2012 IBC®

CODE AND COMMENTARY

Volume 2

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INTERNATIONAL CODE COUNCIL®

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2012 International Building Code® Commentary

First Printing: September 2011

ISBN:978-1-60983-063-2 (soft-cover edition)

PRINTED IN THE U.S.A.

PREFACE

The principal purpose of the Commentary is to provide a basic volume of knowledge and facts relating to building construction as it pertains to the regulations set forth in the 2012 *International Building Code*. The person who is serious about effectively designing, constructing and regulating buildings and structures will find the Commentary to be a reliable data source and reference to almost all components of the built environment

As a follow-up to the International Building Code, we offer a companion document, the *International Building Code Commentary—Volume II*. Volume II covers Chapters 16 through 35 and the appendices of the 2012 *International Building Code*. The basic appeal of the Commentary is thus: it provides in a small package and at reasonable cost thorough coverage of many issues likely to be dealt with when using the *International Building Code* — and then supplements that coverage with historical and technical background. Reference lists, information sources and bibliographies are also included.

Throughout all of this, effort has been made to keep the vast quantity of material accessible and its method of presentation useful. With a comprehensive yet concise summary of each section, the Commentary provides a convenient reference for regulations applicable to the construction of buildings and structures. In the chapters that follow, discussions focus on the full meaning and implications of the code text. Guidelines suggest the most effective method of application, and the consequences of not adhering to the code text. Illustrations are provided to aid understanding; they do not necessarily illustrate the only methods of achieving code compliance.

The format of the Commentary includes the full text of each section, table and figure in the code, followed immediately by the commentary applicable to that text. At the time of printing, the Commentary reflects the most up-to-date text of the 2012 *International Building Code*. As stated in the preface to the *International Building Code*, the content of sections in the code which begin with a letter designation (i.e., Section [P]2903.1) are maintained by another code development committee. Each section's narrative includes a statement of its objective and intent, and usually includes a discussion about why the requirement commands the conditions set forth. Code text and commentary text are easily distinguished from each other. All code text is shown as it appears in the *International Building Code*, and all commentary is indented below the code text and begins with the symbol ❖.

Readers should note that the Commentary is to be used in conjunction with the *International Building Code* and not as a substitute for the code. The Commentary is advisory only; the code official alone possesses the authority and responsibility for interpreting the code.

Comments and recommendations are encouraged, for through your input, we can improve future editions. Please direct your comments to the Codes and Standards Development Department at the Chicago District Office.

The International Code Council would like to extend its thanks to the following individuals for their contributions to the technical contentof this commentary:

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Chapter 16: Structural Design

General Comments

This chapter contains the commentary for the following structural topics: definitions of structural terms, construction document requirements, load combinations, dead loads, live loads, snow loads, wind loads, soil lateral loads, rain loads, flood loads and earthquake loads. This chapter provides minimum design requirements so that all buildings and structures are proportioned to resist the loads and forces that are likely to be encountered. The loads specified herein have been established through research and service performance of buildings and structures. The application of these loads and adherence to the serviceability criteria will enhance the protection of life and property. The earthquake loads, wind loads and snow loads in this chapter are based on the 2010 edition of ASCE 7. The earthquake criteria and ASCE 7 load requirements are based on the National Earthquake Hazards Reduction Program's (NEHRP) *Recommended Provisions for Seismic Regulations for New Buildings and other Structures* (FEMA 450). The NEHRP provisions were prepared by the Building Seismic Safety Council (BSSC) for the Federal Emergency Management Agency (FEMA).

Purpose

The purpose of this chapter is to prescribe minimum structural loading requirements for use in the design and construction of buildings and structures with the intent to minimize hazard to life and improve the occupancy capability of essential facilities after a design level event or occurrence.

SECTION 1601 GENERAL

1601.1 Scope. The provisions of this chapter shall govern the structural design of buildings, structures and portions thereof regulated by this code.

❖ While a significant portion of Chapter 16 is dedicated to the determination of minimum design loads, it also includes other important criteria that impact the design of structures, such as the permitted design methodologies, as well as the combinations of design loads used to establish the required minimum strength of structural members. Unless stated otherwise, the criteria found in this chapter are applicable to all buildings and structures. See Chapter 34 for application of these requirements to alterations, additions or repairs to existing structures.

SECTION 1602 DEFINITIONS AND NOTATIONS

1602.1 Definitions. The following terms are defined in Chapter 2:

❖ Definitions facilitate the understanding of code provisions and minimize potential confusion. To that end, this section lists definitions of terms associated with structural design. Note that these definitions are found in Chapter 2. The use and application of defined terms, as well as undefined terms, are set forth in Section 201.

ALLOWABLE STRESS DESIGN.

DEAD LOADS.

DESIGN STRENGTH.

DIAPHRAGM.

Diaphragm, blocked.

Diaphragm boundary.

Diaphragm chord.

Diaphragm flexible.

Diaphragm, rigid.

DURATION OF LOAD.

ESSENTIAL FACILITIES.

FABRIC PARTITION.

FACTORED LOAD.

HELIPAD.

ICE-SENSITIVE STRUCTURE.

IMPACT LOAD.

LIMIT STATE.

LIVE LOAD.

LIVE LOAD (ROOF).

LOAD AND RESISTANCE FACTOR DESIGN (LRFD).

LOAD EFFECTS.

LOAD FACTOR.

LOADS.

NOMINAL LOADS.

OTHER STRUCTURES.

PANEL (PART OF A STRUCTURE).

RESISTANCE FACTOR.

RISK CATEGORY.

STRENGTH, NOMINAL.

STRENGTH, REQUIRED.

STRENGTH DESIGN.

SUSCEPTIBLE BAY.

VEHICLE BARRIER.

NOTATIONS.

D = Dead load.

D_i = Weight of ice in accordance with Chapter 10 of ASCE 7.

E = Combined effect of horizontal and vertical earthquake induced forces as defined in Section 12.4.2 of ASCE 7.

F = Load due to fluids with well-defined pressures and maximum heights.

F_a = Flood load in accordance with Chapter 5 of ASCE 7.

H = Load due to lateral earth pressures, ground water pressure or pressure of bulk materials.

L = Roof live load greater than 20 psf (0.96 kN/m^2) and floor live load.

L_r = Roof live load of 20 psf (0.96 kN/m^2) or less.

R = Rain load.

S = Snow load.

T = Self-straining load.

V_{asd} = Nominal design wind speed (3-second gust), miles per hour (mph) (km/hr) where applicable.

V_{ult} = Ultimate design wind speeds (3-second gust), miles per hour (mph) (km/hr) determined from Figures 1609A, 1609B, or 1609C or ASCE 7.

W = Load due to wind pressure.

W_i = Wind-on-ice in accordance with Chapter 10 of ASCE 7.

❖ These notations are used to refer to specific nominal loads that are determined in this chapter for use in the load combinations in Section 1605:

- *D* is the nominal dead load determined in Section 1606. Also see the definition of "Dead load."
- D_i is the weight of ice. See the ASCE 7 provisions referenced in Section 1614.
- Earthquake load effect, *E*, in Section 12.4.2 of ASCE 7 includes the effects of the horizontal load, E_h, as well as a vertical component, *E*, E_h is the product of the redundancy factor, ρ, and Q_E, the effects of horizontal earthquake forces. E„ accounts for vertical acceleration due to earthquake ground motion, taken as $0.2S_{DS}D$. Note that its magnitude is not intended to represent a total vertical response, since that is not likely to coincide with the maximum horizontal response. It is essentially a portion of the dead load, D, that is added in "additive" load combinations or subtracted in "counteractive" load combinations. The term S_{DS}, design spectral response acceleration at short periods, is explained in the commentary to Section 1613.3.4.

For example, when this expression is used in the alternative allowable stress design load combinations of Section 1605.3.2 that include earthquake load effects the resulting combinations are as follows:

Equation 16-21

$$D + L + S + E/1.4 = (1 + 0.143S_{Ds})D + L + S + \rho Q_E/1.4$$

Equation 16-22

$$0.9D + E/1.4 = (0.9 - 0.143S_{Ds})D + \rho Q_E/1.4$$

Earthquake design criteria is provided in Section 1613, which, in turn, references the relevant ASCE 7 provisions for computation of the earthquake load effects. While these loads are necessary for establishing the required strength, the computed forces approximate the expected deformations under the design earthquake ground motions and are not applied to a structure in an actual earthquake.

- *F* refers to the nominal load due to fluids having "well defined pressures and maximum heights." Unlike most other nominal loads, there is no code section governing the determination of fluid loads. Also note that *F* includes a vertical component (fluid weight), as well as a horizontal component (lateral pressure).
- F_a is used to refer to the flood load that is determined under Chapter 5 of ASCE 7. Note that F_a is not explicitly included under other loads listed for the alternative ASD combination in Section 1605.3.2.
- *H* is used to refer to the nominal load resulting from lateral soil pressure, lateral pressure of ground water or the lateral pressure of bulk materials. Section 1610 specifies minimum requirements for lateral soil loads. Note that there are not specific provisions for the determination of load resulting from the lateral pressure of bulk materials.
- *L* in the nominal live load determined in accordance with Section 1607 (also see the definition of "Live load"). In addition to floor live loads, it includes roof live loads that exceed the limit on L_r. L_r represents nominal roof live loads up to 20 psf (0.96 N/m^2).
- *R* is the nominal rain load determined in accordance with Section 1611.

- *S* is the nominal snow load determined in accordance with Section 1608.
- *T* is used to refer to self-straining forces resulting from contraction or expansion due to temperature change, shrinkage, moisture change or creep, as well as movement due to differential settlement. A thermal gradient at an exterior wall is an example of a structural element where these self-straining forces can affect the design. Unlike most other nominal loads, there is no code section governing the determination of self-straining forces. *T* is not included directly in the load combinations, but reference to it is found in Sections 1605.2.1 and 1605.3.1.2.
- V_{asd} is the term used to refer to nominal design wind speeds that are determined in Section 1609.3.1.
- V_{ult} is the term used to refer to the mapped wind speeds in order to differentiate them from the nominal design wind speeds.
- *W* is the strength-level wind load determined in accordance with Section 1609.
- W_i is the wind-on-ice loading. See the ASCE 7 provisions referenced in Section 1614.

SECTION 1603 CONSTRUCTION DOCUMENTS

1603.1 General. *Construction documents* shall show the size, section and relative locations of structural members with floor levels, column centers and offsets dimensioned. The design loads and other information pertinent to the structural design required by Sections 1603.1.1 through 1603.1.9 shall be indicated on the *construction documents*.

Exception: *Construction documents* for buildings constructed in accordance with the *conventional light-frame construction* provisions of Section 2308 shall indicate the following structural design information:

1. Floor and roof live loads.
2. Ground snow load, P_g.
3. Ultimate design wind speed, V_{ult}, (3-second gust), miles per hour (mph) (km/hr) and nominal design wind speed, V_{asd}, as determined in accordance with Section 1609.3.1 and wind exposure.
4. *Seismic design category* and *site class*.
5. Flood design data, if located in *flood hazard areas* established in Section 1612.3.
6. Design load-bearing values of soils.

❖ The term “construction documents” is defined in Chapter 2. It is commonly used to refer to calculations, drawings and specifications but it includes other data that is required to indicate compliance with the code as described in Section 107. The purpose of this section is to specifically require the design professional to provide the building official with the appropriate structural details, criteria and design load data for verifying compliance with the provisions of this chapter. Note that additional structural information and specific submittal documents may also be required to be incorporated by Chapters 17 through 23.

The construction documents are required to contain sufficient detail for the building official to perform plan review and field inspection, as well as for construction activity. Dimensions indicated on architectural drawings are not required to be duplicated on the structural drawings and vice versa. The design loads, to be indicated by the design professional on the construction documents, are to be consistent with the loads used in the structural calculations. Note that the loads are not required to be on the construction drawings but must be included within the construction documents in a manner such that the design loads are clear. The building official is to compare the loads on the construction documents with the applicable minimum required loads as specified by this chapter. The inclusion of the load design information is an indication that the structure has been designed for the loads required by the code. It should be emphasized that these requirements for construction documents are applicable regardless of the involvement of a registered design professional, which is regulated by the applicable state’s licensing laws. The exception provides a less extensive list of structural data to be indicated for buildings constructed in accordance with the conventional wood-frame provisions of Section 2308. This is appropriate in view of the prescriptive nature of these requirements.

1603.1.1 Floor live load. The uniformly distributed, concentrated and impact floor live load used in the design shall be indicated for floor areas. Use of live load reduction in accordance with Section 1607.10 shall be indicated for each type of live load used in the design.

❖ The purpose of the requirement in this section is to provide information for the building official to facilitate the plan review process. The floor live loads, which are indicated on the construction documents by the design professional, are required to meet or exceed the loads in Section 1607. Any live load reductions taken are also to be indicated.

1603.1.2 Roof live load. The roof live load used in the design shall be indicated for roof areas (Section 1607.12).

❖ This section provides information allowing the building official to facilitate the plan review process. The roof live loads, indicated on the construction documents by the design professional, are required to meet or exceed the loads in Section 1607.12.

1603.1.3 Roof snow load data. The ground snow load, P_g, shall be indicated. In areas where the ground snow load, P_g, exceeds 10 pounds per square foot (psf) (0.479 kN/m^2), the following additional information shall also be provided, regardless of whether snow loads govern the design of the roof:

1. Flat-roof snow load, P_f.

2. Snow exposure factor, C_e.
3. Snow load importance factor, *I*.
4. Thermal factor, C_t.

❖ The roof snow load design basis, indicated on the construction documents (design drawings or specifications) by the design professional, provides information allowing the building official to facilitate the plan review process. The flat-roof snow load, snow exposure factor, snow load importance factor, *I*, and roof thermal factor are not to be less than the minimum requirements established by Section 1608.

1603.1.4 Wind design data. The following information related to wind loads shall be shown, regardless of whether wind loads govern the design of the lateral force-resisting system of the structure:

1. Ultimate design wind speed, V_{ult}, (3-second gust), miles per hour (km/hr) and nominal design wind speed, V_{asd}, as determined in accordance with Section 1609.3.1.
2. *Risk category*.
3. Wind exposure.Where more than one wind exposure is utilized, the wind exposure and applicable wind direction shall be indicated.
4. The applicable internal pressure coefficient.
5. Components and cladding. The design wind pressures in terms of psf (kN/m^2) to be used for the design of exterior component and cladding materials not specifically designed by the *registered design professional*.

❖ The wind load design basis, indicated on the construction documents (design drawings or specifications) by the design professional, provides information allowing the building official to facilitate the plan review process. All five of the indicated items are to be on the submitted construction documents. Each of the indicated items is an important parameter in the determination of the wind resistance that is required in the building framework. The building official should verify that the information is on the construction documents during the plan review process. The correctness of the listed items is the responsibility of the owner or the owner's design professional.

1603.1.5 Earthquake design data. The following information related to seismic loads shall be shown, regardless of whether seismic loads govern the design of the lateral force-resisting system of the structure:

1. *Risk category*.
2. Seismic importance factor, I_e.
3. Mapped spectral response acceleration parameters, S_S and S_1.
4. *Site class*.
5. Design spectral response acceleration parameters, S_{DS} and S_{D1}.
6. *Seismic design category*.
7. Basic seismic force-resisting system(s).
8. Design base shear(s).
9. Seismic response coefficient(s), C_S.
10. Response modification coefficient(s), *R*.
11. Analysis procedure used.

❖ The earthquake load design basis, indicated on the construction documents by the design professional, provides information that allows the building official to facilitate the plan review process. All buildings, except those indicated in the exceptions to Section 1613.1, are to be designed for earthquake effects. The earthquake design data for a specific building are required to meet or exceed the minimum requirements established by Section 1613.

1603.1.6 Geotechnical information. The design load-bearing values of soils shall be shown on the *construction documents*.

❖ Load-bearing values for soils must be documented so that the foundation design can be verified.

1603.1.7 Flood design data. For buildings located in whole or in part in *flood hazard areas* as established in Section 1612.3, the documentation pertaining to design, if required in Section 1612.5, shall be included and the following information, referenced to the datum on the community's Flood Insurance Rate Map (FIRM), shall be shown, regardless of whether flood loads govern the design of the building:

1. In *flood hazard areas* not subject to high-velocity wave action, the elevation of the proposed lowest floor, including the basement.
2. In *flood hazard areas* not subject to high-velocity wave action, the elevation to which any nonresidential building will be dry flood proofed.
3. In *flood hazard areas* subject to high-velocity wave action, the proposed elevation of the bottom of the lowest horizontal structural member of the lowest floor, including the basement.

❖ The flood hazard elevation information to be shown on the construction documents by the registered design professional provides information that allows the building official to facilitate the plan review process. By providing the design documentation required in Section 1612.5 and by citing the specified flood information, the registered design professional is indicating that the building was designed in accordance with the flood hazard requirements in Section 1612. If any portion of a building is in a flood hazard area, then the building must meet the corresponding flood requirements.

Depending on the nature of the designated flood hazard area, certain elevation requirements are to be met. In flood hazard areas not subject to high-velocity wave action (commonly called A/AE zones), the lowest floor of all buildings and structures, or the elevation to which nonresidential buildings are dry floodproofed, is to be located at or above the elevation specified in Section 1612.4 (which references ASCE 24). In flood hazard areas subject to high-velocity wave action (commonly called V or VE zones), the bottom of the lowest horizontal structural member is to be located at or above the elevation specified in Section 1612.4 (which references ASCE 24). These elevations are the main factor used in determining flood insurance premium rates. Constructing a building or structure with its lowest floor (or dry floodproofing) below the required elevation will result in significantly higher flood insurance premiums for the building owner.

1603.1.8 Special loads. Special loads that are applicable to the design of the building, structure or portions thereof shall be indicated along with the specified section of this code that addresses the special loading condition.

❖ Indication of special loads on the construction documents by the design professional provides information that allows the building official to facilitate the plan review process. The design professional is expected to identify any special loads that the occupancy will impose on the structure. These could include the operating weight of specialty equipment, for instance. There are also instances outside of Chapter 16 where the code specifies loading criteria that the structural design must address. For example, Section 415.8.3 requires that liquid petroleum gas facilities be in accordance with NFPA 58. In that document, a room housing a liquefied petroleum gas distribution facility must be separated from an adjacent use with ceilings and walls that are designed for a static pressure of 100 pounds per square foot (psf) (4788 Pa).

1603.1.9 Systems and components requiring special inspections for seismic resistance. *Construction documents* or specifications shall be prepared for those systems and components requiring *special inspection* for seismic resistance as specified in Section 1705.11 by the *registered design professional* responsible for their design and shall be submitted for approval in accordance with Section 107.1. Reference to seismic standards in lieu of detailed drawings is acceptable.

❖ This section provides for construction documents for the systems and components specified in Section 1705.11. Generally, the systems and components are the seismic-force-resisting systems, certain mechanical and electrical equipment and systems and architectural components for buildings where the seismic performance category is high. Construction documents are needed for these important items to verify that they comply with code requirements.

SECTION 1604
GENERAL DESIGN REQUIREMENTS

1604.1 General. Building, structures and parts thereof shall be designed and constructed in accordance with strength design, *load and resistance factor design*, *allowable stress design*, empirical design or conventional construction methods, as permitted by the applicable material chapters.

❖ This section identifies the various design methods that are permitted by the code and referenced design standards. The details for these design methods are either explicitly specified in the code or are located in the design standards that are referenced in the structural material design chapters for concrete, aluminum, masonry, steel and wood. For example, empirical design of masonry is addressed in Section 2109. The design of masonry using allowable stress design is required to be in accordance with TMS 402/ACI 530/ASCE 5 with the modifications in Section 2107.

1604.2 Strength. Buildings and other structures, and parts thereof, shall be designed and constructed to support safely the factored loads in load combinations defined in this code without exceeding the appropriate strength limit states for the materials of construction. Alternatively, buildings and other structures, and parts thereof, shall be designed and constructed to support safely the *nominal loads* in load combinations defined in this code without exceeding the appropriate specified allowable stresses for the materials of construction.

Loads and forces for occupancies or uses not covered in this chapter shall be subject to the approval of the *building official*.

❖ This section describes the strength and allowable stress design methods in the code. It also gives the building official approval authority for structural loads in buildings used for occupancies that are not specifically addressed in this chapter.

1604.3 Serviceability. Structural systems and members thereof shall be designed to have adequate stiffness to limit deflections and lateral drift. See Section 12.12.1 of ASCE 7 for drift limits applicable to earthquake loading.

❖ The stated objective in this section is to limit member deflections and system drifts by providing adequate stiffness. Note that the definition of “Limit state” describes a serviceability limit state as a condition beyond which a structure or member is no longer useful for its intended purpose. This would be in contrast to a strength limit state beyond which the structure or member is considered unsafe. The code does not directly address a serviceability limit state. Instead the code limits certain member deflections for specific nominal loads in addition to the story drift limits for earthquake load effects.

The deflection limits for structural members must be in accordance with the requirements in this and subsequent sections. This section also provides a reference to the story drift limitations in the earthquake provisions. Generally, deflection limits are needed for the comfort of the building occupants and so that the structural member’s deflection does not cause damage to supported construction. Excessive deflection can also contribute to excessive vibration, which is discomforting to the occupants of the building.

TABLE 1604.3. See below.

❖ The deflection limits in this table apply when they are more restrictive than those in the structural design standards that are indicated in Sections 1604.3.2 through 1604.3.5. Note that the deflection limits for exterior walls and interior partitions vary depending on the type of finish (i.e., flexible, plaster and stucco or other brittle finishes). A flexible finish is intended to be one that has been designed to accommodate the higher deflection indicated and remain serviceable. A brittle finish is any finish that has not been designed to accommodate the deflection allowed for a flexible finish. The more restrictive limit for plaster or stucco is based on ASTM C 926. The deflection limit for a roof member supporting a plaster ceiling is intended to apply only for a plaster ceiling. The limit for a roof member supporting a gypsum board ceiling is that listed in the table for "supporting a nonplaster ceiling."

The answers to the following frequently asked questions provide further guidance on applying the deflection limits of the code.

Frequently Asked Questions—Table 1604.3

Q1. For purposes of checking the deflection limits of Table 1604.3, should the calculated wind load be used directly or the combined loads from Section 1605.3.1 or 1605.3.2?

A1. In computing deflections to verify compliance with Table 1604.3 limits, the loads shown in the column headings of Table 1604.3 are the only loads that must be applied to the member. The load used to check the deflection in this case is W, the nominal wind load in accordance with Section 1609. It is not necessary to use the load combinations of Section 1605.3 for verifying that the deflection limits have been met.

Q2. Note f permits the wind load to be taken as 0.42 times the "component and cladding" loads for the purpose of determining deflection limits. Please explain the basis for this 0.42 adjustment.

