any horizontal direction (see Fig. 26.8-1).

HURRICANE PRONE REGIONS: Areas vulnerable to hurricanes; in the United States and its territories defined as

1. The U.S. Atlantic Ocean and Gulf of Mexico coasts where the basic wind speed for Risk Category II buildings is greater than 115 mi/h, and
2. Hawaii, Puerto Rico, Guam, Virgin Islands, and American Samoa.

IMPACT PROTECTIVE SYSTEM: Construction that has been shown by testing to withstand the impact of test missiles and that is applied, attached, or locked over exterior glazing. See Section 26.10.3.2.

MAIN WIND-FORCE RESISTING SYSTEM (MWFRS): An assemblage of structural elements assigned to provide support and stability for the overall structure. The system generally receives wind loading from more than one surface.

MEAN ROOF HEIGHT, *h*: The average of the roof eave height and the height to the highest point on the roof surface, except that, for roof angles of less than or equal to 10°, the mean roof height is permitted to be taken as the roof eave height.

OPENINGS: Apertures or holes in the building envelope that allow air to flow through the building envelope and that are designed as “open” during design winds as defined by these provisions.

RECOGNIZED LITERATURE: Published research findings and technical papers that are approved.

RIDGE: With respect to topographic effects in Section 26.8 an elongated crest of a hill characterized by strong relief in two directions (see Fig. 26.8-1).

WIND TUNNEL PROCEDURE: A procedure for determining wind loads on buildings and other structures, in which pressures and/or forces and moments are determined for each wind direction considered, from a model of the building or other structure and its surroundings, in accordance with Chapter 31.

WIND-BORNE DEBRIS REGIONS: Areas within hurricane prone regions where impact protection is required for glazed openings, see Section 26.10.3.

26.3 SYMBOLS AND NOTATION

The following symbols and notation apply only to the provisions of Chapters 26 through 31:

A = effective wind area, in ft^2 (m^2)
A_f = area of open buildings and other structures either normal to the wind direction or projected on a plane normal to the wind direction, in ft^2 (m^2)
A_g = the gross area of that wall in which A_o is identified, in ft^2 (m^2)
A_{gi} = the sum of the gross surface areas of the building envelope (walls and roof) not including A_g, in ft^2 (m^2)
A_o = total area of openings in a wall that receives positive external pressure, in ft^2 (m^2)
A_{oi} = the sum of the areas of openings in the building envelope (walls and roof) not including A_o, in ft^2 (m^2)
A_{og} = total area of openings in the building envelope in ft^2 (m^2)
A_s = gross area of the solid freestanding wall or solid sign, in ft^2 (m^2)
a = width of pressure coefficient zone, in ft (m)
B = horizontal dimension of building measured normal to wind direction, in ft (m)
$\bar{b}$ = mean hourly wind speed factor in Eq. 26.9-16 from Table 26.9-1
$\hat{b}$ = 3-s gust speed factor from Table 26.9-1
C_f = force coefficient to be used in determination of wind loads for other structures
C_N = net pressure coefficient to be used in determination of wind loads for open buildings
C_p = external pressure coefficient to be used in determination of wind loads for buildings
c = turbulence intensity factor in Eq. 26.9-7 from Table 26.9-1
D = diameter of a circular structure or member, in ft (m)
D' = depth of protruding elements such as ribs and spoilers, in ft (m)
F = design wind force for other structures, in lb (N)
G = gust-effect factor
G_f = gust-effect factor for MWFRS of flexible buildings and other structures
(GC_{pn}) = combined net pressure coefficient for a parapet
(GC_p) = product of external pressure coefficient and gust-effect factor to be used in determination of wind loads for buildings
(GC_{pf}) = product of the equivalent external pressure coefficient and gust-effect factor to be used in determination of wind loads for MWFRS of low-rise buildings
(GC_{pi}) = product of internal pressure coefficient and gust-effect factor to be used in determination of wind loads for buildings
(GC_r) = product of external pressure coefficient and gust-effect factor to be used in determination of wind loads for rooftop structures
g_Q = peak factor for background response in Eqs. 26.9-6 and 26.9-10
g_R = peak factor for resonant response in Eq. 26.9-10
g_v = peak factor for wind response in Eqs. 26.9-6 and 26.9-10
H = height of hill or escarpment in Fig. 26.8-1, in ft (m)
h = mean roof height of a building or height of other structure, except that eave height shall be used for roof angle θ less than or equal to 10°, in ft (m)
h_e = roof eave height at a particular wall, or the average height if the eave varies along the wall
h_p = height to top of parapet in Fig. 27.6-4 and 30.7-1
$I_{\bar{z}}$ = intensity of turbulence from Eq. 26.9-7
K_1, K_2, K_3 = multipliers in Fig. 26.8-1 to obtain K_{zt}
K_d = wind directionality factor in Table 26.6-1
K_h = velocity pressure exposure coefficient evaluated at height $z = h$
K_z = velocity pressure exposure coefficient evaluated at height z
K_{zt} = topographic factor as defined in Section 26.8
L = horizontal dimension of a building measured parallel to the wind direction, in ft (m)
L_h = distance upwind of crest of hill or escarpment in Fig. 26.8-1 to where the difference in ground elevation is half the height of the hill or escarpment, in ft (m)

L_z = integral length scale of turbulence, in ft (m)
L_r = horizontal dimension of return corner for a solid freestanding wall or solid sign from Fig. 29.4-1, in ft (m)
ℓ = integral length scale factor from Table 26.9-1, ft (m)
N_1 = reduced frequency from Eq. 26.9-14
n_a = approximate lower bound natural frequency (Hz) from Section 26.9.2
n_1 = fundamental natural frequency, Hz
p = design pressure to be used in determination of wind loads for buildings, in lb/ft^2 (N/m^2)
P_L = wind pressure acting on leeward face in Fig. 27.4-8, in lb/ft^2 (N/m^2)
p_{net} = net design wind pressure from Eq. 30.5-1, in lb/ft^2 (N/m^2)
p_{net30} = net design wind pressure for Exposure B at h = 30 ft and I = 1.0 from Fig. 30.5-1, in lb/ft^2 (N/m^2)
p_p = combined net pressure on a parapet from Eq. 27.4-5, in lb/ft^2 (N/m^2)
p_s = net design wind pressure from Eq. 28.6-1, in lb/ft^2 (N/m^2)
p_{s30} = simplified design wind pressure for Exposure B at h = 30 ft and I = 1.0 from Fig. 28.6-1, in lb/ft^2 (N/m^2)
P_W = wind pressure acting on windward face in Fig. 27.4-8, in lb/ft^2 (N/m^2)
Q = background response factor from Eq. 26.9-8
q = velocity pressure, in lb/ft^2 (N/m^2)
q_h = velocity pressure evaluated at height $z = h$, in lb/ft^2 (N/m^2)
q_i = velocity pressure for internal pressure determination, in lb/ft^2 (N/m^2)
q_p = velocity pressure at top of parapet, in lb/ft^2 (N/m^2)
q_z = velocity pressure evaluated at height z above ground, in lb/ft^2 (N/m^2)
R = resonant response factor from Eq. 26.9-12
R_B, R_h, R_L = values from Eqs. 26.9-15
R_i = reduction factor from Eq. 26.11-1
R_n = value from Eq. 26.9-13
s = vertical dimension of the solid freestanding wall or solid sign from Fig. 29.4-1, in ft (m)
r = rise-to-span ratio for arched roofs
V = basic wind speed obtained from Fig. 26.5-1A through 26.5-1C, in mi/h (m/s). The basic wind speed corresponds to a 3-sec gust speed at 33 ft (10 m) above the ground in Exposure Category C
V_i = unpartitioned internal volume, ft^3 (m^3)
$\overline{V}_{\overline{z}}$ = mean hourly wind speed at height $\overline{z}$, ft/s (m/s)
W = width of building in Figs. 30.4-3 and 30.4-5A and 30.4-5B and width of span in Figs. 30.4-4 and 30.4-6, in ft (m)
x = distance upwind or downwind of crest in Fig. 26.8-1, in ft (m)
z = height above ground level, in ft (m)
$\overline{z}$ = equivalent height of structure, in ft (m)
z_g = nominal height of the atmospheric boundary layer used in this standard. Values appear in Table 26.9-1
z_{min} = exposure constant from Table 26.9-1
α = 3-sec gust-speed power law exponent from Table 26.9-1
$\hat{\alpha}$ = reciprocal of α from Table 26.9-1
$\overline{\alpha}$ = mean hourly wind-speed power law exponent in Eq. 26.9-16 from Table 26.9-1
β = damping ratio, percent critical for buildings or other structures
$\in$ = ratio of solid area to gross area for solid freestanding wall, solid sign, open sign, face of a trussed tower, or lattice structure
λ = adjustment factor for building height and exposure from Figs. 28.6-1 and 30.5-1
$\overline{\in}$ = integral length scale power law exponent in Eq. 26.9-9 from Table 26.9-1
η = value used in Eq. 26.9-15 (see Section 26.9.4)
θ = angle of plane of roof from horizontal, in degrees
v = height-to-width ratio for solid sign