A2. There are two aspects to this adjustment. The first is the conversion from a wind speed with a 50-year mean recurrence interval (MRI) to a 10-year MRI event at an allowable stress design level. The mapped wind speeds of the code have generally been based on an MRI of 50 years. Serviceability checks, such as deflection, have typically been based on a lower MRI (for example, 10 years). The ASCE 7 commentary to the 2005 edition provided factors to covert to wind speeds with MRIs other than 50 years in

TABLE 1604.3
DEFLECTION LIMITS[a, b, c, h, i]

CONSTRUCTION	*L*	*S* or *W*[f]	*D* + *L*[d, g]
Roof members:[e]			
Supporting plaster or stucco ceiling	*l*/360	*l*/360	*l*/240
Supporting nonplaster ceiling	*l*/240	*l*/240	*l*/180
Not supporting ceiling	*l*/180	*l*/180	*l*/120
Floor members	*l*/360	—	*l*/240
Exterior walls and interior partitions:			
With plaster or stucco finishes	—	*l*/360	—
With other brittle finishes	—	*l*/240	—
With flexible finishes	—	*l*/120	—
Farm buildings	—	—	*l*/180
Greenhouses	—	—	*l*/120

For SI: 1 foot = 304.8 mm.

a. For structural roofing and siding made of formed metal sheets, the total load deflection shall not exceed *l*/60. For secondary roof structural members supporting formed metal roofing, the live load deflection shall not exceed *l*/150. For secondary wall members supporting formed metal siding, the design wind load deflection shall not exceed *l*/90. For roofs, this exception only applies when the metal sheets have no roof covering.

b. Interior partitions not exceeding 6 feet in height and flexible, folding and portable partitions are not governed by the provisions of this section. The deflection criterion for interior partitions is based on the horizontal load defined in Section 1607.14.

c. See Section 2403 for glass supports.

d. For wood structural members having a moisture content of less than 16 percent at time of installation and used under dry conditions, the deflection resulting from $L + 0.5D$ is permitted to be substituted for the deflection resulting from $L + D$.

e. The above deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to assure adequate drainage shall be investigated for ponding. See Section 1611 for rain and ponding requirements and Section 1503.4 for roof drainage requirements.

f. The wind load is permitted to be taken as 0.42 times the "component and cladding" loads for the purpose of determining deflection limits herein.

g. For steel structural members, the dead load shall be taken as zero.

h. For aluminum structural members or aluminum panels used in skylights and sloped glazing framing, roofs or walls of sunroom additions or patio covers, not supporting edge of glass or aluminum sandwich panels, the total load deflection shall not exceed *l*/60. For continuous aluminum structural members supporting edge of glass, the total load deflection shall not exceed *l*/175 for each glass lite or *l*/60 for the entire length of the member, whichever is more stringent. For aluminum sandwich panels used in roofs or walls of sunroom additions or patio covers, the total load deflection shall not exceed *l*/120.

i. For cantilever members, *l* shall be taken as twice the length of the cantilever.

Table C6-7. For wind speeds between 85 mph and 100 mph, the factor for 10 years in Table C6-7 was 0.84. This conversion factor applies to the wind speed, *V*. Since design wind pressure is a function of V^2, the conversion factor must be squared before applying it to the design wind pressure. The factor 0.84 squared is 0.7056, which is rounded off to 0.7, which was the factor given in previous editions of the code. The second aspect is the conversion of the wind load from strength level to allowable stress design level which means the 0.7 factor is multiplied by 0.6, giving rise to the factor of 0.42 in the footnote.

Q3. In Table 1604.3, Note g states "dead load shall be taken as zero for structural steel members." Would this apply to the precomposition check of composite beam deflection limits under wet weight of concrete?

A3. No. The serviceability requirements of Section 1604.3 apply to the finished construction. The loading condition described would be a construction consideration, which is not directly regulated by the serviceability criteria.

1604.3.1 Deflections. The deflections of structural members shall not exceed the more restrictive of the limitations of Sections 1604.3.2 through 1604.3.5 or that permitted by Table 1604.3.

❖ The deflection of structural members is limited by the code, as well as certain material standards for damage control of supported construction and human comfort. Generally, the public equates visible deflection, or even detectable vibration, with a potentially unsafe condition (which in many cases is not true). The intent of this section is that deflection is not to exceed either the limitations in the applicable material design standard or the applicable specified requirements.

1604.3.2 Reinforced concrete. The deflection of reinforced concrete structural members shall not exceed that permitted by ACI 318.

❖ The deflection limitations in ACI 318 are not to be exceeded for reinforced (and prestressed) concrete (see ACI 318 for detailed deflection requirements).

1604.3.3 Steel. The deflection of steel structural members shall not exceed that permitted by AISC 360, AISI S100, ASCE 8, SJI CJ-1.0, SJI JG-1.1, SJI K-1.1 or SJI LH/DLH-1.1, as applicable.

❖ The design standard to be met depends on the type of steel structural member. For example, steel joists are to meet the deflection limitations in the Steel Joist Institute's (SJI) standard, which is applicable to the type of joist, and rolled steel members are to meet the deflection criteria in AISC 360.

1604.3.4 Masonry. The deflection of masonry structural members shall not exceed that permitted by TMS 402/ACI 530/ASCE 5.

❖ The deflection limits for masonry beams and lintels is specified in TMS 402/ACI 530/ASCE 5.

1604.3.5 Aluminum. The deflection of aluminum structural members shall not exceed that permitted by AA ADM1.

❖ The deflection of aluminum structural members is limited to that in AA-ADM1 for aluminum or that permitted by Table 1604.3 of the code.

1604.3.6 Limits. The deflection limits of Section 1604.3.1 shall be used unless more restrictive deflection limits are required by a referenced standard for the element or finish material.

❖ The limits specified in Table 1604.3 apply to the indicated members for any structural material. As indicated in Section 1604.3.1, the deflection limits in the structural material standards apply when they are more restrictive than indicated in the table.

The deflection limits that are applicable to a flat concrete roof member that does not support any nonstructural elements likely to be damaged by large deflections are summarized in Table 1604.3.6. As can be seen, the code deflection criteria is more stringent and would govern the design of this type of member.

Table 1604.3.6
CONCRETE ROOF MEMBER DEFLECTION CRITERIA

Construction	LOADS		
	L	***S* or *W***	***D* + *L***
IBC Table 1604.3			
Roof members	*l*/360	*l*/360	*l*/240
Concrete per ACI 318 See Table 9.5(b)			
Flat roofs	*l*/180	Note a	Note a

a. No deflection limit specified in ACI 318.

1604.4 Analysis. *Load effects* on structural members and their connections 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.

Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics. Such analysis shall result in a system that provides a complete load path capable of transferring loads from their point of origin to the load-resisting elements.

The total lateral force shall be distributed to the various vertical elements of the lateral force-resisting system in proportion to their rigidities, considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements assumed not to be a part of the lateral force-resisting system are permitted to be incorporated into buildings provided their effect on the action of the system is considered and provided for in the design. Except where diaphragms are flexible, or are permitted to be analyzed as flexible, provisions shall be made for the increased forces induced on resisting elements of the structural system resulting from torsion due to eccentricity between the center of application of the lateral forces and the center of rigidity of the lateral force-resisting system.

Every structure shall be designed to resist the overturning effects caused by the lateral forces specified in this chapter. See Section 1609 for wind loads, Section 1610 for lateral soil loads and Section 1613 for earthquake loads.

❖ This section includes the general requirements for structural analysis. The principles stated in this section are those commonly found in structural engineering textbooks. The requirement that structural analysis be capable of demonstrating a complete load path is essential to the adequate resistance of the structural system to wind loads or earthquake effects. The load path is to be capable of transferring all of the loads from their point of application onto the structure to the foundation. It is also important that nonstructural rigid elements be properly accounted for in the design. For example, a partial-height rigid masonry wall placed between steel columns in a steel frame will resist the horizontal shear load and cause bending in the column unless a flexible joint is provided between the wall and the column.

The definition of "Diaphragm" in Section 202.1 includes definitions that distinguish a rigid diaphragm from a flexible diaphragm. The distribution of forces in buildings with flexible diaphragms differs from those having rigid diaphragms. Where the diaphragm is determined to be flexible, the effect of diaphragm rigidity on the distribution of lateral forces is considered to be negligible and can therefore be neglected in the structural analysis. Otherwise, for structures having rigid diaphragms, this section requires the engineer to distribute lateral forces to the vertical supporting elements in proportion to their rigidities, and to include the effect of the increased forces induced on the vertical supporting elements resulting from torsion due to eccentricity between the center of mass and the center of rigidity.

1604.5 Risk category. Each building and structure shall be assigned a *risk category* in accordance with Table 1604.5. Where a referenced standard specifies an occupancy category, the *risk category* shall not be taken as lower than the occupancy category specified therein.

❖ This section requires classification of the risk category of any building in accordance with the nature of occupancy as described in Table 1604.5. The risk category serves as a threshold for a variety of code provisions related to earthquake, flood, snow and wind loads. Particularly noteworthy are the importance factors that are used in the calculation of design earthquake, snow and wind loads. The value of the importance factor generally increases with the importance of the facility. Structures assigned greater importance factors must be designed for larger forces. The result is a more robust structure that would be less likely to sustain damage under the same conditions than a structure with a lower importance factor. The intent is to enhance a structure's performance based upon its use or the need to remain in operation during and after a design event.

The impact of a higher risk category classification is not limited to increasing the design loads. Compared to Risk Category I, II or III, for instance, a Risk Category IV classification can lead to a higher seismic design category classification that can, in turn, require more stringent seismic detailing and limitations on the seismic-force-resisting system. This can also affect the seismic design requirements for architectural, mechanical and electrical components and systems.

TABLE 1604.5. See page 16-9.

❖ The risk category determined in this table generally increases with the importance of the facility, which relates to the availability of the facility after an emergency, and the consequence of a structural failure on human life. The categories range from Risk Category I, which represents the lowest hazard to life, through Risk Category IV, which encompasses essential facilities.

Risk Category IV: These are buildings that are considered to be essential in that their continuous use is needed, particularly in response to disasters. Fire, rescue and police stations, and emergency vehicle garages must remain operational during and after major events, such as earthquakes, floods or hurricanes. The phrase "designated as essential facilities" refers to designation by the building official that certain facilities are required for emergency response or disaster recovery. This provides jurisdictions the latitude to identify specific facilities that should be considered essential in responding to various types of emergencies. These could include structures that would not otherwise be included in this risk category. This designation would only be made with consideration of broader public policy, as well as emergency preparedness planning within the jurisdiction in question. The reasons for including facilities, such as hospitals, fire stations, police stations, emergency response operations centers, etc., should be self-evident. Some items warranting additional discussion are as follows:

- *Designated emergency shelters and designated emergency response facilities.* These items repeat the term "designated," which is referring to designation by the building official that the facilities have been identified as necessary for sheltering evacuees or responding to emergencies (see discussion of "designated-above). For example, an elementary school having an occupant load of 275 would typically be considered a Risk Category III facility. If that school is designated as an emergency shelter, then the school will be considered a Risk Category IV building.

- *Facilities supplying emergency backup power for Risk Category IV.* A power-generating station or other utility (such as a natural gas facility) is to be classified as Risk Category IV only if the facility serves an emergency backup function for a Risk Category IV building, such as a fire station or police station. Otherwise, the power-generating station or utility should be classified as Risk Category III.
- *Structures with quantities of highly toxic materials in excess of the quantities permitted for a control area in Table 307.1(2).* This applies only to "Highly toxic materials" (see definition in Section 307.2), which are covered in the second row of Table 307.1(2). That table lists the maximum allowable quantities per control area of materials posing a health hazard. Since the use of control areas is permitted by Table 307.1(2), it is reasonable to recognize the control area for the purpose of making this risk category determination. In other words, this applies to occupancies

TABLE 1604.5
RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

RISK CATEGORY	NATURE OF OCCUPANCY
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	Buildings and other structures except those listed in Risk Categories I, III and IV
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: • Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structures containing elementary school, secondary school or day care facilities with an occupant load greater than 250. • Buildings and other structures containing adult education facilities, such as colleges and universities, with an occupant load greater than 500. • Group I-2 occupancies with an occupant load of 50 or more resident care recipients but not having surgery or emergency treatment facilities. • Group I-3 occupancies. • Any other occupancy with an occupant load greater than 5,000[a]. • Power-generating stations, water treatment facilities for potable water, waste water treatment facilities and other public utility facilities not included in Risk Category IV. • Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive materials that: Exceed maximum allowable quantities per control area as given in Table 307.1(1) or 307.1(2) or per outdoor control area in accordance with the *International Fire Code*; and Are sufficient to pose a threat to the public if released [b].
IV	Buildings and other structures designated as essential facilities, including but not limited to: • Group I-2 occupancies having surgery or emergency treatment facilities. • Fire, rescue, ambulance and police stations and emergency vehicle garages. • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communications and operations centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Risk Category IV structures. • Buildings and other structures containing quantities of highly toxic materials that: Exceed maximum allowable quantities per control area as given in Table 307.1(2) or per outdoor control area in accordance with the *International Fire Code*; and Are sufficient to pose a threat to the public if released [b]. • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water storage facilities and pump structures required to maintain water pressure for fire suppression.

a. For purposes of occupant load calculation, occupancies required by Table 1004.1.2 to use gross floor area calculations shall be permitted to use net floor areas to determine the total occupant load.

b. Where approved by the building official, the classification of buildings and other structures as Risk Category III or IV based on their quantities of toxic, highly toxic or explosive materials is permitted to be reduced to Risk Category II, provided it can be demonstrated by a hazard assessment in accordance with Section 1.5.3 of ASCE 7 that a release of the toxic, highly toxic or explosive materials is not sufficient to pose a threat to the public.

that are classified as Group H-4 based on the quantities of highly toxic material exceeding the permitted quantity within a control area. However, recognizing control areas means that both the risk category as well as the occupancy classification could be lowered by adding either fire-resistance-rated walls or floor/ceiling assemblies in order to divide the building into a number of smaller control areas. The additional wording "….sufficient to pose a threat to the public if released" places a further qualification on the material quantity, but it is subjective since the threat to the public could be difficult to determine. Also note that a Group H-4 occupancy classification could be based on exceeding the quantities permitted for toxics or corrosives [see Table 307.1(2)], but the presence of those materials would not affect the assessment of the facility as Risk Category IV.

Risk Category III: Risk Category III buildings include those occupancies that have relatively large numbers of occupants because of the overall size of the building. They also include uses that pose an elevated life-safety hazard to the occupants, such as public assembly, schools or colleges. In addition, Risk Category III includes uses where the occupant's ability to respond to an emergency is either restricted, such as in jails, or otherwise impaired, such as in nursing homes housing patients that require skilled nursing care. A discussion of some of the specific table listings follows:

- *Buildings and other structures with a primary occupancy that is public assembly with an occupant load greater than 300.* Public assembly occupancies meeting this criterion will typically be classified as Group A in Chapter 3. The wording requires agreement on the determination that a building's "primary occupancy" is in fact public assembly. This could be as simple as verifying that the portion of the building housing the public assembly occupancy is more than 50 percent of the total building area.
- *Group 1-2 occupancies with an occupant load of 50 or more resident patients but not having surgery or emergency treatment facilities.* This category applies to health care facilities with at least 50 resident patients. The term "resident patient" is not defined or used elsewhere in the code, but would seem to refer to locations where those patients receive around-the-clock (24-hour) care as opposed to ambulatory surgery centers or outpatient units. This table entry covers facilities where patients have difficulty responding to an emergency or are incapable of self-preservation.
- *Buildings having an occupant load greater than 5,000.* Uses that pose elevated life-safety concerns, such as public assembly uses, schools and health care facilities, are covered elsewhere and have a much lower threshold based on the number of occupants. This table entry covers buildings that are large enough to have more than 5,000 occupants, providing added protection for the occupants of larger structures whatever the use happens to be. In order to determine occupant load, the methods outlined in Section 1004 are normally used. Chapter 10 sets forth standards that provide a reasonably conservative number of occupants for all spaces, and while actual loads are commonly less than the design amount, it is not unusual in the life of a space in a building to have periods when high actual occupant loads exist. Because there is no clear rationale that connects the occupant load used to calculate minimum means of egress requirements to the risks associated with structural design, Note a provides some reasonable adjustment to this determination by permitting the use of net floor area. It provides a more reasonable approach for occupancies, such as office, mercantile and residential, that are required to base occupant load on gross area—an area that includes corridors, stairways, elevators, closets, accessory areas, structural walls and columns, etc.
- *Power-generating stations, potable water treatment facilities, wastewater treatment and other public utilities not included in Risk Category IV.* A failure and subsequent shutdown of these types of facilities would not pose an imme-diate threat to life safety. These infrastructure items are considered Risk Category III because of the impact an extended disruption in service can have on the public.
- *Buildings not included in Risk Category IV containing quantities of toxic or explosive materials that exceed permitted quantities per control area in Table 307.1(1) or 307.1(2) and are sufficient to pose a threat to the public if released.* Buildings included under Risk Category IV would be those containing quantities of highly toxic materials that exceed the permitted quantity in Table 307.1(2) (see discussion under "Risk Category IV" above). This item addresses buildings with explosives or toxic substances, both of which are defined in Section 307.2.

Risk Category II: Risk Category II buildings represent a lesser hazard to life because of fewer building occupants and smaller building size compared to those that are considered Risk Category III. Since Risk Categories III and IV represent buildings with higher risk or essential facilities, on a relative scale Risk Category II can be thought of as a "standard occupancy" building as evidenced by importance factors for earthquake, snow and wind that are all equal to 1.

Risk category I: Risk Category I buildings exhibit the lowest hazard to life since they have little or no human occupants or, for those that are temporary, the exposure to the hazards of earthquakes, floods, snow and wind would be considerably less than that of a permanent structure. Note that this category includes "minor storage facilities," but the code does not provide an explanation of which storage facilities could be considered minor.

1604.5.1 Multiple occupancies. Where a building or structure is occupied by two or more occupancies not included in the same *risk category*, it shall be assigned the classification of the highest *risk category* corresponding to the various occupancies. Where buildings or structures have two or more portions that are structurally separated, each portion shall be separately classified. Where a separated portion of a building or structure provides required access to, required egress from or shares life safety components with another portion having a higher *risk category*, both portions shall be assigned to the higher *risk category*.

❖ Buildings are frequently occupied by a mixture of uses or occupancies. A single-use building is probably the exception rather than the rule. Where multiple occupancies are proposed in a building, the risk category of each one must be considered. In some cases, the proposed occupancies in a building will fall into more than one risk category and the requirements for multiple occupancies stated in Section 1604.5.1 must be satisfied. These requirements were previously part of the earthquake load provisions. As of the 2006 edition, they were relocated so that they now apply regardless of the type of load being considered. The code identifies two design options for mixed-use buildings. The entire structure can be designed as a single unit based on the requirements for the most stringent risk category for the building. Alternatively, the engineer can structurally separate portions of the structure containing distinct occupancy categories and design each portion accordingly based on its risk category.

This section also provides direction regarding access to and egress from adjacent structures that fall into different occupancy categories by making the requirements for the more stringent risk category applicable to both structures. This requirement is the result of lessons learned from events, such as earthquakes, in which essential functions have been rendered unusable because of a failure in an adjacent structure.

1604.6 In-situ load tests. The *building official* is authorized to require an engineering analysis or a load test, or both, of any construction whenever there is reason to question the safety of the construction for the intended occupancy. Engineering analysis and load tests shall be conducted in accordance with Section 1709.

❖ The building official has the option of requiring either a structural analysis, an on-site in-situ load test, or both, in accordance with Section 1714, on an existing structure, building or portion thereof if there is reasonable doubt as to structural integrity. The building official should document his or her reasons for the testing requirement. Whenever possible, the concern should be addressed by structural analysis since load testing a structure is very expensive. One example would be an analysis by a third-party engineering firm acceptable to both the building official and the owner. The structural integrity may be examined for items such as visible signs of excessive settlement or lateral deflection, such as cracks in concrete foundation walls or excessive vibration when the assembly is loaded. The procedure must simulate the actual load conditions to which the structure is subjected during normal use.

1604.7 Preconstruction load tests. Materials and methods of construction that are not capable of being designed by *approved* engineering analysis or that do not comply with the applicable referenced standards, or alternative test procedures in accordance with Section 1707, shall be load tested in accordance with Section 1710.

❖ The alternative test procedure described in Section 1707 and the preconstruction load test procedure described in Section 1710 are intended to apply to materials or an assembly of structural materials that do not have an accepted analysis technique; thus, they are approved for use by way of the alternative test procedure. The preconstruction test procedure in Section 1710 includes the determination of the allowable superimposed design load (see Section 1710 for details).

1604.8 Anchorage. Buildings and other structures, and portions thereof, shall be provided with anchorage in accordance with Sections 1604.8.1 through 1604.8.3, as applicable.

❖ Sections 1604.8.1 through 1606.8.3 contain general anchorage requirements as well as specific requirements for the anchorage of walls and decks.

1604.8.1 General. Anchorage of the roof to walls and columns, and of walls and columns to foundations, shall be provided to resist the uplift and sliding forces that result from the application of the prescribed loads.

❖ This section states a specific load path requirement for anchorage that is already required in more general terms by Section 1604.4. Required anchorage resistance to uplift and sliding is determined in accordance with the appropriate load combinations in Section 1605, which account for the effects of earthquakes, fluid pressures, lateral earth pressure or wind (also see Section 1604.9).

An example of the condition described in this section is roof uplift as a result of high winds. Table 2308.10.1 requires roof uplift connectors for wood-frame construction designed and installed in accordance with the prescriptive requirements for conventional light-frame construction. Note that net roof uplift occurs at all wind speeds indicated in the table; thus, uplift connectors are required for roof wood framing for all of the indicated wind speed locations.

1604.8.2 Structural walls. Walls that provide vertical load-bearing resistance or lateral shear resistance for a portion of the structure shall be anchored to the roof and to all floors and members that provide lateral support for the wall or that are supported by the wall. The connections shall be capable of resisting the horizontal forces specified in Section 1.4.4 of ASCE 7 for walls of structures assigned to Seismic Design Category A and to Section 12.11 of ASCE 7 for walls of structures assigned to all other seismic design categories. Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall. See Sections 1609 for wind design requirements and 1613 for earthquake design requirements.

❖ This section refers to the ASCE 7 technical provisions for the minimum strength design load on anchorages of walls to the horizontal diaphragms that provide lateral support. The connection requirements apply to both bearing walls and walls that resist lateral loads. The wall anchorage must be a positive connection that does not rely on friction for load transfer. Earthquake detailing provisions may result in higher anchorage design forces based on the seismic design category of the building, as well as the mass of the wall.

1604.8.3 Decks. Where supported by attachment to an *exterior wall*, decks shall be positively anchored to the primary structure and designed for both vertical and lateral loads as applicable. Such attachment shall not be accomplished by the use of toenails or nails subject to withdrawal. Where positive connection to the primary building structure cannot be verified during inspection, decks shall be self-supporting. Connections of decks with cantilevered framing members to exterior walls or other framing members shall be designed for both of the following:

1. The reactions resulting from the dead load and live load specified in Table 1607.1, or the snow load specified in Section 1608, in accordance with Section 1605, acting on all portions of the deck.
2. The reactions resulting from the dead load and live load specified in Table 1607.1, or the snow load specified in Section 1608, in accordance with Section 1605, acting on the cantilevered portion of the deck, and no live load or snow load on the remaining portion of the deck.

❖ This requirement for the positive anchorage of decks is a result of failures that have occurred primarily on nonengineered residential decks. Toenail connections are very weak since many of the field connections split the wood framing member. Nails that are installed in line with applied tension forces pull out easily. Wood deck framing could be attached to a supporting ledger board by joist hangers that provide nails loaded in shear for the vertical and horizontal loads.

In order to make the loading considerations perfectly clear for decks with cantilevers, this section states the two loading conditions that are applicable. The second condition covers the case where uplift can occur at the point of connection between the wood framing and the exterior wall for cantilevered deck framing. This highlights the need for resistance to uplift by a positive connection at the exterior wall. These conditions also clarify that snow load should be considered.

1604.9 Counteracting structural actions. Structural members, systems, components and cladding shall be designed to resist forces due to earthquakes and wind, with consideration of overturning, sliding and uplift. 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.

❖ This section requires that all elements of the structure be designed to resist the required earthquake and wind effects. The required level of resistance will be based on the appropriate load combinations of Section 1605. The term "counteracting" used in the section title refers to so-called counteracting load combinations as opposed to those in which the load effects are additive. The effects of overturning, uplift and sliding are most pronounced in these counteractive load combinations. These would consist of the following depending on the design method chosen:

- Load and resistance factor design or strength design, Section 1605.2:

  $0.9D + 1.0W + 1.6H$ Equation 16-6

  $0.9(D + F) + 1.0E + 1.6H$ Equation 16-7

- Basic allowable stress design load combinations, Section 1605.3.1:

  $0.6D + 0.6W + H$ Equation 16-15

  $0.6(D + F) + 0.7E + H$ Equation 16-16

- Alternative allowable stress design load combinations, Section 1605.3.2:

  $0.9D + E/1.4$ Equation 16-22

  (Note the adjustment to the vertical component of earthquake load effect that is permitted for evaluating foundations.)

  $(^2/_3)D + (0.6 \times 1.3)W$ Equation 16-18

  (Equation 16-18 with L = 0 in accordance with Section 1605.1 requirement to investigate each load combination with one or more variable loads equal to zero, two-thirds of the dead load in accordance with requirement in Section 1605.3.2 and w set equal to 1.3 for 3-second gust wind speed.)

As can be seen, for the critical counteracting load combinations, resistance to overturning, uplift and sliding is largely provided by dead load. Note that Section 1605.3.2 requires the designer to consider only the dead load likely to be in place during a design wind event when using the alternative allowable stress load combinations. Regardless of the design methodology utilized, the designer should be cautious in estimating dead loads that resist overturning, sliding and uplift. In order to remain conservative, estimated dead load should be no more than the actual dead load in these cases (see design dead load in Section 1606.2). To keep a structure from sliding horizontally, the dead load must generate enough friction at the base of the structure to resist the horizontal base shear due to earthquake and wind. Otherwise, adequate anchor-age must be provided to resist the base shear.

1604.10 Wind and seismic detailing. Lateral force-resisting systems shall meet seismic detailing requirements and limitations prescribed in this code and ASCE 7, excluding Chapter 14 and Appendix 11A, even when wind *load effects* are greater than seismic *load effects*.

❖ This section is needed to clarify that the seismic detailing requirements of the code always apply, even where the wind load effects are higher than the seismic load effects. This is required because the calculated earthquake load is dependent upon an assumed level of ductility to be provided in the selected seismic-force-resisting system. If the designer does not follow through by providing the detailing required to achieve the ductility of the selected system, the assumption is invalid.

For example, consider the case where the earthquake load provisions apply in accordance with Section 1613.1, the seismic-force-resisting system is an ordinary concentrically braced steel frame and, based on the seismic design category, Section 2205 requires that the AISC 341 seismic provisions are to be met. For this case, the requirements in Section 14 of AISC 341 must be satisfied in the design of the steel frame, even if the wind load effects exceed the seismic load effects.

SECTION 1605 LOAD COMBINATIONS

1605.1 General. Buildings and other structures and portions thereof shall be designed to resist:

1. The load combinations specified in Section 1605.2, 1605.3.1 or 1605.3.2;
2. The load combinations specified in Chapters 18 through 23; and
3. The seismic load effects including overstrength factor in accordance with Section 12.4.3 of ASCE 7 where required by Section 12.2.5.2, 12.3.3.3 or 12.10.2.1 of ASCE 7. With the simplified procedure of ASCE 7 Section 12.14, the seismic load effects including overstrength factor in accordance with Section 12.14.3.2 of ASCE 7 shall be used.

Applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations. Each load combination shall also be investigated with one or more of the variable loads set to zero.

Where the load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7 apply, they shall be used as follows:

1. The basic combinations for strength design with overstrength factor in lieu of Equations 16-5 and 16-7 in Section 1605.2.
2. The basic combinations for *allowable stress design* with overstrength factor in lieu of Equations 16-12, 16-14 and 16-16 in Section 1605.3.1.
3. The basic combinations for *allowable stress design* with overstrength factor in lieu of Equations 16-21 and 16-22 in Section 1605.3.2.

❖ Generally, there are two types of load combinations specified in the code: those to be used with a strength design or load and resistance factor design (LRFD) and those to be used with allowable stress design. Where additional load combinations are specified in the structural material chapters, they apply also. Note that this section also requires that for a given load combination engineers also consider additional load cases where one or more variable loads are not acting concurrently with other variable loads. It is necessary to explore such possibilities, since they can at times result in the most critical load effect for some elements of a structure. According to the definition of "Loads," loads that are not considered permanent are variable loads.

The load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7 only apply where specifically required by the seismic provisions. They do not replace the applicable load combinations for strength design or allowable stress design. They constitute an additional requirement that must be considered in the design of specific structural elements.

The purpose of these load combinations with overstrength factor is to account for the maximum earthquake load effect, E_m, which considers "system overstrength." This system characteristic is accounted for by multiplying the effects of the lateral earthquake load by the overstrength factor, Ω_0, for the seismic-force-resiting system that is utilized. It is representative of the upper bound system strength for purposes of designing nonyielding elements for the maximum expected load. Under the design earthquake ground motions, the forces generated in the seismic-force-resisting system can be much greater than the prescribed seismic design forces. If not accounted for, the system overstrength effect can cause failures of structural elements that are subjected to these forces. Because system overstrength is unavoidable, design for the maximum earthquake force that can be developed is warranted for certain elements. The intent is to provide key elements with sufficient overstrength so that inelastic (ductile) response/behavior appropriately occurs within the vertical-resisting elements.

The ASCE 7 earthquake load provisions that require consideration of the load combinations with over-strength factor are stated in this section. Note that these load combinations are not general load cases, but are only to be applied where specified in the indicated earthquake load provisions (see above) or in the structural materials requirements (see Section 1810.3.11.2 for an example of the latter).

For example, Section 12.3.3.3 of ASCE 7 applies to structural elements that support discontinuous frames or shear wall systems where the discontinuity is severe enough to be deemed a structural irregularity. See Figure 1605.1(1) for examples of building configurations that include discontinuous vertical systems. Also see the commentary to Section 12.3.3.3 of ASCE 7.

Section 12.10.2.1 of ASCE 7 applies to collector elements in structures assigned to Seismic Design Category C or higher. The term “collector” describes an element used to transfer forces from a diaphragm to the supporting element(s) of the lateral-force-resisting system where the length of the vertical element is less than the diaphragm dimension at that location. Figure 1605.1(2) illustrates this relationship. Failures of collectors can result in loads not being delivered to resisting elements and separation of a portion of a building from its lateral-force-resisting system. Where a collector also supports gravity loads, such a failure would result in the loss of a portion of the gravity load-carrying system. The intent is to provide collector elements that have sufficient overstrength so that any inelastic behavior appropriately occurs within the seismic-force-resisting system rather than the collector elements.

Maximum earthquake load effect, E_m, in Section 12.4.3 of ASCE 7 includes effects of the horizontal load, E_{mh}, as well as a vertical component, E_v. Emh is the product of the overstrength factor, Ω_0, and Q_E, the effects of horizontal earthquake forces. E_v is the same vertical component used for the earthquake load effect, E (see commentary, Section 1602.1).

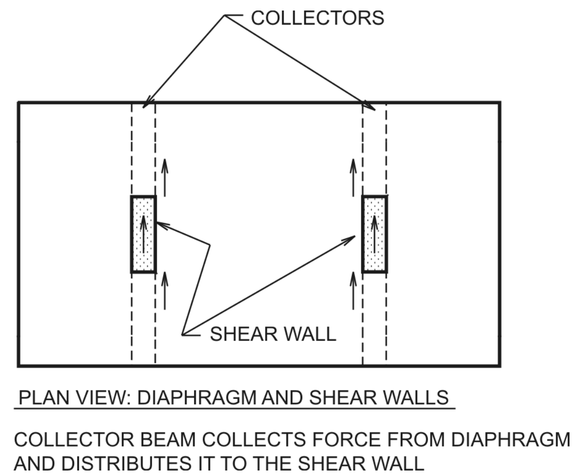


Figure 1605.1(2)
COLLECTORS

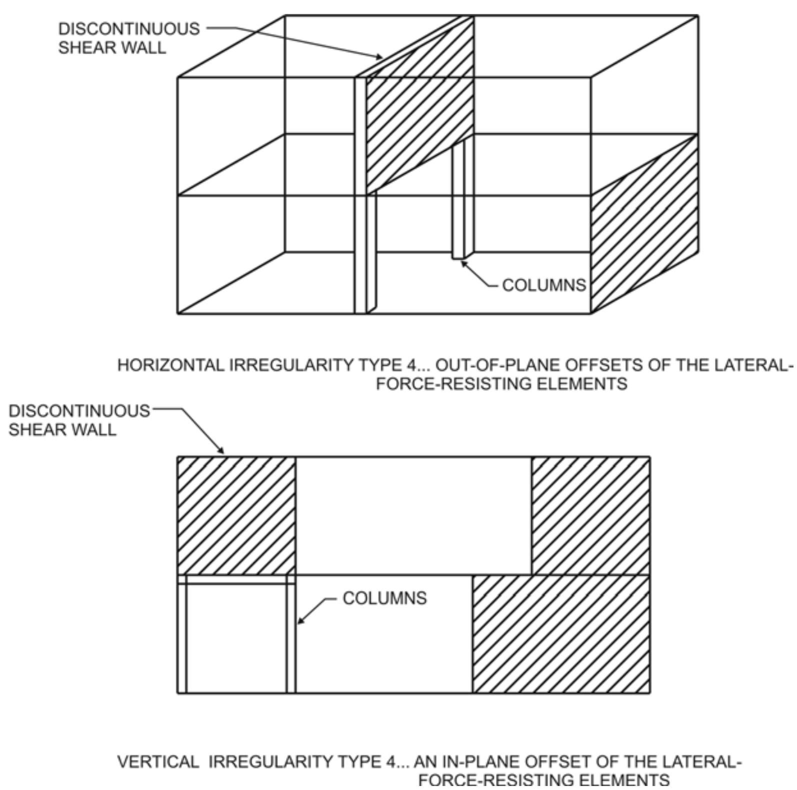


Figure 1605.1(1)
EXAMPLES OF DISCONTINUOUS SHEAR WALLS

1605.1.1 Stability. Regardless of which load combinations are used to design for strength, where overall structure stability (such as stability against overturning, sliding, or buoyancy) is being verified, use of the load combinations specified in Section 1605.2 or 1605.3 shall be permitted. Where the load combinations specified in Section 1605.2 are used, strength reduction factors applicable to soil resistance shall be provided by a *registered design professional*. The stability of retaining walls shall be verified in accordance with Section 1807.2.3.

❖ This section permits soil resistance and strength reduction factors to be considered where strength design factored loads are used in foundation design.

1605.2 Load combinations using strength design or load and resistance factor design. Where strength design or load and resistance factor design is used, buildings and other structures, and portions thereof, shall be designed to resist the most critical effects resulting from the following combinations of factored loads:

$1.4(D + F)$ **(Equation 16-1)**

$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$ **(Equation 16-2)**

$1.2(D + F) + 1.6(L_r \text{ or } S \text{ or } R) + 1.6H + (f_1L \text{ or } 0.5W)$ **(Equation 16-3)**

$1.2(D + F) + 1.0W + f_1L + 1.6H + 0.5(L_r \text{ or } S \text{ or } R)$ **(Equation 16-4)**

$1.2(D + F) + 1.0E + f_1L + 1.6H + f_2S$ **(Equation 16-5)**

$0.9D + 1.0W + 1.6H$ **(Equation 16-6)**

$0.9(D + F) + 1.0E + 1.6H$ **(Equation 16-7)**

where:

f_1 = 1 for places of public assembly live loads in excess of 100 pounds per square foot (4.79 kN/m^2), and parking garages; and 0.5 for other live loads.

f_2 = 0.7 for roof configurations (such as saw tooth) that do not shed snow off the structure, and 0.2 for other roof configurations.

Exceptions:

1. Where other factored load combinations are specifically required by other provisions of this code, such combinations shall take precedence.
2. Where the effect of H resists the primary variable load effect, a load factor of 0.9 shall be included with H where H is permanent and H shall be set to zero for all other conditions.

❖ This section lists the load combinations for strength design or load and resistance factor design methods. See the definitions in Section 1602 for an explanation of the load notations. The basis for these load combinations is Section 2.3.2 of ASCE 7. These combinations of factored loads are the agreed-upon strength limit states that establish the required strength to be provided in the structural component being designed. In spite of the precise appearance of the load and resistance factor design load combinations, one should keep in mind their probabilistic nature. The goal is to allow a wide variety of structures to be designed economically with an acceptably low probability that the strength of the structure will be exceeded. Doing so necessitates combining loads in scenarios that are likely to occur. Dead load is a permanent load and it appears in every combination. The load combinations are constructed by adding the dead load to one of the variable loads at its maximum value, which is typically indicated by the load factor of 1.6. In addition, other variable loads are included with load factors that are less than 1.0. Those so-called "companion loads" represent arbitrary point-in-time values for those loads. The exception is the maximum earthquake load that has a load factor of 1.0 (see the commentary to the definitions "Factored load" and "Nominal load" in Section 202).

1605.2.1 Other loads. Where flood loads, F_a, are to be considered in the design, the load combinations of Section 2.3.3 of ASCE 7 shall be used. Where self-straining loads, T, are considered in design, their structural effects in combination with other loads shall be determined in accordance with Section 2.3.5 of ASCE 7. Where an ice-sensitive structure is subjected to loads due to atmospheric icing, the load combinations of Section 2.3.4 of ASCE 7 shall be considered.

❖ Under a cooperative agreement with FEMA, American Society of Civil Engineers completed an extensive analysis of flood loads and flood load combinations. The results of this study were first included in the 1998 edition of ASCE 7. For all buildings and structures located in flood hazard areas, this section specifies that flood load and flood load combinations using the strength design method are to be determined in accordance with Section 2.3.3 of ASCE 7, which states:

> 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 + 2.0$_a$.
> 2. In noncoastal A Zones, 1.0 W in combinations (4) and (6) shall be replaced with 0.5 W + 1.0F_a.

1605.3 Load combinations using allowable stress design.

1605.3.1 Basic load combinations. Where *allowable stress design* (working stress design), as permitted by this code, is used, structures and portions thereof shall resist the most critical effects resulting from the following combinations of loads:

$D + F$ **(Equation 16-8)**

$D + H + F + L$ **(Equation 16-9)**

$D + H + F + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-10)**

$D + H + F + 0.75(L) + 0.75(L_r \text{ or } S \text{ or } R)$ **(Equation 16-11)**

$D + H + F + (0.6W \text{ or } 0.7E)$ **(Equation 16-12)**

$D + H + F + 0.75(0.6W) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$ **(Equation 16-13)**

$D + H + F + 0.75\ (0.7\ E) + 0.75\ L + 0.75\ S$ **(Equation 16-14)**

$0.6D + 0.6W+H$ **(Equation 16-15)**

$0.6(D + F) + 0.7E+H$ **(Equation 16-16)**

Exceptions:

1. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads of 30 psf (1.44 kN/m^2) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.
3. Where the effect of *H* resists the primary variable load effect, a load factor of 0.6 shall be included with *H* where *H* is permanent and *H* shall be set to zero for all other conditions.
4. In Equation 16-15, the wind load, *W*, is permitted to be reduced in accordance with Exception 2 of Section 2.4.1 of ASCE 7.
5. In Equation 16-16, 0.6 *D* is permitted to be increased to 0.9 *D* for the design of special reinforced masonry shear walls complying with Chapter 21.

❖ See Section 1602 for an explanation of the notations. These basic load combinations for allowable stress design are based on Section 2.4.1 of ASCE 7. Note that a 0.75 factor is applied where these combinations include more than one variable load. This reduces the combined effect of these variable loads in recognition of the lower probability that two or more variable loads will reach their maximum values simultaneously.

Previous editions of the model codes specified that the overturning moment and sliding due to wind load could not exceed two-thirds of the dead load stabilizing moment; however, it was not typically applied to all elements in the building. In the code this limitation on dead load is accomplished through the load combinations. The applicable combination is $0.6D + 0.6W + H$. This load combination limits the dead load resisting wind loads to 60 percent ($^2/_3 = 0.67$; round down) but it applies to all elements. In this form, it is required that this safety factor on dead load applies to all actions where dead load is resisting wind loads.

The load combination, $0.6\ (D + F) + 0.7E + H$, applies throughout the structure and provides for overall stability of the structure similar to the load combination for wind. In determining seismic load effect, *E*, the definition of effective seismic weight requires inclusion of a portion of roof snow under certain conditions. Frequently the question of whether it is logical to include a component of snow load in determining *E*, but not to use that same weight of snow in the load case to resist the overturning or uplift due to *E* is asked. This apparent inconsistency has been built into the seismic requirements for some time. It introduces a bit more conservatism into the earthquake loading where heavier snow loads are concerned.

Exception 1 for crane hook loads has been a long-standing allowance in the alternative basic load combinations of Section 1605.3.1 under the legacy model codes and is now permitted in the code. It allows special consideration in combining crane loads with wind, snow and roof live loads due to a lower probability that these maximum loads occur simultaneously. As noted above, these basic load combinations are based on ASCE 7 allowable stress load combinations, but there is no corresponding exception for crane loads under ASCE 7. In the development of the 2000 edition of this code, this exception was added to the basic load combinations in an attempt to provide "parity" with the alternative basic load combinations. It is worth noting that these two sets of load combinations come from two different sources as described above and, from an overall perspective, they never have provided parity. Crane loads are considered live loads (see Section 1607.13). Exception 1 has no effect on dead load, other floor live loads, rain loads or earthquake loads; thus, where crane live loads are to be considered, the exception eliminates the roof live load, L_r, altogether and reduces the snow load to $0.75S$ and wind load to $0.5(0.6W)$. This exception does not negate the need to combine live loads other than the crane live loads in the usual manner with wind and snow loads. In other words, the load combinations in Equations 16-11 and 16-13 must be investigated without the crane live load and then modified versions of these combinations that include crane live load (use L_0 to denote crane live load) would be considered to make use of the exception as follows:

Equation 16-11
$D + H + F + 0.75L + 0.75\ (L, \text{ or } S \text{ or } R)$

Equation 16-11c
$D + H + F + 0.75\ (L + L_0) + 0.75\ (0.75S \text{ or } R)$

Equation 16-13
$D + H + F + 0.75\ (0.6W) + 0.75L + 0.75\ (L, \text{ or } S \text{ or } R)$

Equation 16-13
$D + H + F + 0.75\ (0.5 \times 0.6W) + 0.75[L + L_0] + 0.75\ (0.75S \text{ or } R)$

Equation 16-14
$D + H + F + 0.75\ (0.7E) + 0.75L + 0.75S$

Equation 16-14
$D + H + F + 0.75\ (0.7E) + 0.75[L + L_0] + 0.75\ (0.75S)$

Exception 2 states that "flat roof snow loads of 30 psf (1.44 kN/m^2) or less need not be combined with seismic loads..." This is an exception to the requirement to combine the effects of snow and earthquake load as would otherwise be required by Equation 16-14.

1605.3.1.1 Stress increases. Increases in allowable stresses specified in the appropriate material chapter or the referenced standards shall not be used with the load combinations of Section 1605.3.1, except that increases shall be permitted in accordance with Chapter 23.

❖ Some code sections and some referenced standards permit increases in the allowable stress where load combinations include either wind or earthquake loads. Such increases are expressly prohibited with the basic allowable stress load combinations. This prohibition is due to the 0.75 reduction applied to multiple variable loads in these load combinations, which has replaced the concept of allowable stress increases. Several adjustment factors for the design of wood construction allow increases to reference design values, such as the load duration factor, size factor, flat use factor, repetitive member factor, buckling stiffness factor and bearing area factor. These wood adjustment factors are considered to be material dependent. Since they are not related to the presence of particular design loads, this section recognizes their use.

1605.3.1.2 Other loads. Where flood loads, F_a, are to be considered in design, the load combinations of Section 2.4.2 of ASCE 7 shall be used. Where self-straining loads, T, are considered in design, their structural effects in combination with other loads shall be determined in accordance with Section 2.4.4 of ASCE 7. Where an ice-sensitive structure is subjected to loads due to atmospheric icing, the load combinations of Section 2.4.3 of ASCE 7 shall be considered.

❖ The study discussed in the commentary to Section 1605.2.1 determined flood load and flood load combination factors for use with the allowable stress method. This section specifies that flood load and flood load combinations using the allowable stress design method are to be determined in accordance with Section 2.4.2 of ASCE 7, which states:

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 (see Section 5.3.1), 1.5F_a shall be added to other load combinations (5), (6) and (7), and E shall be set equal to zero in (5) and (6).

2. In noncoastal A zones, 0.75F_a, shall be added to the combinations (5), (6) and (7), and E shall be set at zero in (5) and (6).

1605.3.2 Alternative basic load combinations. In lieu of the basic load combinations specified in Section 1605.3.1, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. When using these alternative basic load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced where permitted by the material chapter of this code or the referenced standards. For load combinations that include the counteracting effects of dead and wind loads, only two-thirds of the minimum dead load likely to be in place during a design wind event shall be used. When using allowable stresses which have been increased or load combinations which have been reduced as permitted by the material chapter of this code or the referenced standards, where wind loads are calculated in accordance with Chapters 26 through 31 of ASCE 7, the coefficient (ω) in the following equations shall be taken as 1.3. For other wind loads, (ω) shall be taken as 1. When allowable stresses have not been increased or load combinations have not been reduced as permitted by the material chapter of this code or the referenced standards, (ω) shall be taken as 1. When using these alternative load combinations to evaluate sliding, overturning and soil bearing at the soil-structure interface, the reduction of foundation overturning from Section 12.13.4 in ASCE 7 shall not be used. When using these alternative basic load combinations for proportioning foundations for loadings, which include seismic loads, the vertical seismic *load effect*, E_v, in Equation 12.4-4 of ASCE 7 is permitted to be taken equal to zero.

$D + L + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-17)**

$D + L + 0.6\ \omega W$ **(Equation 16-18)**

$D + L + 0.6\ \omega W + S/2$ **(Equation 16-19)**

$D + L + S + 0.6\ \omega W/2$ **(Equation 16-20)**

$D + L + S + E/1.4$ **(Equation 16-21)**

$0.9D + E/1.4$ **(Equation 16-22)**

Exceptions:

1. Crane hook loads need not be combined with roof live loads or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads of 30 psf (1.44 kN/m^2) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.

❖ These are alternative load combinations to those in Section 1605.3.1 for use with allowable stress design. They are based on the allowable stress load combinations in the *Uniform Building Code*® (UBC™).

Note that the dead load is limited to two-thirds of the dead load likely to be in place during a design wind event. Section 1604.9 requires that all elements of a building be anchored to resist overturning, uplift and sliding. Often, a considerable portion of this resistance is provided by the dead load of a building, including the weight of foundations and any soil directly above them. This section requires the designer to give consideration to the dead load used to resist wind loads by stating that only the minimum dead load likely to be in place during a design wind event is permitted to be used. This, however, does not imply that certain parts or elements of a building are not designed to remain in place. This criteria simply cautions the designer against using dead loads that may not be installed, or in place, such as can occur where a design includes an allowance for a planned future expansion.

Exception 1 for crane hook loads has been a long-standing allowance under the legacy model codes that were also the source of these alternative basic load combinations. It allows special consideration in combining crane live loads with wind, snow and roof live loads due to a lower probability that these maximum loads occur simultaneously.

Exception 2 states that "flat roof snow loads of 30 psf (1.44 N/m^2) or less need not be combined with seismic loads...". This is an exception to the requirement to combine the effects of snow and earthquake load as would otherwise be required by Equation 16-21.

Also see the commentary to Section 1602.1 for discussion of earthquake load effect, *E*, as well as other loads required by these load combinations.