26.4 GENERAL

26.4.1 Sign Convention

Positive pressure acts toward the surface and negative pressure acts away from the surface.

26.4.2 Critical Load Condition

Values of external and internal pressures shall be combined algebraically to determine the most critical load.

26.4.3 Wind Pressures Acting on Opposite Faces of Each Building Surface

In the calculation of design wind loads for the MWFRS and for components and cladding for buildings, the algebraic sum of the pressures acting on opposite faces of each building surface shall be taken into account.

26.5 WIND HAZARD MAP

26.5.1 Basic Wind Speed

The basic wind speed, V, used in the determination of design wind loads on buildings and other structures shall be determined from Fig. 26.5-1 as follows, except as provided in Section 26.5.2 and 26.5.3:

For Risk Category II buildings and structures – use Fig. 26.5-1A.
For Risk Category III and IV buildings and structures – use Fig. 26.5-1B.
For Risk Category I buildings and structures - use Fig. 26.5-1C.

The wind shall be assumed to come from any horizontal direction. The basic wind speed shall be increased where records or experience indicate that the wind speeds are higher than those reflected in Fig. 26.5-1.

26.5.2 Special Wind Regions

Mountainous terrain, gorges, and special wind regions shown in Fig. 26.5-1 shall be examined for unusual wind conditions. The authority having jurisdiction shall, if necessary, adjust the values given in Fig. 26.5-1 to account for higher local wind speeds. Such adjustment shall be based on meteorological information and an estimate of the basic wind speed obtained in accordance with the provisions of Section 26.5.3.

26.5.3 Estimation of Basic Wind Speeds from Regional Climatic Data

In areas outside hurricane-prone regions, regional climatic data shall only be used in lieu of the basic wind speeds given in Fig. 26.5-1 when (1) approved extreme-value statistical-analysis procedures have been employed in reducing the data; and (2) the length of record, sampling error, averaging time, anemometer height, data quality, and terrain exposure of the anemometer have been taken into account. Reduction in basic wind speed below that of Fig. 26.5-1 shall be permitted.

In hurricane-prone regions, wind speeds derived from simulation techniques shall only be used in lieu of the basic wind speeds given in Fig. 26.5-1 when approved simulation and extreme value statistical analysis procedures are used. The use of regional wind speed data obtained from anemometers is not permitted to define the hurricane wind-speed risk along the Gulf and Atlantic coasts, the Caribbean, or Hawaii.

In areas outside hurricane-prone regions, when the basic wind speed is estimated from regional climatic data, the basic wind speed shall not be less than the wind speed associated with the specified mean recurrence interval, and the estimate shall be adjusted for equivalence to a 3-sec gust wind speed at 33 ft (10 m) above ground in Exposure C. The data analysis shall be performed in accordance with this chapter.

26.5.4 Limitation

Tornadoes have not been considered in developing the basic wind-speed distributions.

26.6 WIND DIRECTIONALITY

The wind directionality factor, K_d, shall be determined from Table 26.6-1. This directionality factor shall only be included in determining wind loads when the load combinations specified in Sections 2.3 and 2.4 are used for the design. The effect of wind directionality in determining wind loads in accordance with Chapter 31 shall be based on an analysis for wind speeds that conforms to the requirements of Section 26.5.3.

26.7 EXPOSURE

For each wind direction considered, the upwind exposure shall be based on ground surface roughness that is determined from natural topography, vegetation, and constructed facilities.

26.7.1 Wind Directions and Sectors

For each selected wind direction at which the wind loads are to be determined, the exposure of the building or structure shall be determined for the two upwind sectors extending 45° either side of the selected wind direction. The exposure in these two sectors shall be determined in accordance with Sections 26.7.2 and 26.7.3, and the exposure whose use would result in the highest wind loads shall be used to represent the winds from that direction.

26.7.2 Surface Roughness Categories

A ground Surface Roughness within each 45° sector shall be determined for a distance upwind of the site as defined in Section 26.7.3 from the categories defined in the following text, for the purpose of assigning an exposure category as defined in Section 26.7.3.

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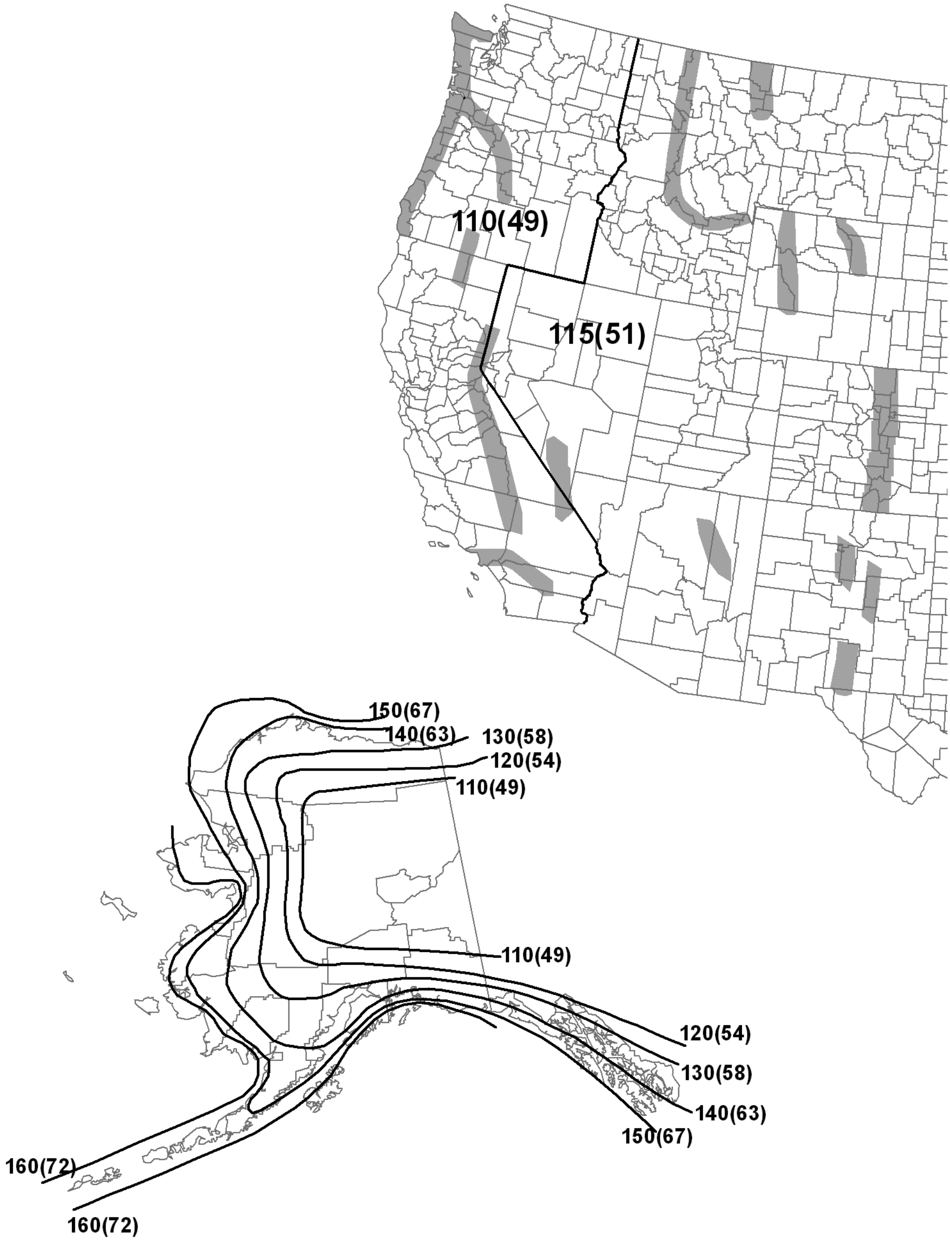
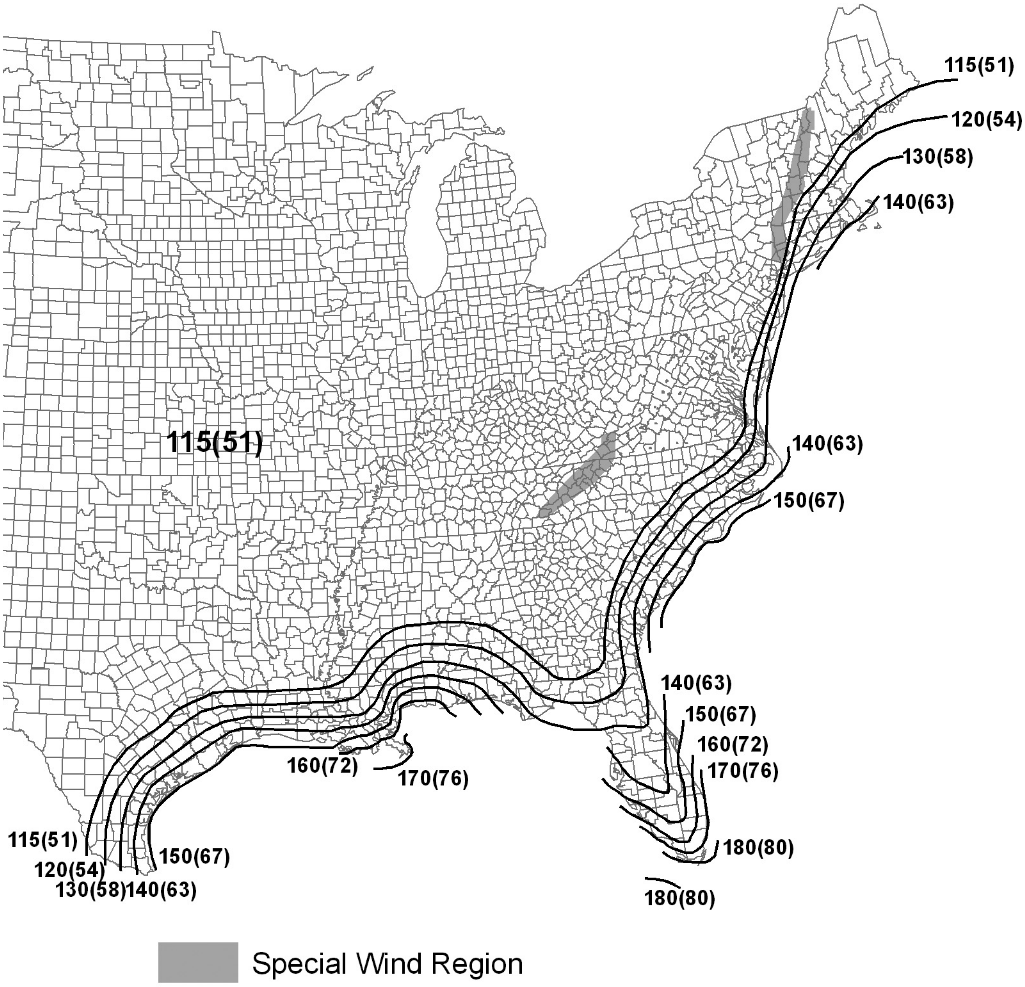


Figure 26.5-1A Basic Wind Speeds for Occupancy Category II Buildings and Other Structures.

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 Years).



Location	Vmph	(m/s)
Guam	195	(87)
Virgin Islands	165	(74)
American Samoa	160	(72)
Hawaii – Special Wind Region Statewide	130	(58)

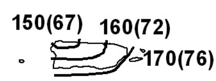


Figure 26.5-1A (*Continued*)