1605.3.2.1 Other loads. Where *F*, *H* or *T* are to be considered in the design, each applicable load shall be added to the combinations specified in Section 1605.3.2. Where self-straining loads, *T*, are considered in design, their structural effects in combination with other loads shall be determined in accordance with Section 2.4.4 of ASCE 7.

❖ See Section 1602 for an explanation of the notation used in this section. As indicated, the applicable loads are to be added to each of the load combinations in Section 1605.3.2.

SECTION 1606
DEAD LOADS

1606.1 General. Dead loads are those loads defined in Section 1602.1. Dead loads shall be considered permanent loads.

❖ The nominal dead load, *D*, is determined in accordance with Section 1606. Similar to the general section on live loads (see Section 1607.1), this section provides a reminder that the term "dead loads" is defined, establishing exactly what should be considered a dead load. It also states that dead loads are considered permanent. This affects how dead loads are classified (permanent versus variable), which is a necessary distinction to make when applying the provisions for load combinations.

1606.2 Design dead load. For purposes of design, the actual weights of materials of construction and fixed service equipment shall be used. In the absence of definite information, values used shall be subject to the approval of the *building official*.

❖ When determining the design dead loads, the actual weights of all materials, equipment, construction, etc., are to be used for the structural design. Often the exact type of materials and equipment to be installed has not yet been determined by the design team at the time the structural design is initiated. Where the actual weights are not known, it is common to use an estimate of the anticipated materials and equipment. While the actual dead load itself does not vary, the estimate of the dead load that is used in structural computations can vary. Such estimates of dead loads are typically greater than the actual dead loads so that the computations are conservative and a redesign of the structure will not be necessary when the actual weights become known. This section clarifies that the building official must approve these estimated dead loads.

Overestimating the actual dead load, so that the design computations are conservative, is acceptable when considering load combinations in Section 1605 that are additive. But the same cannot be said when using counteracting load combinations (see commentary, Section 1604.9). In determining the anchorage required to resist the overturning or uplift effects of wind or seismic loads, resistance is typically provided by the dead load. For example, uplift forces due to wind cause tension to develop in hold-down connections. If the dead load used to resist the wind uplift is overestimated, the result may be an unconservative design. Note that Section 1605.3.2 restricts the dead load used to counteract the effects of overturning and uplift to be the minimum dead load likely to be in place during a design wind event.

As a design guideline, see the unit weights of common construction materials and assemblies in Tables C3-1 and C3-2 of the commentary to ASCE 7. The unit dead loads listed in the tables for assembled elements are usually given in units of pounds per square foot (psf) of surface area (i.e., floor areas, wall areas, ceiling areas, etc.). Unit dead loads for materials used in construction are given in terms of density. The unit weights given in the tables are generally single values, even though a range of weights may actually exist. The average unit weights given are generally suitable for design purposes; however, where there is reason to believe that the actual weights of assembled elements or construction materials may substantially exceed the tabular values, then the situation should be investigated and the highest values used.

SECTION 1607
LIVE LOADS

1607.1 General. Live loads are those loads defined in Section 1602.1.

❖ Nominal live loads are determined in accordance with Section 1607. The live load requirements for the design of buildings and structures are based on the type of occupancy. Live loads are transient loads that vary with time. Generally, the design live load is that which is believed to be near the maximum transient load for a given occupancy.

TABLE 1607.1. See page 16-20.

❖ The design values of live loads for both uniform and concentrated loads are shown in the table as a function of occupancy. The values given are conservative and include both the sustained and variable portions of the live load. Section 1607.3 directs the designer to utilize the greater live loads produced by the intended occupancy, but not less than the minimum uniformly distributed live loads listed in Table 1607.1. It should also be noted that the "occupancy" category listed is not necessarily group specific. For example, an office building may be classified as Group B, but still contain incidental storage areas. Depending on the type of storage, the areas may warrant storage live loads of either 125 or 250 psf (5.98 or 11.9 kN/m^2) to be applied to the space in question.

Table 1607.1 specifies minimum uniform live load in residential attics for three distinct conditions: uninhabitable attics without storage; uninhabitable attics with limited storage; and habitable attics and sleeping areas. Commentary Table 1607.1(1) summarizes the uniform live loads that apply to uninhabitable attics based on the criteria contained in Notes i, j and k to Table 1607.1. The process for determining which load is applicable is summarized in the flow chart in Figure 1607.1.

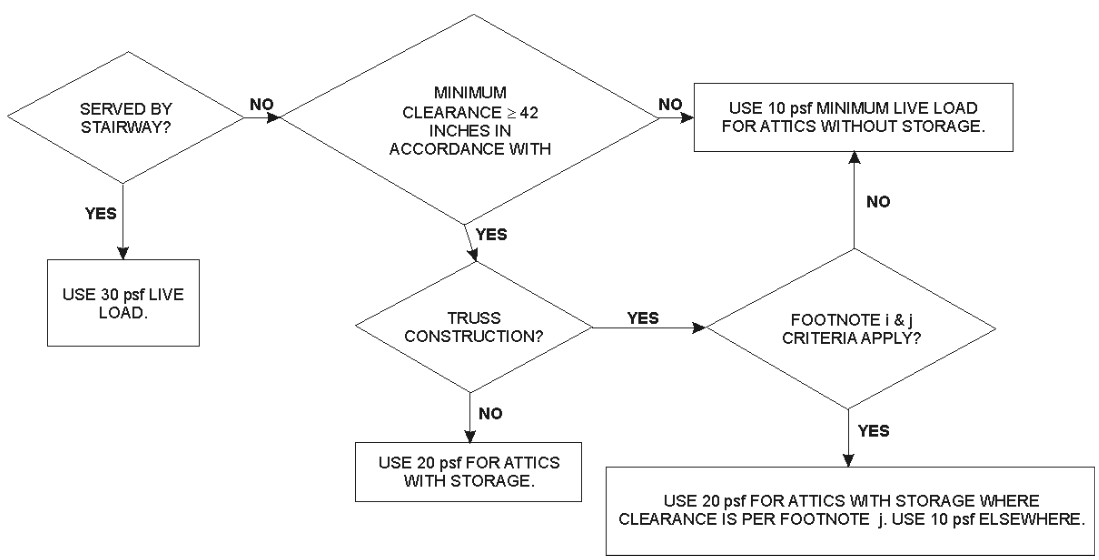
One noteworthy distinction made by Note k is that any uninhabitable attic that is served by a stairway (other than a pull-down type) must be designed using the load of 30 psf (1.44 kN/m^2) that is applicable to habitable attics (see Note k). The implication is that the presence of a stairway is more conducive to using the attic for storage and therefore warrants a greater design live load. This recognizes that the stairway is likely to serve an attic with greater headroom, providing more storage capacity. For an attic that is accessed by other means, such as a framed opening or pull-down stairs, it is necessary to determine whether the attic storage load applies in accordance with the criteria contained in Notes i and j.

Historically, a minimum load of 10 psf (0.48 kN/m^2) has been viewed as appropriate where occasional access to the attic is anticipated for maintenance purposes, but significant storage is restricted by physical constraints, such as low clearance or the configuration of truss webs. It provides a minimum degree of structural integrity, allowing for occasional access to an attic space for maintenance purposes. Allowing the application of this load to be independent of other live loads is deemed appropriate, since it would be rare for this load and other maximum live loads to occur at once.

Note m clarifies that a live load reduction is not permitted unless specific exceptions of Section 1607.10 apply. The note appears at each specific use or occupancy in Table 1607.1 where a live load reduction is restricted, which serves to clarify the limitations on live load reduction. References appear in Sections 1607.10.1 and 1607.10.2 to correlate with the note.

Table 1607.1(1)
SUMMARY OF MINIMUM LIVE LOAD REQUIRED IN UNINHABITABLE ATTICS

Description	Uniform Load
Without storage	10 psf
With storage	20 psf
Served by a stairway	30 psf



For SI: 1 inch = 25.4 mm, 1 pound per square foot = 47.88 Pa.

Figure 1607.1
DETERMINATION OF ATTIC LIVE LOAD IN ACCORDANCE WITH TABLE 1607.1 FOOTNOTES i, j and k

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
1. Apartments (see residential)	—	—
2. Access floor systems Office use Computer use	 50 100	 2,000 2,000
3. Armories and drill rooms	150 [m]	—
4. Assembly areas Fixed seats (fastened to floor) Follow spot, projections and control rooms Lobbies Movable seats Stage floors Platforms (assembly) Other assembly areas	 60 [m] 50 100 [m] 100 [m] 150 [m] 100 [m] 100 [m]	—
5. Balconies and decks[h]	Same as occupancy served	—
6. Catwalks	40	300
7. Cornices	60	—
8. Corridors First floor Other floors	 100 Same as occupancy served except as indicated	—
9. Dining rooms and restaurants	100 [m]	—
10. Dwellings (see residential)	—	—
11. Elevator machine room grating (on area of 2 inches by 2 inches)	—	300
12. Finish light floor plate construction (on area of 1 inch by 1 inch)	—	200
13. Fire escapes On single-family dwellings only	100 40	—
14. Garages (passenger vehicles only) Trucks and buses	40 [m] See Section 1607.7	Note a
15. Handrails, guards and grab bars	See Section 1607.8	
16. Helipads	See Section 1607.6	
17. Hospitals Corridors above first floor Operating rooms, laboratories Patient rooms	 80 60 40	 1,000 1,000 1,000
18. Hotels (see residential)	—	—
19. Libraries Corridors above first floor Reading rooms Stack rooms	 80 60 150[b, m]	 1,000 1,000 1,000
20. Manufacturing Heavy Light	 250 [m] 125 [m]	 3,000 2,000
21. Marquees	75	—
22. Office buildings Corridors above first floor File and computer rooms shall be designed for heavier loads based on anticipated occupancy Lobbies and first-floor corridors Offices	 80 — 100 50	 2,000 — 2,000 2,000

(continued)

TABLE 1607.1—continued
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
23. Penal institutions Cell blocks Corridors	 40 100	—
24. Recreational uses: Bowling alleys, poolrooms and similar uses Dance halls and ballrooms Gymnasiums Reviewing stands, grandstands and bleachers Stadiums and arenas with fixed seats (fastened to floor)	 75[m] 100[m] 100[m] 100[c, m] 60[c, m]	—
25. Residential One- and two-family dwellings Uninhabitable attics without storage[i] Uninhabitable attics with storage [i, j, k] Habitable attics and sleeping areas[k] All other areas Hotels and multifamily dwellings Private rooms and corridors serving them Public rooms[m] and corridors serving them	 10 20 30 40 40 100	—
26. Roofs All roof surfaces subject to maintenance workers Awnings and canopies: Fabric construction supported by a skeleton structure All other construction Ordinary flat, pitched, and curved roofs (that are not occupiable) Where primary roof members are exposed to a work floor, at 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 All other primary roof members Occupiable roofs: Roof gardens Assembly areas All other similar areas	 5 nonreducible 20 20 100 100[m] Note 1	 300 2,000 300 Note 1
27. Schools Classrooms Corridors above first floor First-floor corridors	 40 80 100	 1,000 1,000 1,000
28. Scuttles, skylight ribs and accessible ceilings	—	200
29. Sidewalks, vehicular drive ways and yards, subject to trucking	250[d, m]	8,000[e]

(continued)

TABLE 1607.1—continued
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
30. Stairs and exits One- and two-family dwellings All other	 40 100	 300 [f] 300 [f]
31. Storage warehouses (shall be designed for heavier loads if required for anticipated storage) Heavy Light	 250[m] 125[m]	—
32. Stores Retail First floor Upper floors Wholesale, all floors	 100 75 125[m]	 1,000 1,000 1,000
33. Vehicle barriers	See Section 1607.8.3	
34. Walkways and elevated platforms (other than exitways)	60	—
35. Yards and terraces, pedestrians	100[m]	—

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm^2,
1 square foot = 0.0929 m^2,
1 pound per square foot = 0.0479 kN/m^2, 1 pound = 0.004448 kN,
1 pound per cubic foot = 16 kg/m^3.

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 1607.1 or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of 4.5 inches by 4.5 inches; (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 pounds per wheel.

b. The loading applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:
1. The nominal bookstack unit height shall not exceed 90 inches;
2. The nominal shelf depth shall not exceed 12 inches for each face; and
3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches wide.

c. Design in accordance with ICC 300.

d. Other uniform loads in accordance with an approved method containing provisions for truck loadings shall also be considered where appropriate.

e. The concentrated wheel load shall be applied on an area of 4.5 inches by 4.5 inches.

f. The minimum concentrated load on stair treads shall be applied on an area of 2 inches by 2 inches. This load need not be assumed to act concurrently with the uniform load.

g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official (see Section 1608).

h. See Section 1604.8.3 for decks attached to exterior walls.

i. Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches 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 requirements.

(continued)

TABLE 1607.1—continued
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS[g]

j. Uninhabitable attics with storage are those where the maximum clear height between the joists and rafters is 42 inches or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses.

The live load need only be applied to those portions of the joists or truss bottom chords where both of the following conditions are met:
i. The attic area is accessible from an opening not less than 20 inches in width by 30 inches in length that is located where the clear height in the attic is a minimum of 30 inches; and
ii. The slopes of the joists or truss bottom chords are no greater than two units vertical in 12 units horizontal.

The remaining portions of the joists or truss bottom chords shall be designed for a uniformly distributed concurrent live load of not less than 10 lb./ft^2.

k. Attic spaces served by stairways other than the pull-down type shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.

l. Areas of occupiable roofs, other than roof gardens and assembly areas, shall be designed for appropriate loads as approved by the building official. Unoccupied landscaped areas of roofs shall be designed in accordance with Section 1607.12.3.

m. Live load reduction is not permitted unless specific exceptions of Section 1607.10 apply.

1607.2 Loads not specified. For occupancies or uses not designated in Table 1607.1, the live load shall be determined in accordance with a method *approved* by the *building official*.

❖ Whenever an occupancy or use of a structure cannot be identified with the listing shown in Table 1607.1, then the live load values used for design are required to be determined by the design professional and subject to the approval of the building official. Aside from the obvious intent of this requirement, however, which is to prescribe a minimum design load value, some caution needs to be exercised by the design professional in determining the appropriate design live load value. For example, the table shows that heavy storage areas must be designed for a uniform live load of 250 psf (11.9 kN/m^2). This is a minimum value. Storage warehouses or storage areas within manufacturing facilities containing items, such as automobile parts, electrical goods, coiled steel, plumbing supplies and bulk building materials, generally have live loads ranging between 300 and 400 psf (14.4 and 19 kN/m^2). Similarly, storage facilities containing dry goods, paints, oil, groceries or liquor often have loadings that range between 200 to 300 psf (9.6 to 14.4 kN/m^2). Another example is a heavy manufacturing facility that makes generators for the electric power industry. Some of the production areas in this type of facility require structural floors that support loads of 1,000 psf (47.9 kN/m^2) or more, which is about seven times the live load specified in Table 1607.1.

1607.3 Uniform 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 live loads given in Table 1607.1.

❖ Studies have shown that building live loads consist of a sustained portion based on the day-to-day use of the facilities, and a variable portion created by unusual events, such as remodeling, temporary storage of materials, the extraordinary assemblage of people for an occasional business meeting or social function (i.e., holiday party) and similar events. The sustained portion of the live load will likely vary during the life of a building because of tenant changes, rearrangement of office space and furnishings, changes in the nature of the occupancy (i.e., number of people or type of business), traffic patterns and so on. In light of this variability of loadings that are apt to be imposed on a building, the code provisions simplify the design procedure by expressing the applicable load as either a uniformly distributed live load or a concentrated live load on the floor area. It should be pointed out that this section does not require the concurrent application of uniform live load and concentrated live load. In other words, this section requires that either the uniform load or the concentrated load be applied, so long as the type of load that produces the greater stress in the structural element under consideration is utilized.

1607.4 Concentrated live loads. Floors and other similar surfaces shall be designed to support the uniformly distributed live loads prescribed in Section 1607.3 or the concentrated live loads, in pounds (kiloNewtons), given in Table 1607.1, whichever produces the greater *load effects*. Unless otherwise specified, the indicated concentration shall be assumed to be uniformly distributed over an area of $2^1/_2$ feet by $2^1/_2$ feet (762 mm by 762 mm) and shall be located so as to produce the maximum *load effects* in the structural members.

❖ A building or portion thereof is subjected to concentrated floor loads commensurate with the use of the facility. For example, in Group B, a law office may have stacks of books and files that impose large concentrations of loads on the supporting structural elements. An industrial facility may have a tank full of liquid material on a mezzanine that feeds a machine on the floor below. The structural floor of a stockroom may support heavy bins containing metal parts and so on. The exact locations or nature of such concentrated loadings is not usually known at the time of the design of the building. Furthermore, new sources of concentrated loadings will be added during the life of the structure, while some or all of the existing sources will be relocated; therefore, because of the uncertainties of the sources of concentrated loads, as well as their weights and locations, the code provides typical loads to be used in the design of structural floors consistent with the type of use of the facility. The minimum concentrated loads to be used for design are contained in Table 1607.1. Concentrated loads are not required to be applied simultaneously, with the uniform live loads also specified in Table 1607.1. Concentrated loads are to be applied as an independent load condition at the location on the floor that produces the greatest stress in the structural members being designed. The single concentrated load is to be placed at any location on the floor. For example, in an office use area, the floor system is to be designed for either a 2,000-pound (8897 N) concentrated load (unless the anticipated actual concentrated load is higher) applied at any location in the office area, or the 50 psf (2.40 kN/m^2) live load specified in Table 1607.1, whichever results in the greater stress in the supporting structural member.

1607.5 Partition loads. In office buildings and in other buildings where partition locations are subject to change, provisions for partition weight shall be made, whether or not partitions are shown on the *construction documents*, unless the specified live load exceeds 80 psf (3.83 kN/m^2). The partition load shall not be less than a uniformly distributed live load of 15 psf (0.72 kN/m^2).

❖ Provision for the weight of partitions must be made in the structural design. The weight of any built-in partitions should be considered a dead load in accordance with the definition in Section 202. Buildings where partitions are readily relocated must include a live load of 15 psf (0.74 kN/m^2) if the uniform floor live load is 80 psf (3.83 kN/m^2) or less. This partition allowance is included under live loads because of its variable nature.

1607.6 Helipads. Helipads shall be designed for the following live loads:

1. A uniform live load, *L*, as specified below. This load shall not be reduced.
   1.1. 40 psf (1.92 kN/m^2) where the design basis helicopter has a maximum take-off weight of 3,000 pounds (13.35 kN) or less.
   1.2. 60 psf (2.87 kN/m^2) where the design basis helicopter has a maximum take-off weight greater than 3,000 pounds (13.35 kN).
2. A single concentrated live load, *L*, of 3,000 pounds (13.35 kN) applied over an area of 4.5 inches by 4.5 inches (114 mm by 114 mm) and located so as to produce the maximum load effects on the structural elements under consideration. The concentrated load is not required to act concurrently with other uniform or concentrated live loads.
3. Two single concentrated live loads, *L*, 8 feet (2438 mm) apart applied on the landing pad (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 so as to produce the maximum load effects on the structural elements under consideration. The concentrated loads shall be applied over an area of 8 inches by 8 inches (203 mm by 203 mm) and are not required to act concurrently with other uniform or concentrated live loads.

Landing areas designed for a design basis helicopter with maximum take-off weight of 3,000 pounds (13.35 kN) shall be identified with a 3,000 pound (13.34 kN) weight limitation. The landing area weight limitation shall be indicated by the numeral "3" (kips) located in the bottom right corner of the landing area as viewed from the primary approach path. The indication for the landing area weight limitation shall be a minimum 5 feet (1524 mm) in height.

❖ Detailed requirements for helistops and heliports, including definitions, can be found in Section 412 of the code. For example Section 412.7.1 requires a minimum landing area size of 20 feet by 20 feet (6096 mm by 6096 mm) for helicopters weighing less than 3,500 pounds (15.57 kN). In addition, Section 2007 of the *International Fire Code*® (IFC®) regulates helistops and heliports. Section 2007.8 of the IFC requires Federal Aviation Administration (FAA) approval of these facilities.

The term "helipad" is used to describe a helicopter landing area, which is the subject of this provision. These structural design requirements establish the minimum live load criteria that are specific to the design of the landing area and the supporting structural elements.

Items 1 and 2 specify the uniform live load based on the threshold weight of 3,000 pounds (13.34 kN). The majority of helicopters used in general aviation have a gross weight of 3,000 pounds (13.34 kN) or less. With weights comparable to those of passenger vehicles and considering the size of the landing area, the equivalent uniform load on these landing areas is actually lower than the minimum uniform live load required for a passenger vehicle parking garage. Thus, a reduced design live load is permitted for the design of such helipads. Since marking the weight limitation of the landing area is standard practice, as well as an FAA recommendation, indicating the weight limitation on the landing area was included as a condition for using the uniform load associated with helicopters weighing up to 3,000 pounds (13.34 kN).

1607.7 Heavy vehicle loads. Floors and other surfaces that are intended to support vehicle loads greater than a 10,000 pound (4536 kg) gross vehicle weight rating shall comply with Sections 1607.7.1 through 1607.7.5.

❖ Sections 1607.7.1 through 1607.7.5 give criteria for addressing heavy vehicle loads, including fire trucks and forklifts.

1607.7.1 Loads. Where any structure does not restrict access for vehicles that exceed a 10,000-pound (4536 kg) gross vehicle weight rating, those portions of the structure subject to such loads shall be designed using the vehicular live loads, including consideration of impact and fatigue, in accordance with the codes and specifications required by the jurisdiction having authority for the design and construction of the roadways and bridges in the same location of the structure.

❖ Where heavy highway-type vehicles have access onto a structure, this section requires the structure to be designed using the same requirements that are applicable to roadways in that jurisdiction. This loading may in fact be the loading from American Association of State Highway and Transportation Officials (AASHTO), or the loading for other elements such as lids of large detention tanks or utility vaults. The registered design professional should consult with the jurisdiction for design loads for these special conditions.

1607.7.2 Fire truck and emergency vehicles. Where a structure or portions of a structure are accessed and loaded by fire department access vehicles and other similar emergency vehicles, the structure shall be designed for the greater of the following loads:

1. The actual operational loads, including outrigger reactions and contact areas of the vehicles as stipulated and approved by the building official; or
2. The live loading specified in Section 1607.7.1.

❖ This establishes two criteria for addressing heavy vehicle loads due to fire trucks and other similar emergency vehicles.

1607.7.3 Heavy vehicle garages. Garages designed to accommodate vehicles that exceed a 10,000 pound (4536 kg) gross vehicle weight rating, shall be designed using the live loading specified by Section 1607.7.1. For garages the design for impact and fatigue is not required.

Exception: The vehicular live loads and load placement are allowed to be determined using the actual vehicle weights for the vehicles allowed onto the garage floors, provided such loads and placement are based on rational engineering principles and are approved by the building official, but shall not be less than 50 psf (2.9 kN/m^2). This live load shall not be reduced.

❖ This section helps clarify that the passenger vehicle garage loads are not applicable to garages that accommodate heavier vehicles.

1607.7.4 Forklifts and movable equipment. Where a structure is intended to have forklifts or other movable equipment present, the structure shall be designed for the total vehicle or equipment load and the individual wheel loads for the anticipated vehicles as specified by the owner of the facility. These loads shall be posted per Section 1607.7.5.

❖ Heavy vehicle loads due to forklifts and other moveable equipment require that a structure be designed for the total vehicle or equipment load as well as the individual wheel loads. The owner of the facility needs to make the planned usage of such a vehicle clear to the design team. As a precaution these loads must be posted (see Section 1607.7.5).