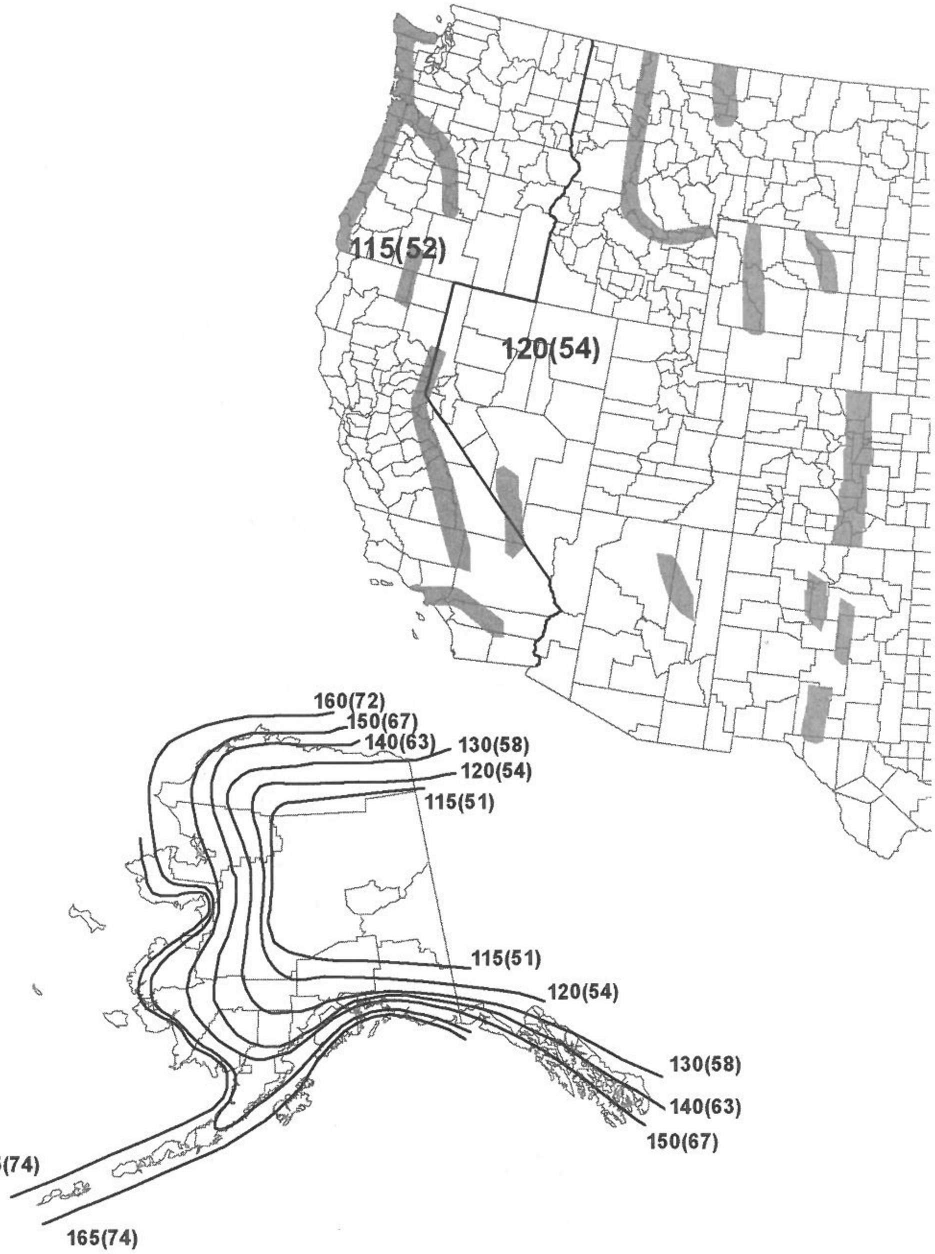
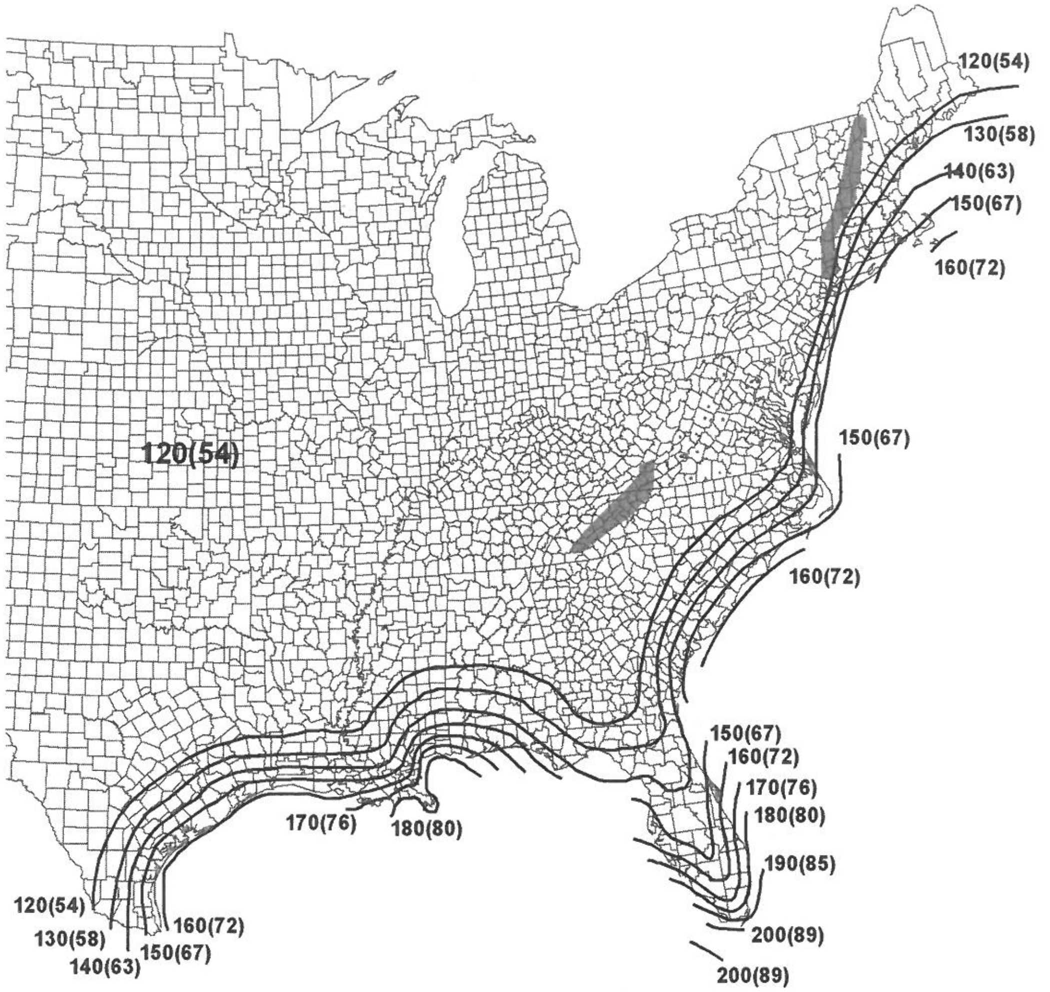


Figure 26.5-1B Basic Wind Speeds for Risk Category III and IV Buildings and Other Structures.

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (Annual Exceedance Probability = 0.000588, MRI = 1700 Years).



Special Wind Region

Location	Vmph	(m/s)
Guam	210	(94)
Virgin Islands	175	(78)
American Samoa	170	(76)
Hawaii – Special Wind Region Statewide	145	(65)

Puerto Rico

Figure 26.5-1B (*Continued*)

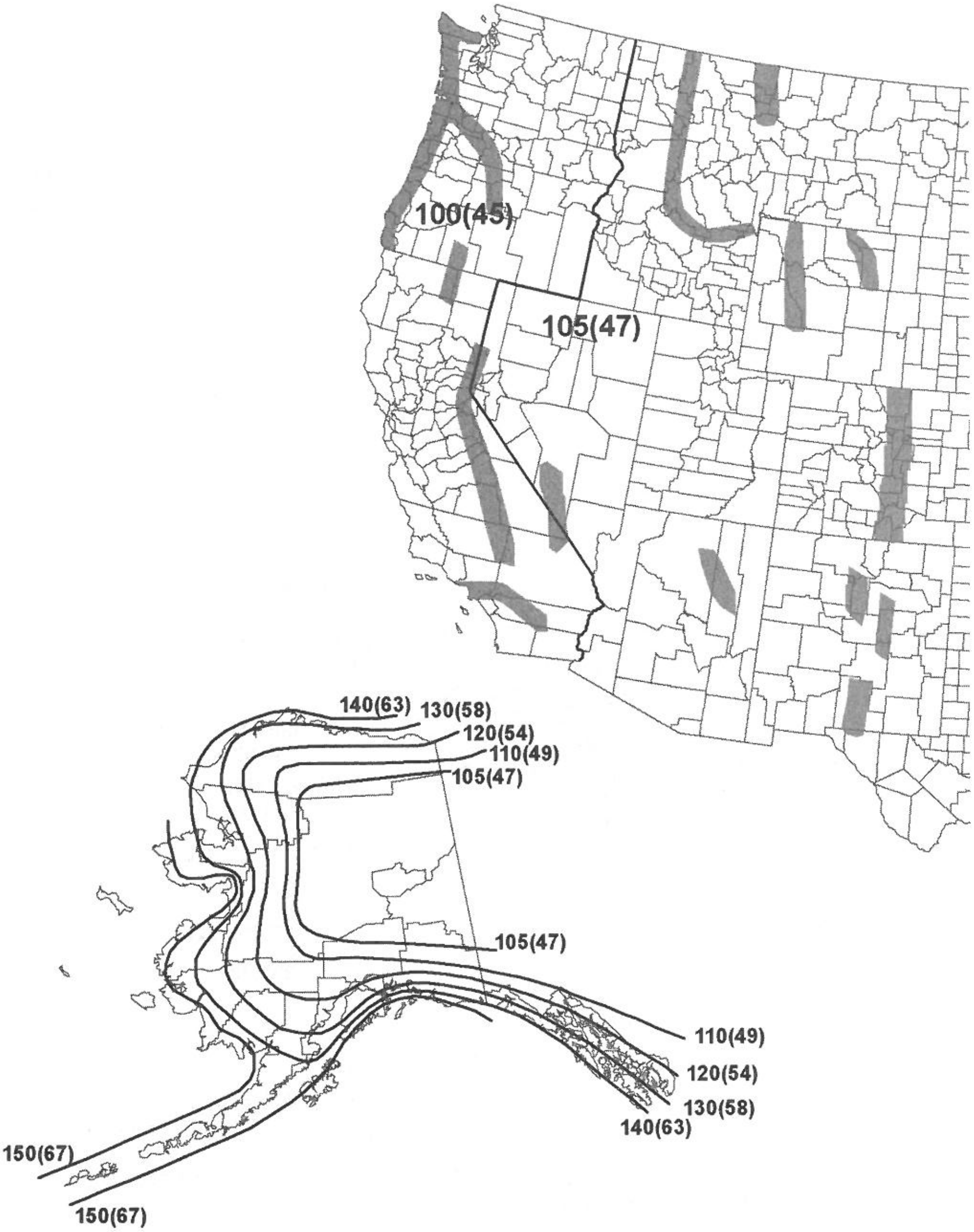
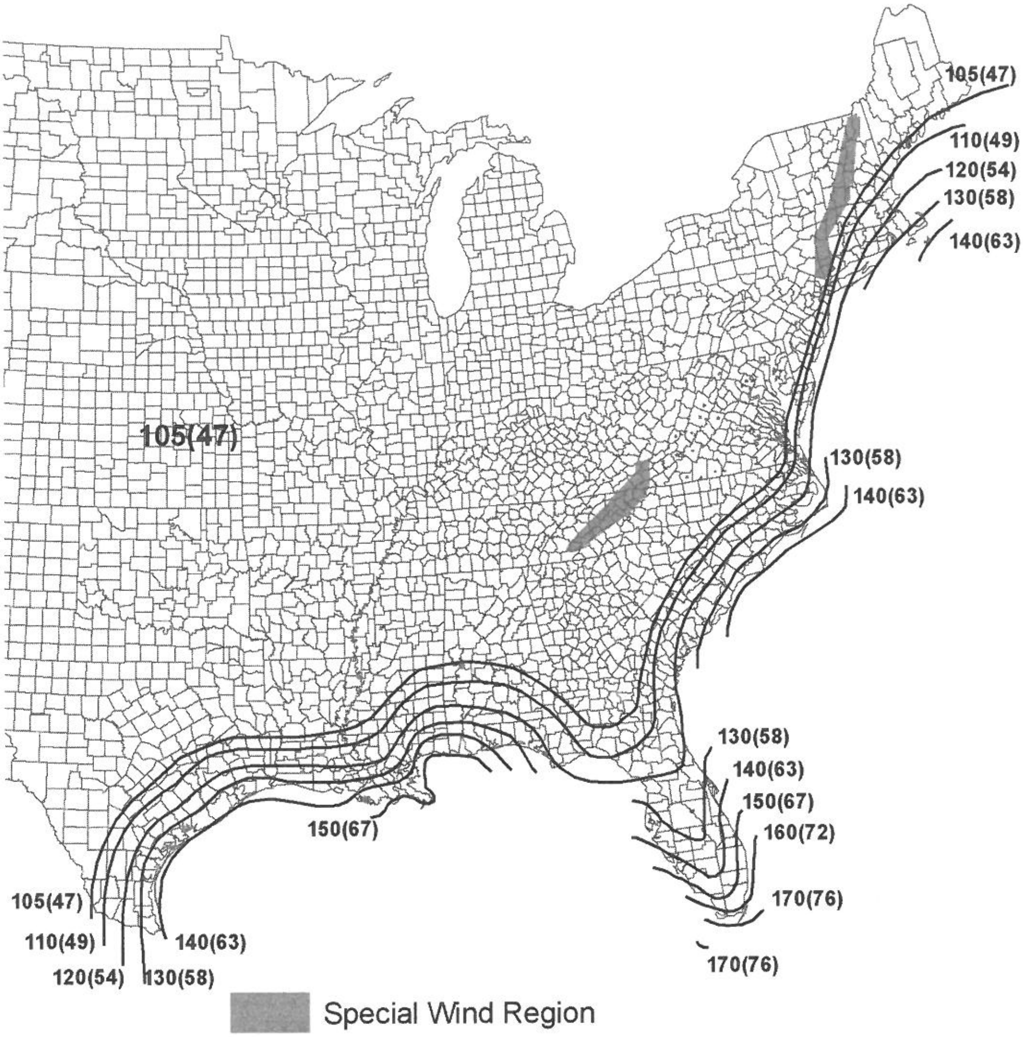


Figure 26.5-1C Basic Wind Speeds for Risk Category I Buildings and Other Structures.

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 15% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00333, MRI = 300 Years).



Location	Vmph	(m/s)
Guam	180	(80)
Virgin Islands	150	(67)
American Samoa	150	(67)
Hawaii – Special Wind Region Statewide	115	(51)

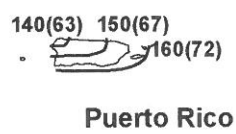


Figure 26.5-1c (*Continued*)

Wind Directionality Factor, K_d

Table 26.6-1

Structure Type	Directionality Factor K_d*
Buildings	
Main Wind Force Resisting System	0.85
Components and Cladding	0.85
Arched Roofs	0.85
Chimneys, Tanks, and Similar Structures	
Square	0.90
Hexagonal	0.95
Round	0.95
Solid Freestanding Walls and Solid Freestanding and Attached Signs	0.85
Open Signs and Lattice Framework	0.85
Trussed Towers	
Triangular, square, rectangular	0.85
All other cross sections	0.95

*Directionality Factor K_d has been calibrated with combinations of loads specified in Chapter 2. This factor shall only be applied when used in conjunction with load combinations specified in Sections 2.3 and 2.4.

Surface Roughness B: Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness C: Open terrain with scattered obstructions having heights generally less than 30 ft (9.1 m). This category includes flat open country and grasslands.

Surface Roughness D: Flat, unobstructed areas and water surfaces. This category includes smooth mud flats, salt flats, and unbroken ice.