1607.7.4.1 Impact and fatigue. Impact loads and fatigue loading shall be considered in the design of the supporting structure. For the purposes of design, the vehicle and wheel loads shall be increased by 30 percent to account for impact.

❖ This section clarifies that consideration of impact and fatigue loading is a design requirement for structures subjected to heavy moving equipment, such as forklifts.

1607.7.5 Posting. The maximum weight of the vehicles allowed into or on a garage or other structure shall be posted by the owner in accordance with Section 106.1.

❖ As a precaution against overloading a structure, the maximum weight of the vehicles that are anticipated and used in the design should be posted by the owner (see Section 106.1).

1607.8 Loads on handrails, guards, grab bars, seats and vehicle barriers. Handrails, *guards*, grab bars, accessible seats, accessible benches and vehicle barriers shall be designed and constructed to the structural loading conditions set forth in this section.

❖ The requirements of this section are intended to provide an adequate degree of structural strength and stability to handrails, guards, grab bars and vehicle barriers.

1607.8.1 Handrails and guards. Handrails and *guards* shall be designed to resist a linear load of 50 pounds per linear foot (plf) (0.73 kN/m) in accordance with Section 4.5.1 of ASCE 7. Glass handrail assemblies and *guards* shall also comply with Section 2407.

Exceptions:

1. For one- and two-family dwellings, only the single concentrated load required by Section 1607.8.1.1 shall be applied.
2. In Group I-3, F, H and S occupancies, for areas that are not accessible to the general public and that have an *occupant load* less than 50, the minimum load shall be 20 pounds per foot (0.29 kN/m).

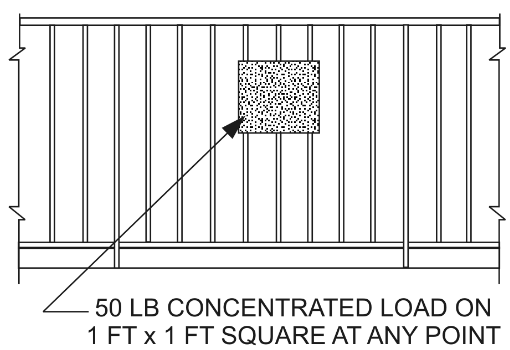
❖ The loading in this section represents the maximum anticipated load on a handrail or guard due to a crowd of people on the adjacent walking surface. The exceptions allow lower loads for handrails in locations that are not typically open to the public. These loads, depicted in Figure 1607.8.1, are permitted to be applied independent of other loads.

1607.8.1.1 Concentrated load. Handrails and guards shall also be designed to resist a concentrated load of 200 pounds (0.89 kN) in accordance with Section 4.5.1 of ASCE 7.

❖ The concentrated loading in this section is not to be applied with any other design load; it is a separate load case. The load simulates the maximum anticipated load from a person grabbing or falling into the handrail or guard.

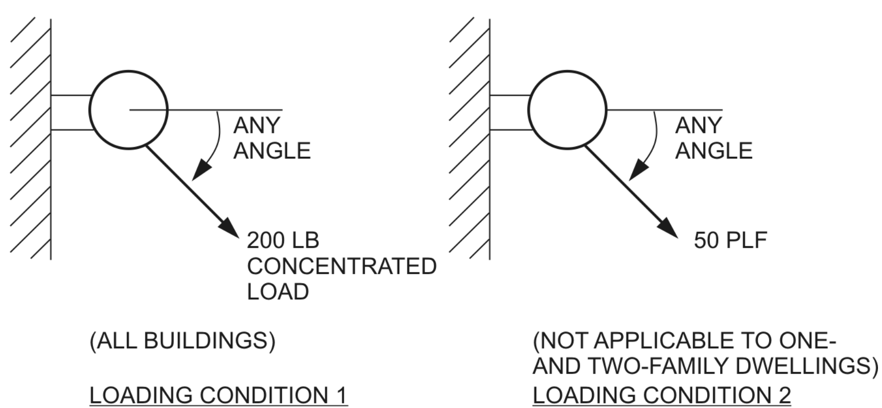
1607.8.1.2 Intermediate rails. Intermediate rails (all those except the handrail), balusters and panel fillers shall be designed to resist a concentrated load of 50 pounds (0.22 kN) in accordance with Section 4.5.1 of ASCE 7.

❖ This is a localized design load for the guard members and is not to be applied with any other loads. It is to be applied horizontally at a 90-degree (1.57 rad) angle with the guard members. The number of balusters that would resist this load are those within the 1 square foot (0.093 m^2) area in the plane of the guard as shown in Figure 1607.8.1.2.



For SI: 1 pound per square foot = 47.88 Pa.

Figure 1607.8.1.2
COMPONENT DESIGN LOAD



For SI: 1 pound = 0.454 kg, 1 pound per foot = 14.59 N/m.

Figure 1607.8.1
HANDRAIL DESIGN LOAD

1607.8.2 Grab bars, shower seats and dressing room bench seats. Grab bars, shower seats and dressing room bench seat systems shall be designed to resist a single concentrated load of 250 pounds (1.11 kN) applied in any direction at any point on the grab bar or seat so as to produce the maximum load effects.

❖ These live loads provide for the normal anticipated loads from the use of the grab bars, shower seats and dressing room bench seats. These structural requirements provide consistency with Americans with Disabilities Act (ADA) *Accessibility Guidelines* (ADAAG).

1607.8.3 Vehicle barriers. Vehicle barriers for passenger vehicles shall be designed to resist a concentrated load of 6,000 pounds (26.70 kN) in accordance with Section 4.5.3 of ASCE 7. Garages accommodating trucks and buses shall be designed in accordance with an *approved* method that contains provisions for traffic railings.

❖ Vehicle barriers provide a passive restraint system in locations where vehicles could fall to a lower level (see definition of "Vehicle barrier," Section 202). Figure 1607.8.3 depicts criteria for the design of passenger car and light truck vehicle barriers. The 6,000-pound (26.70 kN) load considers impact. The load is applied at a height, h, that is representative of vehicle bumper heights. Due to the variety of barrier configurations and anchorage methods, it is necessary to consider any height within the specified size in order to determine the most critical load effects for design of the barrier. For bus and heavy truck vehicle barrier design criteria, a state's Department of Transportation should be contacted (also see Section 1607.7).

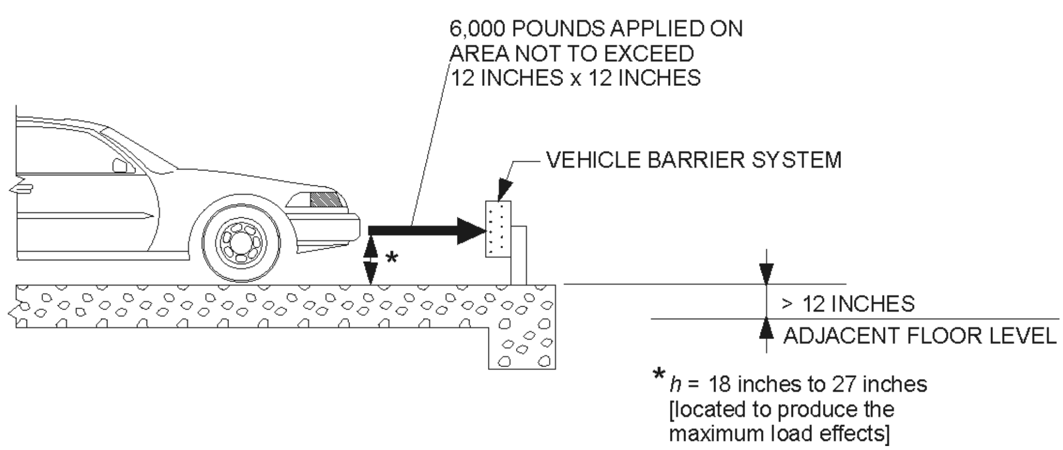
1607.9 Impact loads. The live loads specified in Sections 1607.3 through 1607.8 shall be assumed to include adequate allowance for ordinary impact conditions. Provisions shall be made in the structural design for uses and loads that involve unusual vibration and impact forces.

❖ In cases where "unusual" live loads occur in a building that impose impact or vibratory forces on structural elements (i.e., elevators, machinery, craneways, etc.), additional stresses and deflections are imposed on the structural system. Where unusual vibration (dynamic) and impact loads are likely to occur, the code requires that the structural design take these effects into account. Typically, the dynamic effects are approximated through the application of an equivalent static load equal to the dynamic load effects. In most cases, an equivalent static load is sufficient. A dynamic analysis is usually not required.

1607.9.1 Elevators. Members, elements and components subject to dynamic loads from elevators shall be designed for impact loads and deflection limits prescribed by ASME A17.1.

❖ The static load of an elevator must be increased to account for the effect of the elevator's motion. For example, when an elevator comes to a stop, the load on the elevator's supports is significantly higher than the weight of the elevator and the occupants. This effect varies with the acceleration and deceleration rate of the elevator. This section clarifies that the impact load from elevators applies specifically to members, elements and components subject to dynamic loading from the elevator mechanism and directs the code user to the elevator standard to determine the increases.

1607.9.2 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-



For SI: 1 inch = 25.4 mm, 1 pound = 4.448 N.

Figure 1607.8.3
VEHICLE BARRIER REQUIREMENTS

driven units, 50 percent. Percentages shall be increased where specified by the manufacturer.

❖ The specified increases for machinery loads include the vibration of the equipment, which increases the effective load. The load increase for reciprocating machinery versus rotating shaft-driven machinery is to account for the higher vibration.

1607.10 Reduction in uniform live loads. Except for uniform live loads at roofs, all other minimum uniformly distributed live loads, L_o, in Table 1607.1 are permitted to be reduced in accordance with Section 1607.10.1 or 1607.10.2. Uniform live loads at roofs are permitted to be reduced in accordance with Section 1607.12.2.

❖ Small floor areas are more likely to be subjected to the full uniform load than larger floor areas. Unloaded or lightly loaded areas tend to reduce the total load on the structural members supporting those floors. The specified uniformly distributed live loads from Table 1607.1 are permitted to be reduced, with some notable exceptions or limitations, in recognition that the larger the tributary area of a structural member, the lower the likelihood that the full live load will be realized. The basis for the live load reduction in Sections 1607.10.1 through 1607.10.1.3 is ASCE 7. An alternative method of live load reduction, retained from legacy model codes, is provided in Section 1607.10.2.

1607.10.1 Basic uniform live load reduction. Subject to the limitations of Sections 1607.10.1.1 through 1607.10.1.3 and Table 1607.1, members for which a value of $K_{LL}A_T$ is 400 square feet (37.16 m^2) or more are permitted to be designed for a reduced uniformly distributed live load, L, in accordance with the following equation:

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \qquad \textbf{(Equation 16-23)}$$

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

where:

L = Reduced design live load per square foot (m^2) of area supported by the member.

L_o = Unreduced design live load per square foot (m^2) of area supported by the member (see Table 1607.1).

K_{LL} = Live load element factor (see Table 1607.10.1).

A_T = Tributary area, in square feet (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.

❖ This section provides a method of reducing uniform floor live loads that is based on the provisions of ASCE 7. The concept is that where the design live load is governed by the minimum live loads in Table 1607.1, the actual load on a large area of the floor is very likely to be less than the nominal live load in the table. Thus, the allowable reduction increases with the tributary area of the floor that is supported by a structural member; therefore, a girder that supports a large tributary area would be allowed to be designed for somewhat lower uniform live load than a floor beam that supports a smaller floor area.

The following example demonstrates the live load reduction calculation for the conditions shown in Figure 1607.10.1:

Solution:

For interior beam K_a = 2 (Table 1607.10.1)

K, A_T = (2)(750 sq. ft.) = 1500 sq. ft. (139 m^2)

Using Equation 16-23

$$L = 50\left(0.25 + \frac{15}{\sqrt{1500}}\right) = 35\text{psf } (1.68 \text{ kN/m}^2)$$

0.5 L_o = 25 psf < 32 psf (1.53 kN/m^2)

Use reduced live load, L = 32 psf (1.53 kN/m^2)

TABLE 1607.10.1
LIVE LOAD ELEMENT FACTOR, K_{LL}

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

❖ The purpose of this table is to provide tributary area adjustment factors, K_{LL}, for determining live load reductions in Section 1607.10.1. The factor converts the tributary area of the structural member to an "influence area." This "influence area" of a structural member is considered to be the adjacent floor area from which it derives any of its load. These adjustments to the tributary area range from 1 through 4 based on the type of structural element being designed and are meant to reflect the element's ability to share load with adjacent elements.

1607.10.1.1 One-way slabs. The tributary area, A_T, for use in Equation 16-23 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.

❖ This section limits the tributary area that can be utilized to determine a live load reduction for one-way slabs.

1607.10.1.2 Heavy live loads. Live loads that exceed 100 psf (4.79 kN/m^2) shall not be reduced.

Exceptions:

1. The live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than *L* as calculated in Section 1607.10.1.
2. For uses other than storage, where *approved*, additional live load reductions shall be permitted where shown by the *registered design professional* that a rational approach has been used and that such reductions are warranted.

❖ The purpose of this section is to prohibit live load reductions where the live loads exceed 100 psf (4.79 kN/m^2). Such live loads are typically intended for storage or related purposes. It is more likely that the full live load will be realized at a given floor level for such occupancies. Thus, reduced live loads are not allowed for these conditions except as described in this section.

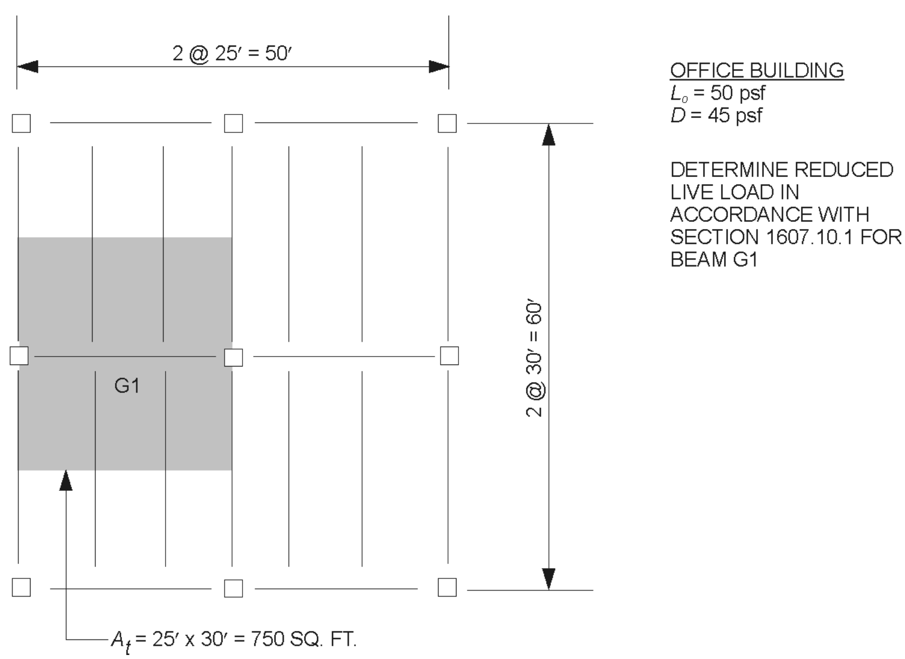
In Exception 1, the loads on structural members, such as columns and bearing walls that support two or more floors, are allowed to be reduced by 20 percent. Surveys have indicated that it is rare for the total live load on any story to exceed 80 percent of the tabulated uniform live loads. Conservatively, the full load should apply to beams and girders, but a member supporting multiple floors is allowed some live load reduction.

Recognizing that there are circumstances under which live loads exceed 100 psf (4.79 kN/m^2) in occupancies other than storage, Exception 2 allows the registered design professional to present a “rational” live load reduction proposal to be applied for members with larger tributary areas (e.g., girders, columns, foundations, etc.). Examples would be mechanical rooms, electrical rooms, process mezzanines in industrial buildings, etc. These types of areas may have very high localized uniform loads under the equipment footprints, for instance, but the live loads to members having larger tributary areas are much less, on average.

1607.10.1.3 Passenger vehicle garages. The live loads shall not be reduced in passenger vehicle garages.

Exception: The live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than *L* as calculated in Section 1607.10.1.

❖ This section limits the live load reduction for passenger vehicle garages to only those members that support more than two floors. Thus, floor framing members that support only a part of one floor do not warrant a reduction to the live load of 40 psf (1.92 kN/m^2) that is specified in Table 1607.1. The rationale for allowing some live load reduction for members supporting multiple floors is similar to that given under Section 1607.10.1.2, Exception 1.



For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929 m^2, 1 pound per square foot = 47.88 Pa.

Figure 1607.10.1
LIVE LOAD REDUCTION EXAMPLE

1607.10.2 Alternative uniform live load reduction. As an alternative to Section 1607.10.1 and subject to the limitations of Table 1607.1, uniformly distributed live loads are permitted to be reduced in accordance with the following provisions. Such reductions shall apply to slab systems, beams, girders, columns, piers, walls and foundations.

1. A reduction shall not be permitted where the live load exceeds 100 psf (4.79 kN/m^2) except that the design live load for members supporting two or more floors is permitted to be reduced by a maximum of 20 percent.

   Exception: For uses other than storage, where *approved*, additional live load reductions shall be permitted where shown by the *registered design professional* that a rational approach has been used and that such reductions are warranted.

2. A reduction shall not be permitted in passenger vehicle parking garages except that the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent.
3. For live loads not exceeding 100 psf (4.79 kN/m^2), the design live load for any structural member supporting 150 square feet (13.94 m^2) or more is permitted to be reduced in accordance with Equation 16-24.
4. For one-way slabs, the area, A, for use in Equation 16-24 shall not exceed the product of the slab span and a width normal to the span of 0.5 times the slab span.

$$R = 0.08(A - 150) \qquad \textbf{(Equation 16-24)}$$

For SI: $R = 0.861(A - 13.94)$

Such reduction shall not exceed the smallest of:

1. 40 percent for horizontal members;
2. 60 percent for vertical members; or
3. R as determined by the following equation.

$$R = 23.1(1 + D/L_o) \qquad \textbf{(Equation 16-25)}$$

where:

A = Area of floor supported by the member, square feet (m^2).

D = Dead load per square foot (m^2) of area supported.

L_o = Unreduced live load per square foot (m^2) of area supported.

R = Reduction in percent.

❖ This section includes an alternative floor live load method that is permitted to be used instead of the method indicated in Sections 1607.10.1 through 1607.10.1.3. The basis for this section is the 1997 UBC.

Where reductions are permitted, they are allowed at a rate of 0.08 percent per square foot of area in excess of 150 square feet (14 m^2). This value cannot exceed 60 percent for vertical members, such as columns or bearing walls, or 40 percent for horizontal members, such as beams or girders. Additionally, the reduction cannot be more than the value determined by Equation 16-25.

Example of an alternate floor live load reduction:

For beam G1 given in Figure 1607.10.1, determine the reduced floor live load in accordance with Section 1607.10.2.

Solution: A_T = 750 square feet (69 m^2) > 150 square feet (14 m^2); therefore, a reduction is permitted.

Equation 16-24 R = (0.08%)(750 - 150) = 48% > 40%

Equation 16-25

$$R = (23.1\%)\left(1 + \frac{45}{50}\right) = 43.7\% > 40\%$$

Use the maximum 40 percent reduction allowed for horizontal members.

Use L = 50(1-0.4) = 30 psf (144 kN/m^2).

1607.11 Distribution of floor loads. Where uniform floor live loads are involved in the design of structural members arranged so as to create continuity, the minimum applied loads shall be the full dead loads on all spans in combination with the floor live loads on spans selected to produce the greatest *load effect* at each location under consideration. Floor live loads are permitted to be reduced in accordance with Section 1607.10.

❖ For continuous floor members loaded such that the live loads of a building are distributed in some bays and not in others, some of the structural elements will be subjected to greater stresses because of partial loading conditions as compared to full loading on all spans. This code section requires the engineer to consider partial loadings that produce the greatest design forces for any location in the design of continuous floor elements.

For example, Figure 1607.11 shows a continuous multispan girder with partial loading. The Type I loading condition shows that only the alternate spans have uniform live loads, which produces:

- Maximum positive moments at the centers of the loaded spans (A-B, C-D, E-F) and
- Maximum negative moments at the centers of the unloaded spans (B-C, D-E, F-G).

The Type II live load distribution shows two loaded adjacent spans with alternate spans loaded beyond these, which produces:

- Maximum negative moment at Support D and
- Maximum girder shears.

To obtain the maximum total stresses imposed on the girder, the dead load moments and shears must be added to those produced by the partial live loadings.

1607.12 Roof loads. The structural supports of roofs and marquees shall be designed to resist wind and, where applicable, snow and earthquake loads, in addition to the dead load of construction and the appropriate live loads as prescribed in this section, or as set forth in Table 1607.1. The live loads acting on a sloping surface shall be assumed to act vertically on the horizontal projection of that surface.

❖ In addition to dead and live loads, the roof's structural system is to be designed and constructed to resist environmental loads caused by wind, snow and earthquakes. According to the definition of "Roof live loads" in Section 202, these are typically an allowance for maintenance of equipment as well as the roof system itself. Other roof live-loads must be considered where appropriate, such as "occupiable roofs" (see Section 1607.12.3) where an occupancy-related live load would be applicable.

1607.12.1 Distribution of roof loads. Where uniform roof live loads are reduced to less than 20 psf (0.96 kN/m^2) in accordance with Section 1607.12.2.1 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 most unfavorable *load effect*. See Section 1607.12.2 for reductions in minimum roof live loads and Section 7.5 of ASCE 7 for partial snow loading.

❖ For continuous roof construction, where live loads are reduced to less than 20 psf (0.96 kN/m^2) as permitted in Section 1607.12.2.1, partial loadings must be included in the design of structural elements to determine the governing loading situation. For example, Figure 1607.11 shows a continuous multispan girder with partial loading. The Type I loading condition shows that only the alternate spans have uniform live loads, which produces:

- Maximum positive moments at the centers of the loaded spans (A-B, C-D, E-F) and
- Maximum negative moments at the centers of the unloaded spans (B-C, D-E, F-G).

The Type II live load distribution shows two loaded adjacent spans with alternate spans loaded beyond these, which produces:

- Maximum negative moment at Support D and
- Maximum girder shears.

To obtain the maximum total stresses imposed on the girder, the dead load moments and shears must be added to those produced by the partial live loadings.

1607.12.2 General. The minimum uniformly distributed live loads of roofs and marquees, L_o, in Table 1607.1 are permitted to be reduced in accordance with Section 1607.12.2.1.

❖ The minimum roof live loads typically reflect loads that occur during roof maintenance, construction or repair. In addition to the standard roof live load of 20 psf (0.96 kN/m^2), Table 1607.1 includes roof live loads for special purpose roofs and fabric awnings. While this section seems to refer to reducing any of the tabulated uniformly distributed live loads, the actual reduction method provided in Section 1607.12.2.1 is limited to the 20 psf (0.96 kN/m^2) live load. This is made evident by the limits on Equation 16-26 of $12 \le L_r \le 20$.

1607.12.2.1 Ordinary roofs, awnings and canopies. Ordinary flat, pitched and curved roofs, and awnings and canopies other than of fabric construction supported by a skeleton structure, are permitted to be designed for a reduced uniformly distributed roof live load, L_r, as specified in the following equations or other controlling combinations of loads as specified in Section 1605, 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

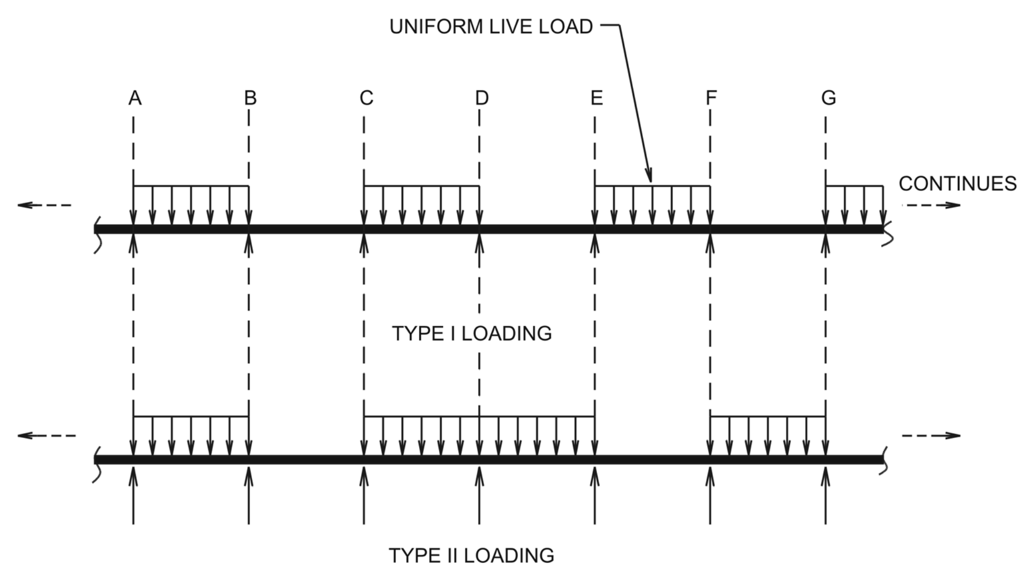


Figure 1607.11
PARTIAL LOADING

specified in the following equations shall not be used unless *approved* by the *building official*. Such structures shall be designed for a minimum roof live load of 12 psf (0.58 kN/m^2).

$L_r = L_o R_1 R_2$ **(Equation 16-26)**

where: $12 \leq L_r \leq 20$

For SI: $L_r = L_o R_1 R_2$

where: $0.58 \leq L_r \leq 0.96$

L_o = Unreduced roof live load per square foot (m^2) of horizontal projection supported by the member (see Table 1607.1).

L_r = Reduced roof live load per square foot (m^2) of horizontal projection supported by the member.

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

$R_1 = 1$ for $A_t \leq 200$ square feet (18.58 m^2) **(Equation 16-27)**

$R_1 = 1.2 - 0.001A_t$ for 200 square feet $< A_t <$ 600 square feet **(Equation 16-28)**

For SI: $1.2 - 0.011A_t$ for 18.58 square meters $< A_t <$ 55.74 square meters

$R_1 = 0.6$ for $A_t \geq 600$ square feet (55.74 m^2) **(Equation 16-29)**

where:

A_t = Tributary area (span length multiplied by effective width) in square feet (m^2) supported by the member, and

$R_2 = 1$ for $F \leq 4$ **(Equation 16-30)**

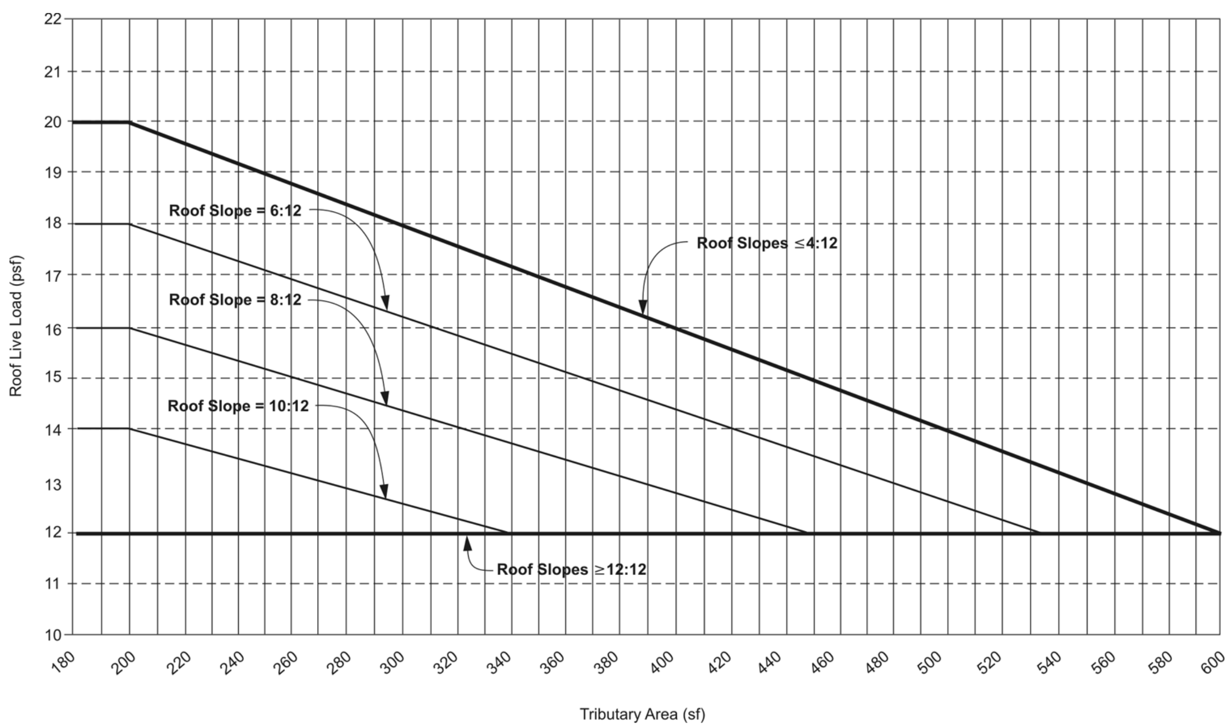
$R_2 = 1.2 - 0.05\,F$ for $4 < F < 12$ **(Equation 16-31)**

$R_2 = 0.6$ for $F \geq 12$ **(Equation 16-32)**

where:

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

❖ This section provides a formula for the determination of the live load for the design of flat, pitched or curved roofs. The live load from Table 1607.1 that applies is 20 psf (0.96 kN/m^2). Reduced roof live loads are based on the roof slope and the tributary area of the member being considered. The portion of the live load reduction based on tributary area does not apply to roof members that support small tributary areas of less than 200 square feet (18.58 m^2). The load can be reduced as the tributary area increases but never to less than 12 psf (0.58 kN/m^2). For roof slopes between 4:12 and 12:12, live load reductions based on slope apply. Figure 1607.12.2.1 shows the roof



For SI: 1 pound per square foot = 47.88 Pa, 1 square foot = 0.0929 m^2.

Figure 1607.12.2.1
MINUMUM ROOF LIVE LOAD IN ACCORDANCE WITH EQUATION 16-26

live load, L_r, determined by Equation 16-26 for increments of roof slope. Since the relationship of variables is linear, intermediate values can be interpolated from the figure.

This section also provides for a lower roof live load for a greenhouse, since it is not likely that loads from maintenance or repair will exceed the specified 12 psf (0.58 kN/m^2).

1607.12.3 Occupiable roofs. Areas of roofs that are occupiable, such as roof gardens, or for assembly or other similar purposes, and marquees are permitted to have their uniformly distributed live loads reduced in accordance with Section 1607.10.

❖ Roofs that are to be occupied during social events incidental to the principal use of the facility are to be designed for a minimum uniform live load of 60 psf (2.87 kN/m^2). The promenade deck of a residential penthouse located on the main roof of an apartment building is an example of this type of use. Where roofs are designed to be used as roof gardens or to support large gatherings of people as a function accompanying the educational or assembly uses of a facility, the roofs are required to be designed to a minimum live load of 100 psf (4.79 kN/m^2). The minimum live loads specified in Table 1607.1 are not required to be added to the design load requirements for occupiable roofs specified in this section.

1607.12.3.1 Landscaped roofs. The uniform design live load in unoccupied landscaped areas on roofs shall be 20 psf (0.958 kN/m^2). The weight of all landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil.

❖ Those areas of a roof that are to be landscaped are required to be designed for a minimum uniform live load of 20 psf (0.96 kN/m^2) to accommodate the occasional loads associated with the maintenance of plantings. The weight of landscaping materials and saturated soil is to be considered a dead load in the design of the roof structure, which is to be combined with the live load (see Section 1605 for applicable load combinations).

1607.12.4 Awnings and canopies. Awnings and canopies shall be designed for uniform live loads as required in Table 1607.1 as well as for snow loads and wind loads as specified in Sections 1608 and 1609.

❖ Awning structures are lightweight frames that are typically covered with fabric materials and are designed to sustain a live load of 5 psf (0.24 kN/m^2), as well as the specified snow and wind loads of Sections 1608 and 1609. The live load, snow load and wind load are to be combined according to Section 1605.

1607.13 Crane loads. 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.

❖ This section provides a general description of the crane loads that are required to be included in the design. The supporting structure for the crane is to be designed for a combination of the maximum wheel load, vertical impact and horizontal load as a simultaneous load combination. The typical arrangement for a top-running bridge crane is shown in Figure 1607.13.

1607.13.1 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.

❖ The maximum vertical wheel load occurs when the trolley is moved as close as possible to the supporting beams under consideration. This results in the greatest portion of the crane weight, the design weight lifted load and the wheel vertical impact load on the supporting beams.

1607.13.2 Vertical impact force. The maximum wheel loads of the crane shall be increased by the percentages shown below to determine the induced vertical impact or vibration force:

Monorail cranes (powered) . . . . 25 percent

Cab-operated or remotely operated bridge cranes (powered). . . . . 25 percent

Pendant-operated bridge cranes (powered). . . . . 10 percent

Bridge cranes or monorail cranes with hand-geared bridge, trolley and hoist . . . . 0 percent

❖ A vertical impact force is necessary to account for the impact from the starting and stopping movement of the suspended weight from the crane. Vertical impact is also created by the movement of the crane along the rails.

1607.13.3 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.

❖ This section is necessary to define the design lateral force on the crane supports. Lateral force at the right angle to the crane rail is caused by the lateral movement of the lifted load and from the frame action of the crane.

1607.13.4 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.

❖ This section is needed to define the longitudinal force on the crane supports, which is caused from the longitudinal motion of the crane with the lifted load.

1607.14 Interior walls and partitions. Interior walls and partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength to resist the loads to which they are subjected but not less than a horizontal load of 5 psf (0.240 kN/m^2).

Exception: Fabric partitions complying with Section 1607.14.1 shall not be required to resist the minimum horizontal load of 5 psf (0.24 kN/m^2).

❖ The minimum lateral live load is intended to provide sufficient strength and durability of the wall framing and of the finished construction to provide a minimum level of resistance to nominal impact loads that commonly occur in the use of a facility, such as impacts from moving furniture or equipment, as well as to resist heating, ventilating and air-conditioning (HVAC) pressurization.

1607.14.1 Fabric partitions. Fabric partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength to resist the following load conditions:

1. A horizontal distributed load of 5 psf (0.24 kN/m^2) applied to the partition framing. The total area used to determine the distributed load shall be the area of the fabric face between the framing members to which the fabric is attached. The total distributed load shall be uniformly applied to such framing members in proportion to the length of each member.
2. A concentrated load of 40 pounds (0.176 kN) applied to an 8-inch diameter (203 mm) area [50.3 square inches (32 452 mm^2)] of the fabric face at a height of 54 inches (1372 mm) above the floor.

❖ This section provides criteria for fabric partitions (see definition, Section 202) as an alternative to Section 1607.14. The construction of these partitions is unique, which makes it difficult to meet the full requirements of Section 1607.14. Condition 1 requires the partition framing to be capable of resisting a minimum lateral load. Condition 2 approximates the load of a person leaning against the fabric using their hand as the point of contact. This criterion is based on test standards that are used to evaluate the tip-over resistance of office furniture panel systems that are often used to provide open plan offices.

SECTION 1608 SNOW LOADS

1608.1 General. Design snow loads shall be determined in accordance with Chapter 7 of ASCE 7, but the design roof load shall not be less than that determined by Section 1607.

❖ The determination of the nominal snow load, S, must be in accordance with this section. The intent of this section is that the code requirements are based on the technical requirements in Chapter 7 of ASCE 7. The snow load provisions in ASCE 7 are based on over 40 years of ground snow load data, and include consideration of thermal resistance of the roof structure, a rain-on-snow surcharge, partial loading on

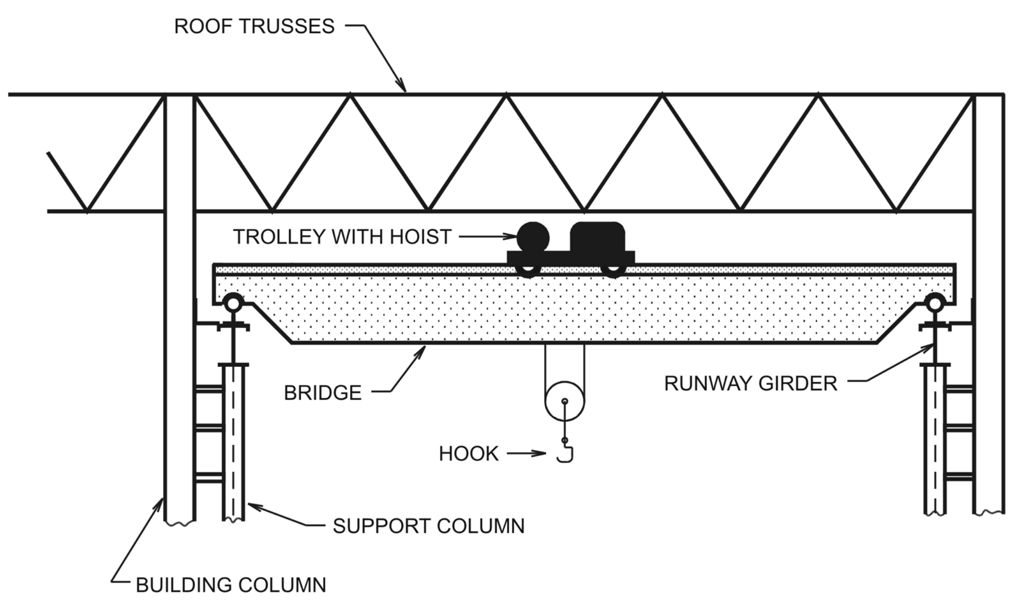


Figure 1607.13
TOP-RUNNING BRIDGE CRANE

continuous beam systems and ponding instability from melting snow or rain on snow. The variables that affect the determination of roof snow loads are:

- Ground snow load (p_g) – see Section 1608.2.
- Importance factor (I) – a factor determined in Section 7.3.3 of ASCE 7, ranging from 0.8 to 1.2, based on the risk category assigned in accordance with Section 1604.5.
- Exposure factor (C_e) and Thermal factor (C_t) – see Sections 7.3.1 and 7.3.2 of ASCE 7.

In addition to the above, the roof slope affects the snow load determination, as well as other design considerations. Roofs with low slopes are designed for a flat roof snow load, p_f determined in accordance with Section 7.3 of ASCE 7, using the previously mentioned criteria. Other considerations for low roof slopes include ponding instability and rain-on-snow surcharge loading. For sloped roofs, the sloped roof snow load, p_s in accordance with Section 7.4 of ASCE 7 applies. The sloped roof snow load essentially modifies the flat roof snow load by the slope factor (C_s), which varies from 1.0 at low a slope (i.e., no effect) to zero at a slope of 70 degrees (1.36 rad).

The flat roof snow load or sloped roof snow load applied uniformly to the entire roof is referred to as the balanced snow load condition. This loading is always a design consideration. Depending on factors such as the type of roof structural system, the geometry of the roof, etc., the following additional snow loadings may require evaluation:

- Partial loading (see Section 7.5, ASCE 7) is a pattern consisting of balanced snow load and one-half of balanced snow load arranged to produce the maximum effects on the structural member being considered.
- Unbalanced snow loads (see Section 7.6, ASCE 7) reflect an uneven loading pattern, such as can occur on a sawtooth roof [see Figure 1608.1(1)].
- Drifting (see Sections 7.7 and 7.8, ASCE 7) is a concern where adjacent roof surfaces are at different elevations [see Figures 1608.1(2) and (3)] or at projections above the roof level, such as at equipment or parapets.

1608.2 Ground snow loads. The ground snow loads to be used in determining the design snow loads for roofs shall be determined in accordance with ASCE 7 or Figure 1608.2 for the contiguous United States and Table 1608.2 for Alaska. Site-specific case studies shall be made in areas designated "CS" in Figure 1608.2. Ground snow loads for sites at elevations above the limits indicated in Figure 1608.2 and for all sites within the CS areas shall be *approved*. 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 *approved* by the *building official*.

❖ The ground snow loads on the maps in Figure 1608.2 of the code are generally based on over 40 years of snow depth records. The snow loads on the maps are those that have a 2-percent annual probability of being exceeded (a 50-year mean recurrence interval). The maps were generated from data through the winter of 1991-92, and from data through the winter of 1993-94 where the snows were heavy. The mapped snow loads are not increased much from a single snowy winter, since most reporting stations have more than 20 years of snow data. The map values indicate the ground snow load in pounds per square foot. In mountainous areas, the map also indi-

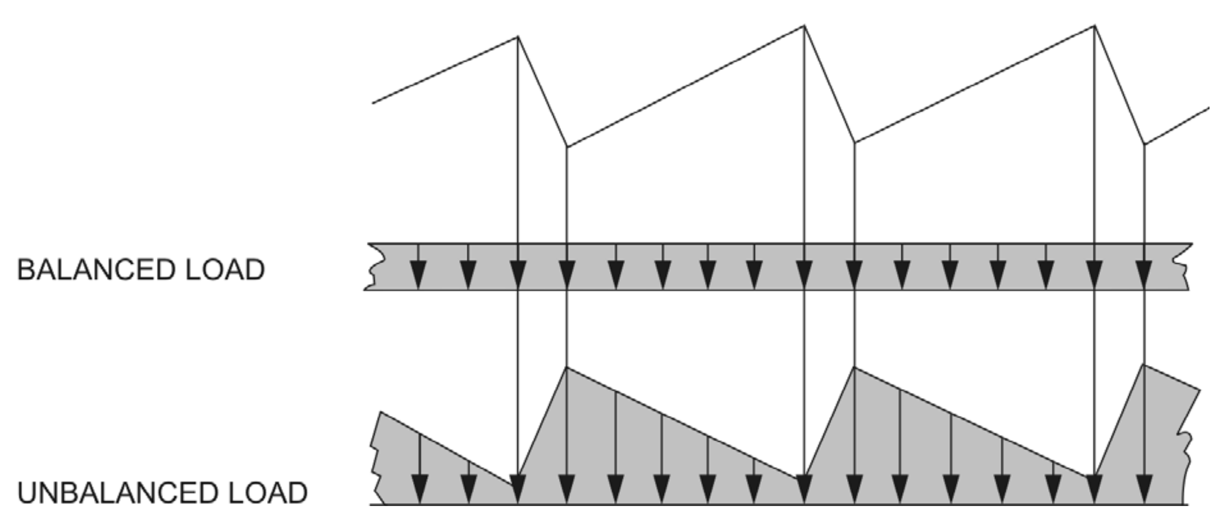


Figure 1608.1(1)
BALANCED AND UNBALANCED LOADS FOR A SAWTOOTH ROOF

cates the highest elevation that is appropriate for the use of the associated snow load. Where the elevation limit is exceeded, a site-specific case study is necessary to establish the appropriate ground snow load. In some areas the ground snow load is too variable to allow mapping. These areas are noted as “CS,” which indicates a site-specific case study is necessary. Assistance in the determination of an appropriate ground snow load for these areas may be obtained from the U.S. Department of Army Cold Regions Research and Engineering Laboratory (CRREL) in Hanover, New Hampshire.

TABLE 1608.2. See page 16-35.

❖ Since Alaska in not shown on Figure 1608.2, this table provides the ground snow loads, Pg, for Alaskan locations. These values are needed to determine the appropriate roof snow loads for the indicated locations. The roof snow load is to be determined according to Section 7.3 of ASCE 7 for flat roofs and Section 7.4 of ASCE 7 for sloped roofs.

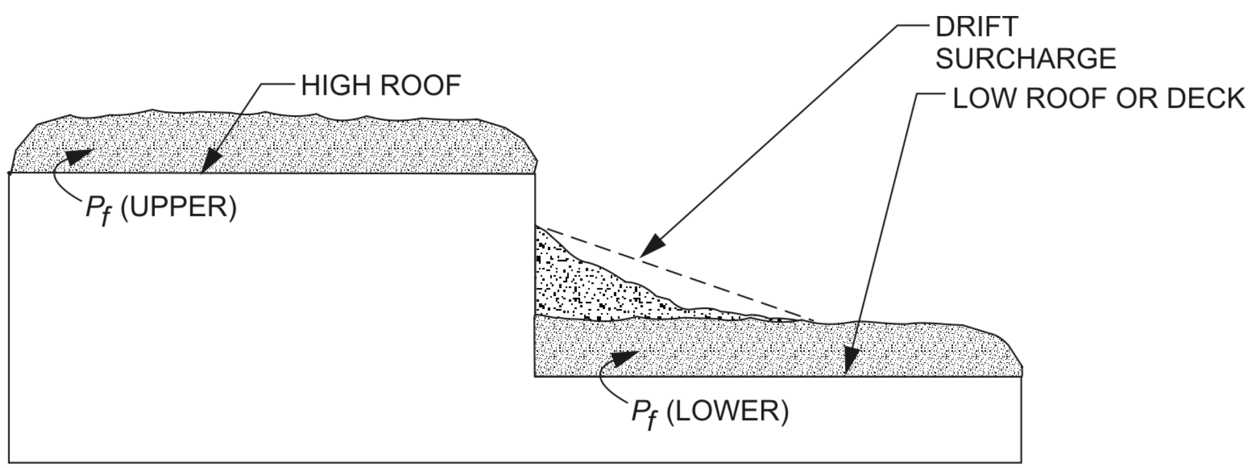
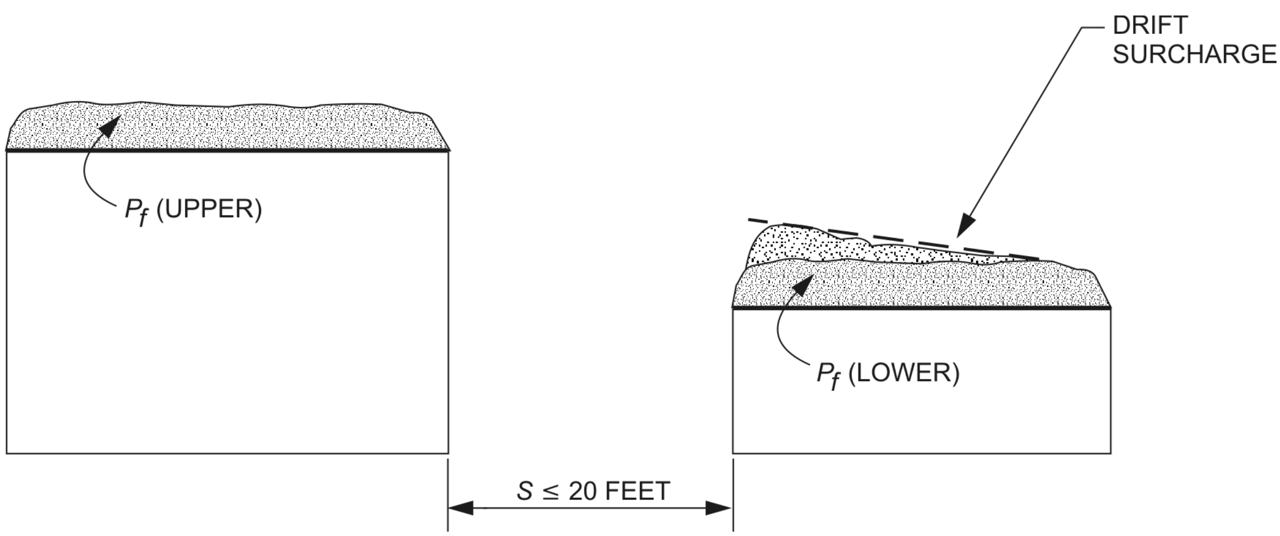


Figure 1608.1(2)
DRIFTING SNOW ON LOW ROOFS AND DECKS



For SI: 1 foot = 304.8 mm.