26.7.3 Exposure Categories

Exposure B: For buildings with a mean roof height of less than or equal to 30 ft (9.1 m), Exposure B shall apply where the ground surface roughness, as defined by Surface Roughness B, prevails in the upwind direction for a distance greater than 1,500 ft (457 m). For buildings with a mean roof height greater than 30 ft (9.1 m), Exposure B shall apply where Surface Roughness B prevails in the upwind direction for a distance greater than 2,600 ft (792 m) or 20 times the height of the building, whichever is greater.

Exposure C: Exposure C shall apply for all cases where Exposures B or D do not apply.

Exposure D: Exposure D shall apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance greater than 5,000 ft (1,524 m) or 20 times the building height, whichever is greater. Exposure D shall also apply where the ground surface roughness immediately upwind of the site is B or C, and the site is within a distance of 600 ft (183 m) or 20 times the building height, whichever is greater, from an Exposure D condition as defined in the previous sentence.

For a site located in the transition zone between exposure categories, the category resulting in the largest wind forces shall be used.

EXCEPTION: An intermediate exposure between the preceding categories is permitted in a transition zone provided that it is determined by a rational analysis method defined in the recognized literature.

26.7.4 Exposure Requirements.

26.7.4.1 Directional Procedure (Chapter 27)

For each wind direction considered, wind loads for the design of the MWFRS of enclosed and partially enclosed buildings using the Directional Procedure of Chapter 27 shall be based on the exposures as defined in Section 26.7.3. Wind loads for the design of open buildings with monoslope, pitched, or troughed free roofs shall be based on the exposures, as defined in Section 26.7.3, resulting in the highest wind loads for any wind direction at the site.

26.7.4.2 Envelope Procedure (Chapter 28)

Wind loads for the design of the MWFRS for all low-rise buildings designed using the Envelope Procedure of Chapter 28 shall be based on the exposure category resulting in the highest wind loads for any wind direction at the site.

26.7.4.3 Directional Procedure for Building Appurtenances and Other Structures (Chapter 29)

Wind loads for the design of building appurtenances (such as rooftop structures and equipment) and other structures (such as solid freestanding walls and freestanding signs, chimneys, tanks, open signs, lattice frameworks, and trussed towers) as specified in Chapter 29 shall be based on the appropriate exposure for each wind direction considered.

26.7.4.4 Components and Cladding (Chapter 30)

Design wind pressures for components and cladding shall be based on the exposure category resulting in the highest wind loads for any wind direction at the site.

26.8 TOPOGRAPHIC EFFECTS

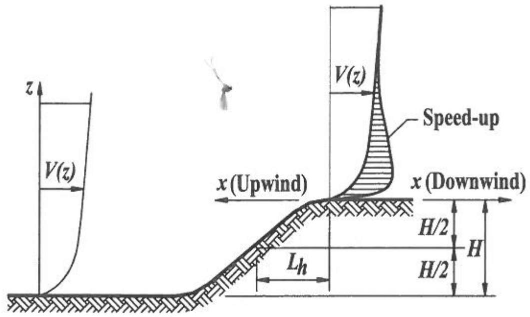
26.8.1 Wind Speed-Up over Hills, Ridges, and Escarpments

Wind speed-up effects at isolated hills, ridges, and escarpments constituting abrupt changes in the general topography, located in any exposure category, shall be included in the determination of the wind loads when buildings and other site conditions and locations of structures meet all of the following conditions:

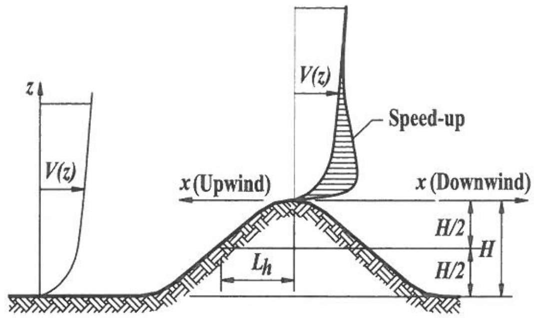
1. The hill, ridge, or escarpment is isolated and unobstructed upwind by other similar topographic features of comparable height for 100 times the height of the topographic feature (100*H*) or 2 mi (3.22 km), whichever is less. This distance shall be measured horizontally from the point at which the height *H* of the hill, ridge, or escarpment is determined.
2. The hill, ridge, or escarpment protrudes above the height of upwind terrain features within a 2-mi (3.22-km) radius in any quadrant by a factor of two or more.
3. The structure is located as shown in Fig. 26.8-1 in the upper one-half of a hill or ridge or near the crest of an escarpment.

Topographic Factor, K_{zt}

Figure 26.8-1



ESCARPMENT



2-D RIDGE OR 3-D AXISYMMETRICAL HILL

Topographic Multipliers for Exposure C										
H/L_h	K_1 Multiplier			x/L_h	K_2 Multiplier		z/L_h	K_3 Multiplier		
	2-D Ridge	2-D Escarp.	3-D Axisym. Hill		2-D Escarp.	All Other Cases		2-D Ridge	2-D Escarp.	3-D Axisym. Hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Notes:

1. For values of H/L_h, x/L_h and z/L_h other than those shown, linear interpolation is permitted.
2. For $H/L_h > 0.5$, assume $H/L_h = 0.5$ for evaluating K_1 and substitute 2H for L_h for evaluating K_2 and K_3.
3. Multipliers are based on the assumption that wind approaches the hill or escarpment along the direction of maximum slope.
4. Notation:
   - H: Height of hill or escarpment relative to the upwind terrain, in feet (meters).
   - L_h: Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in feet (meters).
   - K_1: Factor to account for shape of topographic feature and maximum speed-up effect.
   - K_2: Factor to account for reduction in speed-up with distance upwind or downwind of crest.
   - K_3: Factor to account for reduction in speed-up with height above local terrain.
   - x: Distance (upwind or downwind) from the crest to the building site, in feet (meters).
   - z: Height above ground surface at building site, in feet (meters).
   - μ: Horizontal attenuation factor.
   - γ: Height attenuation factor.

Topographic Factor, K_{zt}

Figure 26.8-1 (cont'd)

Equations:

$$K_{zt} = (1 + K_1 K_2 K_3)^2$$

K_1 determined from table below

$$K_2 = \left(1 - \frac{|x|}{\mu L_h}\right)$$

$$K_3 = e^{-\gamma z / L_h}$$

Parameters for Speed-Up Over Hills and Escarpments						
Hill Shape	**$K_1/(H/L_h)$**			**γ**	**μ**	
	Exposure				**Upwind of Crest**	**Downwind of Crest**
	B	**C**	**D**			
2-dimensional ridges (or valleys with negative H in $K_1/(H/L_h)$)	1.30	1.45	1.55	3	1.5	1.5
2-dimensional escarpments	0.75	0.85	0.95	2.5	1.5	4
3-dimensional axisym. hill	0.95	1.05	1.15	4	1.5	1.5

4. $H/L_h \geq 0.2$.
5. H is greater than or equal to 15 ft (4.5 m) for Exposure C and D and 60 ft (18 m) for Exposure B.

26.8.2 Topographic Factor

The wind speed-up effect shall be included in the calculation of design wind loads by using the factor K_{zt}:

$$K_{zt} = (1 + K_1K_2K_3)^2 \qquad (26.8\text{-}1)$$

where K_1, K_2, and K_3 are given in Fig. 26.8-1.

If site conditions and locations of buildings and other structures do not meet all the conditions specified in Section 26.8.1 then $K_{zt} = 1.0$.