Figure 1608.1(3)
DRIFTING SNOW ONTO ADJACENT LOW STRUCTURE

FIGURE 1608.2. See page 16-36.

❖ See the commentary to Section 1608.2 for an overview of the snow load map. See the commentary for Section 7.2 of ASCE 7 for a complete description of methodology used in developing the contour lines shown on the figure.

1608.3 Ponding instability. Susceptible bays of roofs shall be evaluated for ponding instability in accordance with Section 7.11 of ASCE 7.

❖ Susceptible bays of roofs are required to meet the technical provisions of ASCE 7 for consideration of progressive deflection. The term “Susceptible bay” is defined in Section 202 and provides a technical basis for determining which bays of a roof need to be investigated for ponding instability. Section 1611.2 also relies on the determination of which bays are susceptible bays.

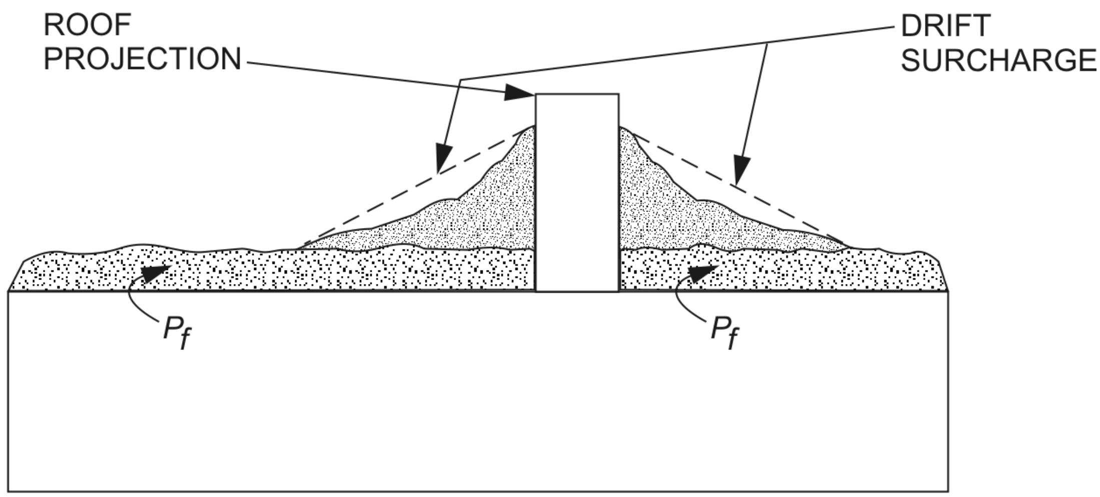
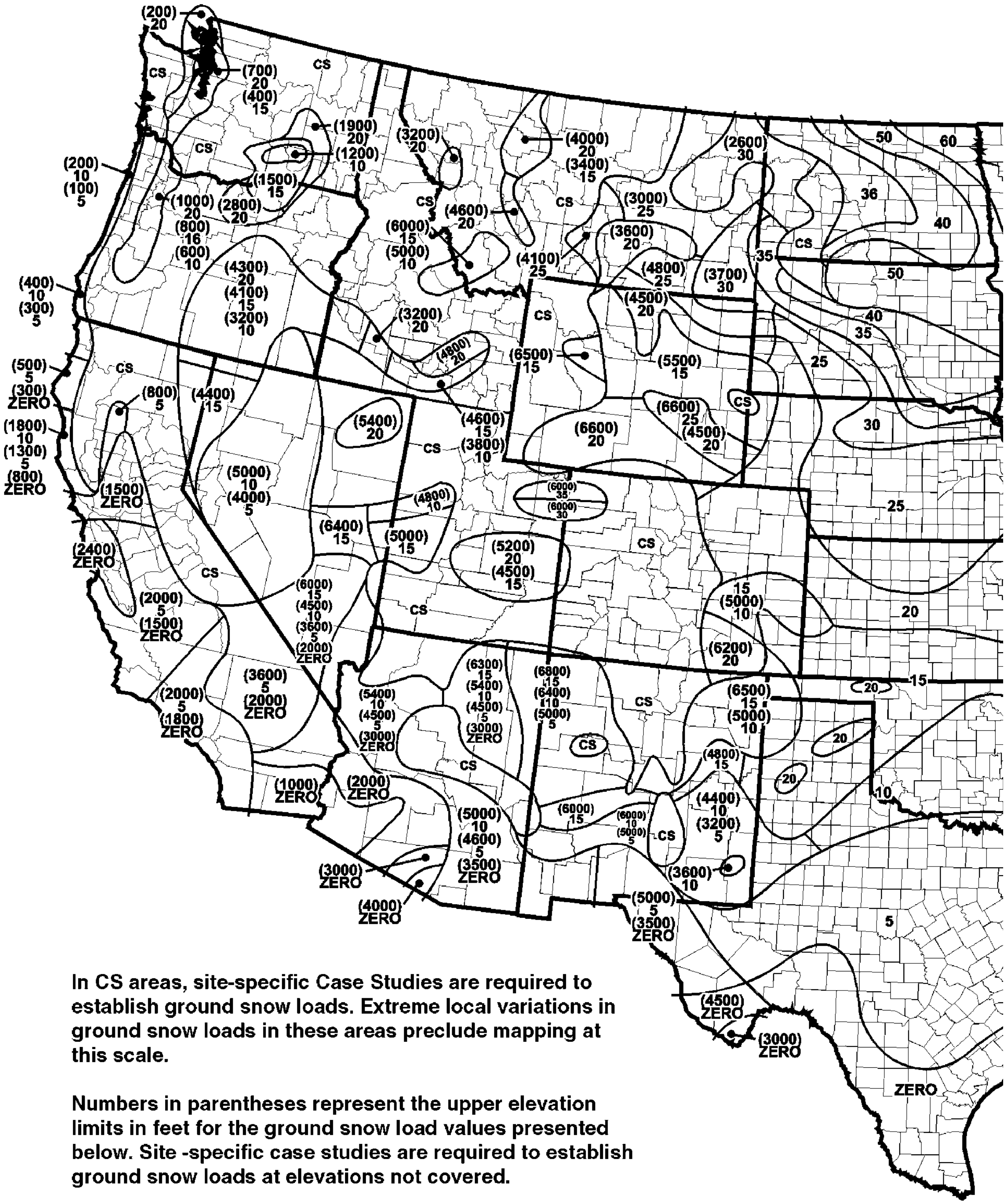


Figure 1608.1(4)
SNOW DRIFTING AT ROOF PROJECTIONS

TABLE 1608.2
GROUND SNOW LOADS, p_g, FOR ALASKAN LOCATIONS

LOCATION	POUNDS PER SQUARE FOOT	LOCATION	POUNDS PER SQUARE FOOT	LOCATION	POUNDS PER SQUARE FOOT
Adak	30	Galena	60	Petersburg	150
Anchorage	50	Gulkana	70	St. Paul Islands	40
Angoon	70	Homer	40	Seward	50
Barrow	25	Juneau	60	Shemya	25
Barter Island	35	Kenai	70	Sitka	50
Bethel	40	Kodiak	30	Talkeetna	120
Big Delta	50	Kotzebue	60	Unalakleet	50
Cold Bay	25	McGrath	70	Valdez	160
Cordova	100	Nenana	80	Whittier	300
Fairbanks	60	Nome	70	Wrangell	60
Fort Yukon	60	Palmer	50	Yakutat	150

For SI: 1 pound per square foot = 0.0479 kN/m^2.



In CS areas, site-specific Case Studies are required to establish ground snow loads. Extreme local variations in ground snow loads in these areas preclude mapping at this scale.

Numbers in parentheses represent the upper elevation limits in feet for the ground snow load values presented below. Site -specific case studies are required to establish ground snow loads at elevations not covered.

To convert lb/sq ft to kNm^2 , multiply by 0.0479.

To convert feet to meters, multiply by 0.3048.

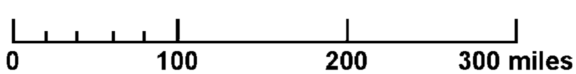


FIGURE 1608.2—continued
GROUND SNOW LOADS, p_g, FOR THE UNITED STATES (psf)

FIGURE 1608.2—continued
GROUND SNOW LOADS, p_g, FOR THE UNITED STATES (psf)

SECTION 1609
WIND LOADS

FIGURES 1609A, 1609B & 1609C. See pages 16-41 through 16-43.

❖ Over the past decade, new data and research have indicated that the mapped hurricane wind speeds have been overly conservative. Significantly more hurricane data has become available, which in turn allows for improvements in the hurricane simulation model that is used to develop wind speed maps. The new hurricane hazard model yields hurricane wind speeds that are lower than those given in previous editions of ASCE 7 and the code, even though the overall rate of intense storms (as defined by central pressure) produced by the new model is increased compared to those produced by the hurricane simulation model used to develop previous maps. In preparing new maps, it was decided to use multiple ultimate event or strength design maps in conjunction with a wind load factor of 1.0 for strength design (see Section 1605.2). For allowable stress design, the load factor has been reduced from 1.0 to 0.6 (see Section 1605.3). Several factors that are important to an accurate wind load standard led to this decision:

1. An ultimate event or strength design wind speed map makes the overall approach consistent with that used in seismic design in that they both map ultimate events and use a load factor of 1.0 for strength design.
2. Utilizing different maps for the different risk categories eliminates the problems associated with using “importance factors” that vary with category. The difference in the importance factors in hurricane-prone and nonhurricane-prone regions for Risk Category I structures, which prompted many questions, is gone.
3. The use of multiple maps eliminates the confusion associated with the recurrence interval associated with the previous wind speed map—the map was not a uniform 50-year return period map. This created a situation where the level of safety that was provided within the overall design was not consistent along the hurricane coast.

Utilizing the new wind speed maps and integrating their use into the code necessitated the introduction of the terms “V_{ult}” and “V_{asd}” to be associated with the “ultimate” design wind speed and the “nominal” design wind speed, respectively. Because of the number of different provisions that use the wind speed map to “trigger” different requirements, it was necessary to provide a conversion methodology in Section 1609.3.1 so that those provisions were not affected. The terms “ultimate design wind speed” and “nominal design wind speed” have been incorporated to clarify the different levels of wind speed.

Prior to 1998, earlier editions of ASCE 7 as well as the legacy model codes incorporated fastest-mile wind speed maps. “Fastest mile” is defined as the average speed of one mile of air that passes a specific reference point. It is important to recognize the differences in averaging times between fastest-mile and the 3-second-gust maps. The averaging time for a 90-mph fastest-mile wind speed is (t = 3,600/V) 40 seconds. Obviously, due to greater averaging time, for a given location the fastest-mile wind speed will be less than the 3-second-gust wind speed. The fact that the wind speed values are higher does not necessarily indicate higher wind loads. Buildings and structures resist wind loads, not wind speeds. Wind speed, although a significant contributor, is only one of several variables and factors that affect wind forces. Wind loads are affected by atmosphere and aerodynamics. Other elements that affect actual wind forces as wind flows across a bluff body include shape factors (C_a), gust effect factors (G) and the velocity pressure that is a function of wind speed, exposure and topography, among others.

The change from the fastest-mile wind speed map to a 3-second-gust map in the 2000 code (based on the 1998 edition of ASCE 7) was necessary for the following reasons. First, weather stations across the United States no longer collect fastest-mile wind speed data. Additionally, the perception of the general public will be more favorable where the code wind speeds are higher, although the design wind pressures were not changed significantly. The map includes a more complete analysis of hurricane wind speeds than previous maps, since more data was available for sites away from the coast. In western states, the 85- to 90-mph contour boundary follows along the Washington, Oregon and California eastern state lines. This is because inland wind data was such that there was no statistical basis to place them elsewhere. The reference to the 50-year MRI used in previous maps was removed, reflecting the fact that the MRI is greater than 50 years along the hurricane coastline. However, nonhurricane wind speeds are based on a 50-year MRI, as Section 1609.3 specifically requires when estimating basic wind speeds from local climatic data.

1609.1 Applications. Buildings, structures and parts thereof shall be designed to withstand the minimum wind loads prescribed herein. Decreases in wind loads shall not be made for the effect of shielding by other structures.

❖ The determination of the wind load, W, must be in accordance with Section 1609. The intent of this section is to provide minimum criteria for the design and construction of buildings and other structures to resist wind loads. These regulations serve to reduce the potential for damage to property caused by windstorms and to provide an acceptable level of protection to building occupants. The objective also includes the prevention of damage to adjacent properties because of the possible detachment of major building components (e.g., walls, roofs, etc.), structural collapse or flying debris and for the safety of people in the immediate vicinity.

The criteria for wind design given in this section of the code generally reflect the wind load provisions of ASCE 7. For a better understanding of the wind load provisions it is important to have a fundamental knowledge of the effects of high-velocity wind forces on buildings and other structures. Wind/structure interactions can be characterized as follows:

When wind encounters a stationary object, such as a building, the airflow changes direction and produces several effects on the building that are illustrated in Figure 1609.1(1). Exterior walls and other vertical surfaces facing the wind (windward side) and perpendicular to its path are subjected to inward (positive) pressures; however, wind does not stop on contact with a facing surface, but flows around and over the building surfaces. This airflow does not instantaneously change directions at surface discontinuities, such as corners of walls or eaves, or over ridges and roof corners. Instead it separates from the downwind surfaces due to high turbulence and localized pressures, resulting in outward (negative) pressures. This phenomena produces suction or outward pressures (negative) on the sidewalls, leeward wall and, depending on geometry, the roof.

Figure 1609.1(1) shows a flat roof and the resulting negative pressures caused by external wind; however, pressures may differ on sloping roofs in the direction perpendicular to the ridge. Roof surfaces (on the windward side) with shallow slopes are generally subjected to outward (negative) pressures—the same as flat roofs. Moderately sloping roofs [about 30 degrees (0.5235 rad)] may be subjected to either inward (positive) pressures, outward (negative) pressures or both—negative pressure in the lower part of the roof and positive pressure in the area of the ridge; however, the code does not require consideration of this scenario. High-sloping roofs (windward side) respond similar to walls and sustain positive wind pressures. The leeward side of a sloped roof is subjected to negative pressure, regardless of the angle. When the wind acts parallel to the ridge, the pressures on a sloped roof are similar to the pressures on a flat roof, meaning the roof is subject to negative pressures.

Openings in the building envelope can impact the magnitude of wind pressure on a structure by increasing the internal pressures that, in turn, will affect the net pressures on the structure. Figure 1609.1(2) illustrates the effects of openings in a building's exterior walls. An opening in the leeward wall causes negative pressure on the interior, increasing the total load on the windward wall. Openings on the windward wall cause positive internal pressure against all the walls from the interior of the building. As the figure shows, the result is an increase in the total load on the leeward wall. In extreme cases, fail-

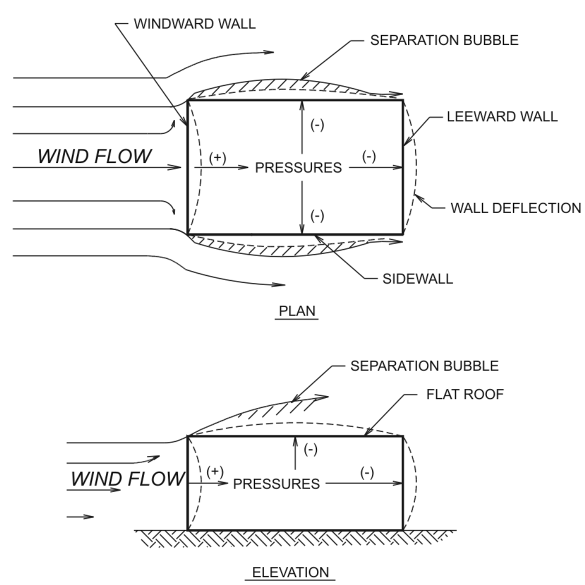


Figure 1609.1(1)
WIND PRESSURES CAUSED BY EXTERNAL WIND FLOW

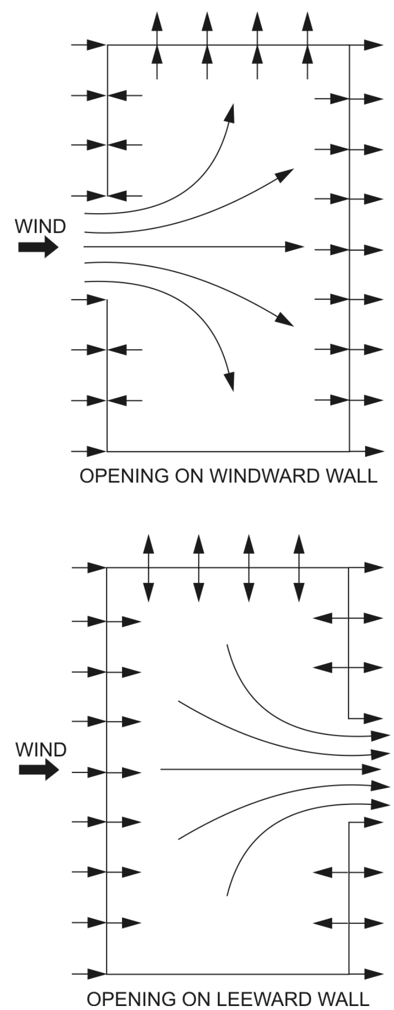


Figure 1609.1(2)
OPENING IN EXTERIOR WALL OF BUILDING

ures resulting from this type of wind flow appear as if the building has exploded.

The definition of "Openings" is worth noting, since in the nonstructural code provisions the term "openings" typically refers to doors, ducts or windows. The definition of "Openings" that is relevant to wind load is in Section 26.2 of ASCE 7. They are all described as "apertures or holes in the building envelope that allow air to flow through the building envelope and that are designed as 'open' during design wind loads." Such openings are then considered in classifying the building as enclosed, partially enclosed or open. The wind load considerations will differ based on that classification.

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 of ASCE 7 or provisions of the alternate all-heights method in Section 1609.6. The type of opening protection required, the ultimate design wind speed, V_{ult}, and the exposure category for a site is permitted to be determined in accordance with Section 1609 or ASCE 7. Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of ICC 600 shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AF&PA WFCM.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AISI S230.
4. Designs using NAAMM FP 1001.
5. Designs using TIA-222 for antenna-supporting structures and antennas, provided the horizontal extent of Topographic Category 2 escarpments in Section 2.6.6.2 of TIA-222 shall be 16 times the height of the escarpment.
6. Wind tunnel tests in accordance with Chapter 31 of ASCE 7.

The wind speeds in Figures 1609A, 1609B and 1609C are ultimate design wind speeds, V_{ult}, and shall be converted in accordance with Section 1609.3.1 to nominal design wind speeds, V_{asd}, when the provisions of the standards referenced in Exceptions 1 through 5 are used.

❖ The intent of Section 1609 is to require that buildings and structures be designed and constructed to resist the wind loads quantified in Chapters 26 through 30 of ASCE 7.

There are six exceptions to using the provisions of ASCE 7 for the determination of wind loads.

Exception 1 provides for the use of ICC 600 for Group R-2 and R-3 buildings where they are located within Exposure B or C as defined in Section 1609.4 and not sited on the upper half of an isolated hill, escarpment or ridge with the characteristics described in Section 1609.1.1.1. ICC 600 has prescriptive construction requirements and required load capacity tables that replace the requirement for structural analysis, which is intended to provide improved design construction details to achieve greater structural performance for single- and multiple-family dwellings in a high-wind event (see ICC 600 for other detailed application limitations).

Exception 2 provides for the use of the AF&PA *Wood Frame Construction Manual for One- and Two-family Dwellings* where the building is sited within Exposure B or C as defined in Section 1609.4 and not on the upper half of an isolated hill, escarpment or ridge with the characteristics described in Section 1609.1.1.1. The AF&PA *Wood Frame Construction Manual for One- and Two-family Dwellings* has prescriptive construction requirements and required load-resistance tables that replace the requirement for structural analysis. The tabulated engineered and prescriptive design provisions apply to one- and two-family wood frame dwellings where the fastest-mile basic wind speed is between 90 and 120 mph. See Table 1609.3.1 for conversion between the 3-second-gust and fastest-mile wind speeds (see the AF&PA *Wood Frame Construction Manual for One- and Two-family Dwellings*, Chapter 1, for other detailed application limitations). Similarly, Exception 3 allows the cold-formed steel prescriptive standard for one- and two-family dwellings.

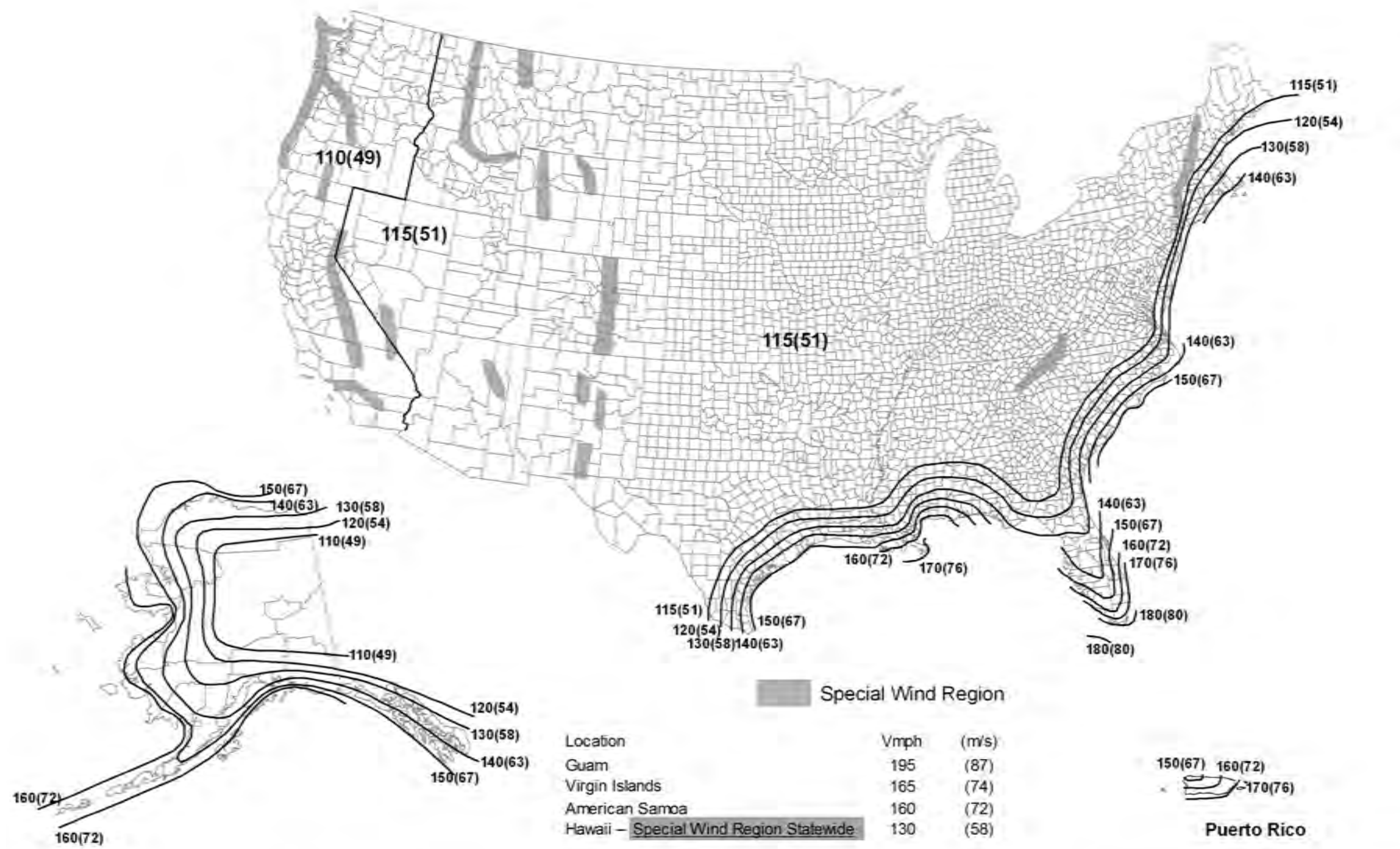
Exceptions 4 and 5 simply refer to national standards dealing specifically with the design of flag poles and telecommunication towers.