26.9 GUST-EFFECTS

26.9.1 Gust-Effect Factor: The gust-effect factor for a rigid building or other structure is permitted to be taken as 0.85.

26.9.2 Frequency Determination

To determine whether a building or other structure is rigid or flexible as defined in Section 26.2, the fundamental natural frequency, n_1, shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. Low-Rise Buildings, as defined in 26.2, are permitted to be considered rigid.

26.9.2.1 Limitations for Approximate Natural Frequency

As an alternative to performing an analysis to determine n_1, the approximate building natural frequency, n_a, shall be permitted to be calculated in accordance with Section 26.9.3 for structural steel, concrete, or masonry buildings meeting the following requirements:

1. The building height is less than or equal to 300 ft (91 m), and
2. The building height is less than 4 times its effective length, L_{eff}.

The effective length, L_{eff}, in the direction under consideration shall be determined from the following equation:

$$L_{eff} = \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} \qquad (26.9\text{-}1)$$

The summations are over the height of the building where

h_i is the height above grade of level i
L_i is the building length at level i parallel to the wind direction

26.9.3 Approximate Natural Frequency

The approximate lower-bound natural frequency (n_a), in Hertz, of concrete or structural steel buildings meeting the conditions of Section 26.9.2.1, is permitted to be determined from one of the following equations:

For structural steel moment-resisting-frame buildings:

$$n_a = 22.2/h^{0.8} \qquad (26.9\text{-}2)$$

For concrete moment-resisting frame buildings:

$$n_a = 43.5/h^{0.9} \qquad (26.9\text{-}3)$$

For structural steel and concrete buildings with other lateral-force-resisting systems:

$$n_a = 75/h \qquad (26.9\text{-}4)$$

For concrete or masonry shear wall buildings, it is also permitted to use

$$n_a = 385(C_w)^{0.5}/h \qquad (26.9\text{-}5)$$

where

$$C_w = \frac{100}{A_B}\sum_{i=1}^{n}\left(\frac{h}{h_i}\right)^2 \frac{A_i}{\left[1+0.83\left(\frac{h_i}{D_i}\right)^2\right]}$$

where

h = mean roof height (ft)
n = number of shear walls in the building effective in resisting lateral forces in the direction under consideration
A_B = base area of the structure (ft^2)
A_i = horizontal cross-section area of shear wall "i" (ft^2)
D_i = length of shear wall "i" (ft)
h_i = height of shear wall "i" (ft)

26.9.4 Rigid Buildings or Other Structures

For rigid buildings or other structures as defined in Section 26.2, the gust-effect factor shall be taken as 0.85 or calculated by the formula:

$$G = 0.925\left(\frac{1+1.7g_Q I_{\bar{z}} Q}{1+1.7g_v I_{\bar{z}}}\right) \qquad (26.9\text{-}6)$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} \qquad (26.9\text{-}7)$$

In SI: $I_{\bar{z}} = c\left(\frac{10}{\bar{z}}\right)^{1/6}$

where $I_{\bar{z}}$ is the intensity of turbulence at height $\bar{z}$ where $\bar{z}$ is the equivalent height of the structure defined as $0.6h$, but not less than z_{min} for all building heights h. z_{min} and c are listed for each exposure in Table 26.9-1; g_Q and g_v shall be taken as 3.4. The background response Q is given by

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \qquad (26.9\text{-}8)$$

where B and h are defined in Section 26.3 and $L_{\bar{z}}$ is the integral length scale of turbulence at the equivalent height given by

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} \qquad (26.9\text{-}9)$$

In SI: $L_{\bar{z}} = \ell\left(\frac{\bar{z}}{10}\right)^{\bar{\epsilon}}$

in which ℓ and $\bar{\epsilon}$ are constants listed in Table 26.9-1.

26.9.5 Flexible or Dynamically Sensitive Buildings or Other Structures

For flexible or dynamically sensitive buildings or other structures as defined in Section 26.2, the gust-effect factor shall be calculated by

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right) \qquad (26.9\text{-}10)$$

g_Q and g_v shall be taken as 3.4 and g_R is given by

$$g_R = \sqrt{2\ln(3{,}600 n_1)} + \frac{0.577}{\sqrt{2\ln(3{,}600 n_1)}} \qquad (26.9\text{-}11)$$

R, the resonant response factor, is given by

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} \qquad (26.9\text{-}12)$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} \qquad (26.9\text{-}13)$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} \qquad (26.9\text{-}14)$$

$$R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) \quad \text{for} \quad \eta > 0 \qquad (26.9\text{-}15a)$$

$$R_\ell = 1 \quad \text{for} \quad \eta = 0 \qquad (26.9\text{-}15b)$$

where the subscript ℓ in Eqs. 26.9-15 shall be taken as h, B, and L, respectively, where h, B, and L are defined in Section 26.3.

n_1 = fundamental natural frequency
$R_\ell = R_h$ setting $\eta = 4.6 n_1 h/\bar{V}_{\bar{z}}$
$R_\ell = R_B$ setting $\eta = 4.6 n_1 B/\bar{V}_{\bar{z}}$
$R_\ell = R_L$ setting $\eta = 15.4 n_1 L/\bar{V}_{\bar{z}}$
β = damping ratio, percent of critical (i.e. for 2% use 0.02 in the equation)
$\bar{V}_{\bar{z}}$ = mean hourly wind speed (ft/s) at height $\bar{z}$ determined from Eq. 26.9-16:

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V \qquad (26.9\text{-}16)$$

In SI: $\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{10}\right)^{\bar{\alpha}} V$

where $\bar{b}$ and $\bar{\alpha}$ are constants listed in Table 26.9-1 and V is the basic wind speed in mi/h.

26.9.6 Rational Analysis

In lieu of the procedure defined in Sections 26.9.3 and 26.9.4, determination of the gust-effect factor by any rational analysis defined in the recognized literature is permitted.

26.9.7 Limitations

Where combined gust-effect factors and pressure coefficients (GC_p), (GC_{pi}), and (GC_{pf}) are given in figures and tables, the gust-effect factor shall not be determined separately.

26.10 ENCLOSURE CLASSIFICATION

26.10.1 General

For the purpose of determining internal pressure coefficients, all buildings shall be classified as enclosed, partially enclosed, or open as defined in Section 26.2.

26.10.2 Openings

A determination shall be made of the amount of openings in the building envelope for use in determining the enclosure classification.

26.10.3 Protection of Glazed Openings

Glazed openings in Risk Category II, III or IV buildings located in hurricane-prone regions shall be protected as specified in this Section.

26.10.3.1 Wind-borne Debris Regions

Glazed openings shall be protected in accordance with Section 26.10.3.2 in the following locations:

Terrain Exposure Constants

Table 26.9-1

Exposure	α	z_g (ft)	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ (ft)	$\bar{\epsilon}$	z_{min} (ft)*
B	7.0	1200	1/7	0.84	1/4.0	0.45	0.30	320	1/3.0	30
C	9.5	900	1/9.5	1.00	1/6.5	0.65	0.20	500	1/5.0	15
D	11.5	700	1/11.5	1.07	1/9.0	0.80	0.15	650	1/8.0	7

*z_{min} = minimum height used to ensure that the equivalent height $\bar{z}$ is greater of $0.6h$ or z_{min}.
For buildings with $h \leq z_{min}$, $\bar{z}$ shall be taken as z_{min}.

In metric

Exposure	α	z_g (m)	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ (m)	$\bar{\epsilon}$	z_{min} (m)*
B	7.0	365.76	1/7	0.84	1/4.0	0.45	0.30	97.54	1/3.0	9.14
C	9.5	274.32	1/9.5	1.00	1/6.5	0.65	0.20	152.4	1/5.0	4.57
D	11.5	213.36	1/11.5	1.07	1/9.0	0.80	0.15	198.12	1/8.0	2.13

*z_{min} = minimum height used to ensure that the equivalent height $\bar{z}$ is greater of $0.6h$ or z_{min}.
For buildings with $h \leq z_{min}$, $\bar{z}$ shall be taken as z_{min}.

1. Within 1 mi of the coastal mean high water line where the basic wind speed is equal to or greater than 130 mi/h (58 m/s), or
2. In areas where the basic wind speed is equal to or greater than 140 mi/h (63 m/s).

For Risk Category II buildings and other structures and Risk Category III buildings and other structures, except health care facilities, the wind-borne debris region shall be based on Fig. 26.5-1A. For Risk Category III health care facilities and Risk Category IV buildings and other structures, the wind-borne debris region shall be based on Fig. 26.5-1B. Risk Categories shall be determined in accordance with Section 1.5.

EXCEPTION: Glazing located over 60 ft (18.3 m) above the ground and over 30 ft (9.2 m) above aggregate-surfaced-roofs, including roofs with gravel or stone ballast, located within 1,500 ft (458 m) of the building shall be permitted to be unprotected.

26.10.3.2 Protection Requirements for Glazed Openings

Glazing in buildings requiring protection shall be protected with an impact-protective system or shall be impact-resistant glazing.

Impact-protective systems and impact-resistant glazing shall be subjected to missile test and cyclic pressure differential tests in accordance with ASTM E1996 as applicable. Testing to demonstrate compliance with ASTM E1996 shall be in accordance with ASTM E1886. Impact-resistant glazing and impact-protective systems shall comply with the pass/fail criteria of Section 7 of ASTM E1996 based on the missile required by Table 3 or Table 4 of ASTM E1996.

EXCEPTION: Other testing methods and/or performance criteria are permitted to be used when approved.

Glazing and impact-protective systems in buildings and other structures classified as Risk Category IV in accordance with Section 1.5 shall comply with the "enhanced protection" requirements of Table 3 of ASTM E1996. Glazing and impact-protective systems in all other structures shall comply with the "basic protection" requirements of Table 3 of ASTM E1996.

> **User Note:** The wind zones that are specified in ASTM E1996 for use in determining the applicable missile size for the impact test, have to be adjusted for use with the wind speed maps of ASCE 7-10 and the corresponding wind borne debris regions, see Section C26.10.3.2.

26.10.4 Multiple Classifications

If a building by definition complies with both the "open" and "partially enclosed" definitions, it shall be classified as an "open" building. A building that does not comply with either the "open" or "partially enclosed" definitions shall be classified as an "enclosed" building.

26.11 INTERNAL PRESSURE COEFFICIENT

26.11.1 Internal Pressure Coefficients

Internal pressure coefficients, (GC_{pi}), shall be determined from Table 26.11-1 based on building enclosure classifications determined from Section 26.10.

26.11.1.1 Reduction Factor for Large Volume Buildings, R_i

For a partially enclosed building containing a single, unpartitioned large volume, the internal pressure coefficient, (GC_{pi}), shall be multiplied by the following reduction factor, R_i:

$R_i = 1.0$ or

$$R_i = 0.5\left(1+\frac{1}{\sqrt{1+\dfrac{V_i}{22.800A_{og}}}}\right) < 1.0 \quad (26.11\text{-}1)$$

where

A_{og} = total area of openings in the building envelope (walls and roof, in ft^2)
V_i = unpartitioned internal volume, in ft^3

Main Wind Force Resisting System and Components and Cladding		All Heights
Table 26.11-1	**Internal Pressure Coefficient, (GC_{pi})**	**Walls & Roofs**
Enclosed, Partially Enclosed, and Open Buildings		

Enclosure Classification	(GC_{pi})
Open Buildings	0.00
Partially Enclosed Buildings	+0.55 -0.55
Enclosed Buildings	+0.18 -0.18

Notes:

1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
2. Values of (GC_{pi}) shall be used with q_z or q_h as specified.
3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:

   (i) a positive value of (GC_{pi}) applied to all internal surfaces
   (ii) a negative value of (GC_{pi}) applied to all internal surfaces

Chapter 27
WIND LOADS ON BUILDINGS—MWFRS (DIRECTIONAL PROCEDURE)

27.1 SCOPE

27.1.1 Building Types

This chapter applies to the determination of MWFRS wind loads on enclosed, partially enclosed, and open buildings of all heights using the Directional Procedure.

1) Part 1 applies to buildings of all heights where it is necessary to separate applied wind loads onto the windward, leeward, and side walls of the building to properly assess the internal forces in the MWFRS members.
2) Part 2 applies to a special class of buildings designated as enclosed simple diaphragm buildings, as defined in Section 26.2, with $h \leq 160$ ft (48.8 m).

27.1.2 Conditions

A building whose design wind loads are determined in accordance with this chapter shall comply with all of the following conditions:

1. The building is a regular-shaped building or structure as defined in Section 26.2.
2. The building does not have response characteristics making it subject to across-wind loading, vortex shedding, instability due to galloping or flutter; or it does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

27.1.3 Limitations

The provisions of this chapter take into consideration the load magnification effect caused by gusts in resonance with along-wind vibrations of flexible buildings. Buildings not meeting the requirements of Section 27.1.2, or having unusual shapes or response characteristics shall be designed using recognized literature documenting such wind load effects or shall use the wind tunnel procedure specified in Chapter 31.

27.1.4 Shielding

There shall be no reductions in velocity pressure due to apparent shielding afforded by buildings and other structures or terrain features.

27.1.5 Minimum Design Wind Loads

The wind load to be used in the design of the MWFRS for an enclosed or partially enclosed building shall not be less than 16 lb/ft^2 (0.77 kN/m^2) multiplied by the wall area of the building and 8 lb/ft^2 (0.38 kN/m^2) multiplied by the roof area of the building projected onto a vertical plane normal to the assumed wind direction. Wall and roof loads shall be applied simultaneously. The design wind force for open buildings shall be not less than 16 lb/ft^2 (0.77 kN/m^2) multiplied by the area A_f.

PART 1: ENCLOSED, PARTIALLY ENCLOSED, AND OPEN BUILDINGS OF ALL HEIGHTS

27.2 GENERAL REQUIREMENTS

The steps to determine the wind loads on the MWFRS for enclosed, partially enclosed and open buildings of all heights are provided in Table 27.2-1.

User Note: Use Part 1 of Chapter 27 to determine wind pressures on the MWFRS of enclosed, partially enclosed or an open building with any general plan shape, building height or roof geometry that matches the figures provided. These provisions utilize the traditional "all heights" method (Directional Procedure) by calculating wind pressures using *specific wind pressure equations* applicable to each building surface.

27.2.1 Wind Load Parameters Specified in Chapter 26

The following wind load parameters shall be determined in accordance with Chapter 26:

– Basic Wind Speed, V (Section 26.5)
– Wind directionality factor, K_d (Section 26.6)
– Exposure category (Section 26.7)
– Topographic factor, K_{zt} (Section 26.8)
– Gust-effect factor (Section 26.9)
– Enclosure classification (Section 26.10)
– Internal pressure coefficient, (GC_{pi}) (Section 26-11).