Exception 5 allows the use of TIA 222-G for antennas and their supporting structures. This standard provides a simplification of the topographic wind speed-up effect. The TIA 222-G standard allows a designer to use the full topographic wind speed-up method of ASCE 7 in order to avoid the conservatism of the simplified method. A code modification makes sure that the simplified method is safe in all cases.

TIA 222-G accounts for the worst-case wind speed-up at the crest for a steep slope, but overlooks the fact that lesser sloped escarpments create pressure increases that cannot be safely ignored beyond the "steep slope" influence. As written, the standard stops considering the topographic wind speed-up effect at "8 x height" from the crest. At this distance, a shallow slope can still increase wind pressure by more than 20 percent.

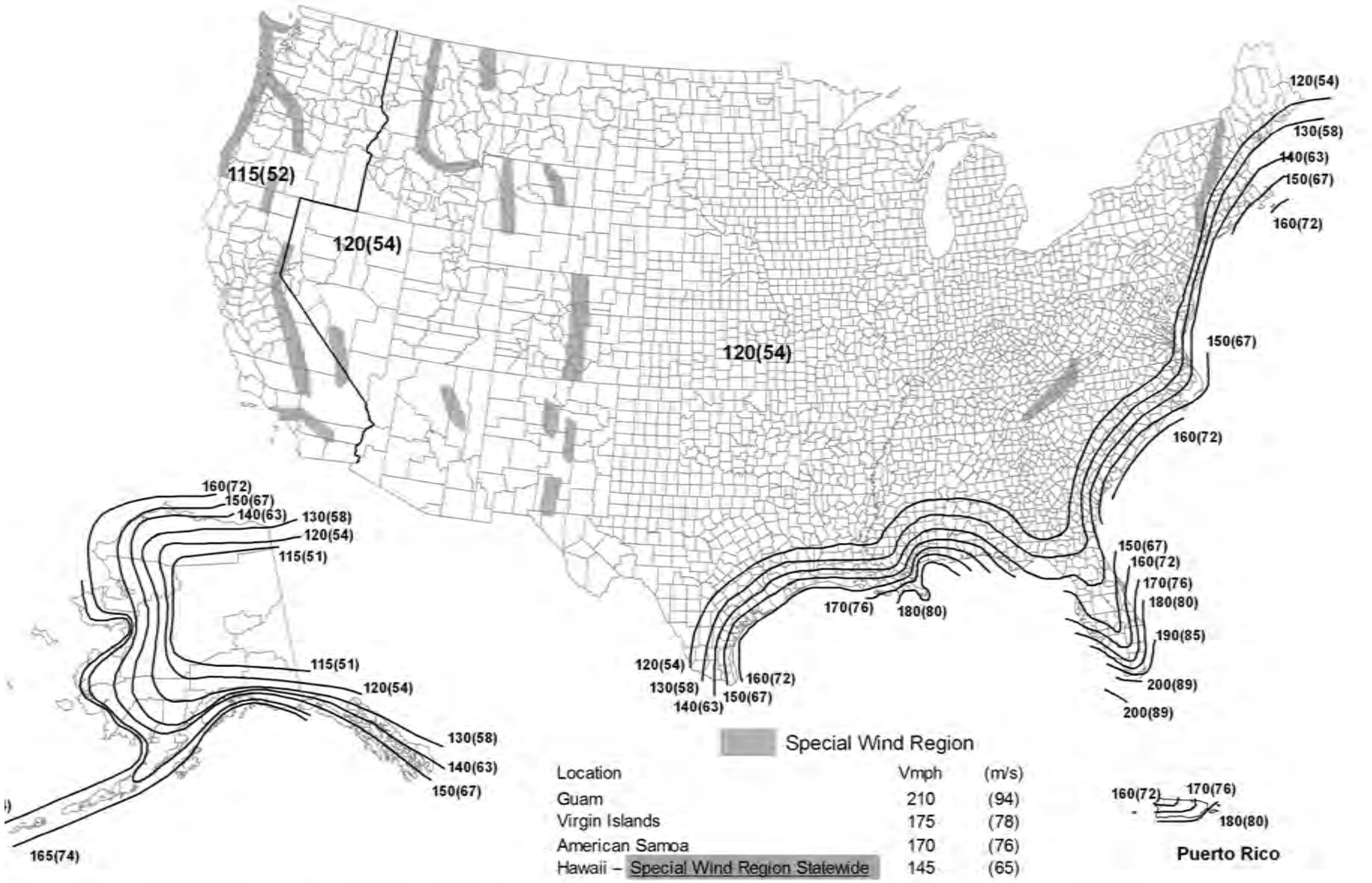
The modification changes "8 x height" in the standard to "16 x height." The intent is that the "Topographic category 2" definition in TIA 222-G should be applied as follows:

> Category 2. Structures located at or near the crest of an escarpment. Wind speed-up shall be considered to occur in all directions. Structures located vertically on the lower half of an escarpment or horizontally beyond 16 times the height of the escarpment from its crest, shall be permitted to be considered as Topographic Category 1.



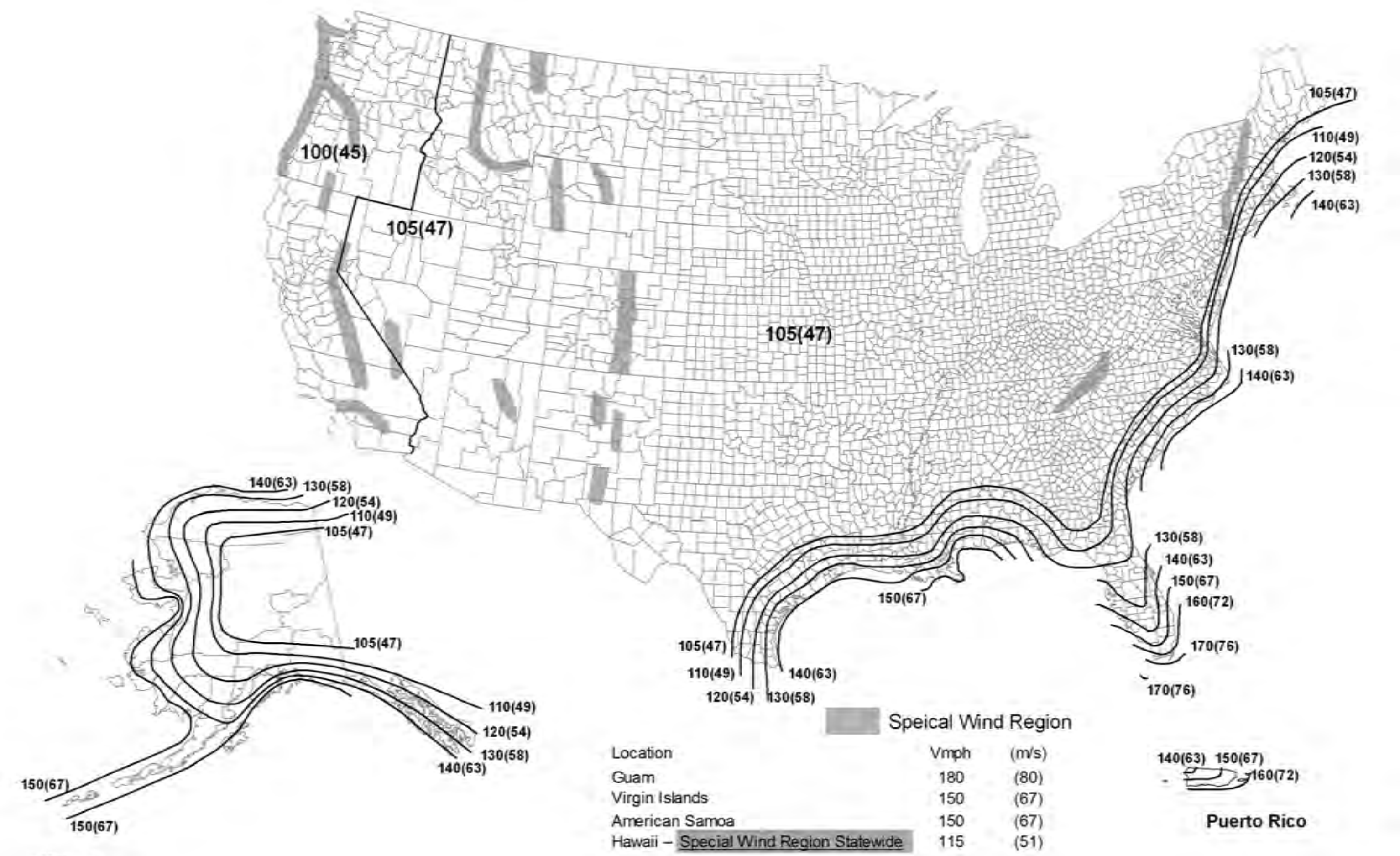
Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal 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).

FIGURE 1609A
ULTIMATE DESIGN WIND SPEEDS, V_{ULT}, FOR RISK CATEGORY II BUILDINGS AND OTHER STRUCTURES



tes:
/alues are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
.inear interpolation between contours is permitted.
slands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
/lountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
Vind speeds correspond to approximately a 3% probability of exceedance in 50 years (Annual Exceedance Probability = 0.000588, MRI = 1700 Years).

FIGURE 1609B
ULTIMATE DESIGN WIND SPEEDS, V_{ULT}, 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 15% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00333, MRI = 300 Years).

FIGURE 1609C
ULTIMATE DESIGN WIND SPEEDS, V_{ULT}, FOR RISK CATEGORY I BUILDINGS AND OTHER STRUCTURES

1609.1.1.1 Applicability. The provisions of ICC 600 are applicable only to buildings located within Exposure B or C as defined in Section 1609.4. The provisions of ICC 600, AF&PA WFCM and AISI S230 shall not apply to buildings sited on the upper half of an isolated hill, ridge or escarpment meeting the following conditions:

1. The hill, ridge or escarpment is 60 feet (18 288 mm) or higher if located in Exposure B or 30 feet (9144 mm) or higher if located in Exposure C;
2. The maximum average slope of the hill exceeds 10 percent; and
3. The hill, ridge or escarpment is unobstructed upwind by other such topographic features for a distance from the high point of 50 times the height of the hill or 1 mile (1.61 km), whichever is greater.

❖ This section places limitations on the use of ICC 600 and the AF&PA *Wood Frame Construction Manual for One- and Two-family Dwellings.* Neither of these standards accounts for the effect of isolated hills, ridges or escarpments. As illustrated in Figure 1609.1.1.1, the wind speed increases when a mass of air passes over those terrain features. This phenomenon is referred to as the "wind speed-up" effect and requires the use of ASCE 7.

1609.1.2 Protection of openings. In *wind-borne debris regions,* glazing in buildings shall be impact resistant or protected with an impact-resistant covering meeting the requirements of an *approved* impact-resistant standard or ASTM E 1996 and ASTM E 1886 referenced herein as follows:

1. Glazed openings located within 30 feet (9144 mm) of grade shall meet the requirements of the large missile test of ASTM E 1996.
2. Glazed openings located more than 30 feet (9144 mm) above grade shall meet the provisions of the small missile test of ASTM E 1996.

Exceptions:

1. Wood structural panels with a minimum thickness of $^7/_{16}$ inch (11.1 mm) and maximum panel span of 8 feet (2438 mm) shall be permitted for opening protection in one- and two-story buildings classified as Group R-3 or R-4 occupancy. Panels shall be precut so that they shall be attached to the framing surrounding the opening containing the product with the glazed opening. Panels shall be predrilled as required for the anchorage method and shall be secured with the attachment hardware provided. Attachments shall be designed to resist the components and cladding loads determined in accordance with the provisions of ASCE 7, with corrosion-resistant attachment hardware provided and anchors permanently installed on the building. Attachment in accordance with Table 1609.1.2 with corrosion-resistant attachment hardware provided and anchors permanently installed on the building is permitted for buildings with a mean roof height of 45 feet (13 716 mm) or less where V_{asd} determined in accordance with Section 1609.3.1 does not exceed 140 mph (63 m/s).
2. Glazing in *Risk Category* I buildings as defined in Section 1604.5, including greenhouses that are occupied for growing plants on a production or research basis, without public access shall be permitted to be unprotected.
3. Glazing in *Risk Category* II, III or IV buildings located over 60 feet (18 288 mm) above the ground and over 30 feet (9144 mm) above aggregate surface roofs located within 1,500 feet (458 m) of the building shall be permitted to be unprotected.

❖ The purpose of this section is to address risks associated with wind-borne debris in high-wind areas. See the definitions of "Wind-borne debris region" and "Hurricane-prone regions" in Section 1609.2 for an explanation of where these provisions apply. This section requires protection of glazed openings in buildings in wind-borne debris regions. This can be provided in the form of a protective assembly that is impact tested or by using impact-resistant glazing.

During a hurricane, buildings are impacted from

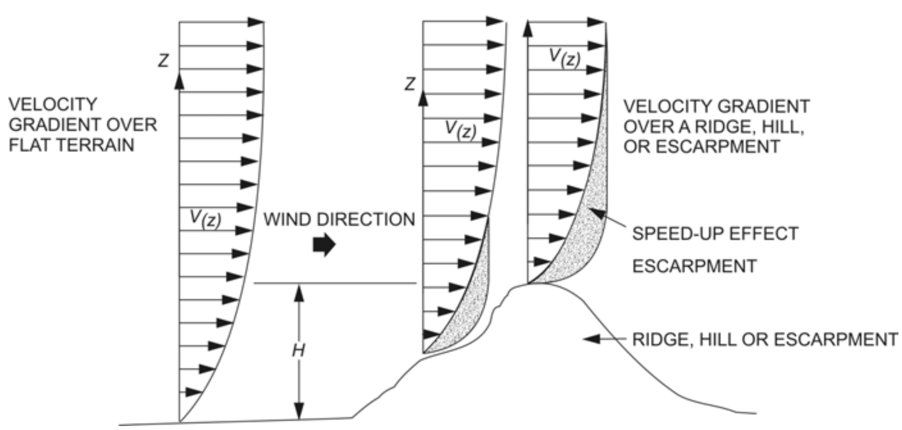


Figure 1609.1.1.1
WIND SPEED-UP EFFECT

wind-borne debris due to high-velocity winds. This debris can impact the glazing, causing breakage and creating an opening within the building envelope. The presence of openings in the building envelope can have a significant effect on the magnitude of the total wind pressure required to be resisted by each structural element of a building. Depending on the location of these openings with respect to wind direction and the amount of background porosity, external and internal pressures may act in the same direction to produce higher forces on some walls and the roof. An example of this is shown in Figure 1609.1.2. In this scenario, as the wind flows over the building, pressures are developed on the external surface, as shown. Introduction of an opening in the windward wall causes the wind to rush into the building, exerting internal pressures (positive) against all interior surfaces. This type of opening has the net effect of producing potentially high internal pressures that will act in the same direction as the external pressures on the roof, side and leeward walls. Considering the high probability of wind-borne debris during a hurricane and the effect of an unintended opening in the building envelope, the code requires glazing in designated regions to be protected.

Where wind-borne protection is provided, the section specifies two types of tests to demonstrate adequate resistance: the large missile test to simulate large debris up to 30 feet (9144 mm) above grade and the small missile test to simulate smaller debris up to 60 feet (18 288 mm) above grade, both of which are common during very high winds. An example of small debris is gravel from the surrounding area that becomes air borne.

Impact-resistant coverings or glazing must meet the test requirements of an approved impact standard or ASTM E 1886 and ASTM E 1996. Other impact standards the building official may consider are SBCCI SSTD 12 or Florida Building Code Test Protocol TAS 201, TAS 202 and TAS 203. These standards specify similar-type testing with a large missile test (2 by 4), small missile test (2 gram balls) and cyclic pressure loading test. ASTM E 1886 and ASTM E 1996 work together with the standard test method (E 1886) and the test specification (E 1996), including scoping, technical requirements and pass/fail criteria.

Exception 1 permits the use of $^{7}/_{16}$-inch (11.1 mm) wood structural panels with maximum spans of 8 feet (2438 mm) as an impact-resistant covering for one- and two-story buildings. Panel attachments have to be designed to resist the component and cladding pressure from ASCE 7 or attached in accordance with Table 1609.1.2. This protective system has been tested and meets the requirements of SBCCI SSTD 12. The intent is that precut panel coverings and attachment hardware are provided on site.

Exception 2 exempts low-hazard (Risk Category I) buildings from this requirement for protecting openings against wind-borne debris.

Exception 3 exempts openings in buildings that are classified in Risk Category II, III or IV where the opening locations meet the stated conditions.

TABLE 1609.1.2. See page 16-46.

❖ This table provides the connections for the wood structural panel impact-resistant covering that is described in the exception to Section 1609.1.2. The table lists the spacing of the screws around the perimeter of the panel for the indicated panel spans. Note that Table 1609.1.2 is only applicable to buildings with a mean roof height of 33 feet (10 058 mm) or less and located where the basic wind speed is 130 mph or less.

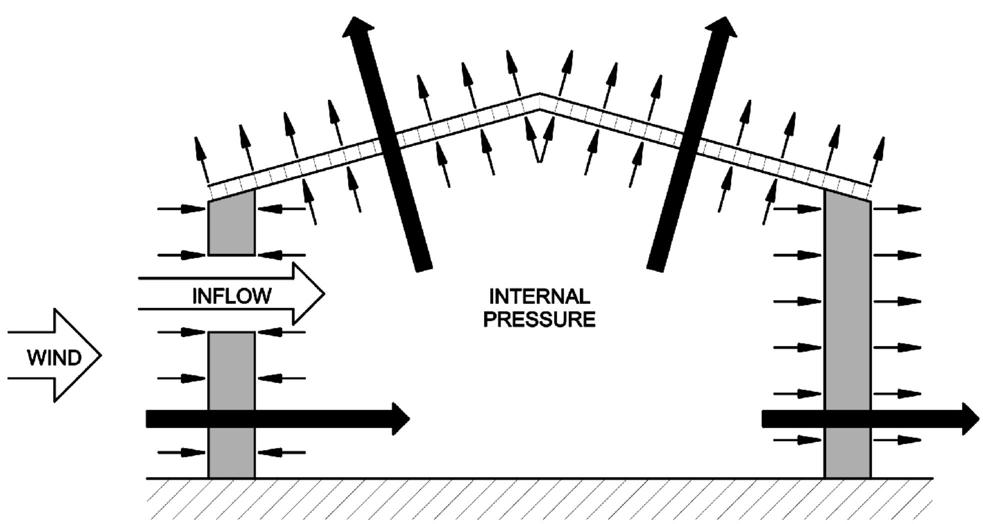


Figure 1609.1.2
EFFECTS OF OPENINGS IN THE BUILDING ENVELOPE

TABLE 1609.1.2
WIND-BORNE DEBRIS PROTECTION FASTENING SCHEDULE FOR WOOD STRUCTURAL PANELS[a, b, c, d]

FASTENER TYPE	FASTENER SPACING (inches)		
	Panel Span ≤ 4 feet	4 feet < Panel Span ≤ 6 feet	6 feet < Panel Span ≤ 8 feet
No. 8 wood-screw-based anchor with 2-inch embedment length	16	10	8
No. 10 wood-screw-based anchor with 2-inch embedment length	16	12	9
$^{1}/_{4}$-inch diameter lag-screw-based anchor with 2-inch embedment length	16	16	16

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound = 4.448 N, 1 mile per hour = 0.447 m/s.

a. This table is based on 140 mph wind speeds and a 45-foot mean roof height.

b. Fasteners shall be installed at opposing ends of the wood structural panel. Fasteners shall be located a minimum of 1 inch from the edge of the panel.

c. Anchors shall penetrate through the exterior wall covering with an embedment length of 2 inches minimum into the building frame. Fasteners shall be located a minimum of $2^{1}/_{2}$ inches from the edge of concrete block or concrete.

d. Where panels are attached to masonry or masonry/stucco, they shall be attached using vibration-resistant anchors having a minimum ultimate withdrawal capacity of 1,500 pounds.

1609.1.2.1 Louvers. Louvers protecting intake and exhaust ventilation ducts not assumed to be open that are located within 30 feet (9144 mm) of grade shall meet the requirements of AMCA 54.

❖ This section provides direction for impact testing of louvers that cover intake or exhaust duct openings in wind-borne debris regions. Louvers often have blades or slats affixed to and covering an opening in the exterior envelope, making them similar to certain types of porous shutters used to protect glazed openings. When a louver in an exterior wall is damaged by wind-borne debris during a high-wind event, the air-leakage-rated damper inside the ventilation duct may also be exposed to damage.

The scope of ASTM E 1996 covers impact testing of exterior building features, such as windows, glazed curtain walls, doors and storm shutters, in buildings located in geographic regions that are prone to hurricanes, simulating impact by both large and small missiles. For glazed openings and nonporous shutters that protect the fenestration assembly, the impact testing is followed by a cyclic loading test. There is no specific provision in the standard for testing louvers that cover ventilation openings, either for impact or air pressure cycling. Nevertheless, in the absence of an appropriate specification and test method for louvers, some jurisdictions have based their product approvals on the performance criteria of the large missile test of ASTM E 1886 and ASTM E 1996. The Air Movement Control Association (AMCA) has developed a standard specification for louvers that provides a uniform set of guidelines and a consistent basis for evaluating the ability of the louver to maintain its integrity during the large missile test of ASTM E 1886 and ASTM E 1996.

1609.1.2.2 Application of ASTM E 1996. The text of Section 6.2.2 of ASTM E 1996 shall be substituted as follows:

6.2.2 Unless otherwise specified, select the wind zone based on the strength design wind speed, V_{ult}, as follows:

6.2.2.1 *Wind Zone 1*—130 mph ≤ ultimate design wind speed, V_{ult} < 140 mph.

6.2.2.2 *Wind Zone 2*—140 mph ≤ ultimate design wind speed, V_{ult} < 150 mph at greater than one mile (1.6 km) from the coastline. The coastline shall be measured from the mean high water mark.

6.2.2.3 *Wind Zone 3*—150 mph (58 m/s) ≤ ultimate design wind speed, V_{ult} ≤ 160 mph (63 m/s), or 140 mph (54 m/s) ≤ ultimate design wind speed, V_{ult} ≤ 160 mph (63 m/s) and within one mile(1.6 km) of the coastline. The coastline shall be measured from the mean high water mark.

6.2.2.4 *Wind Zone 4*— ultimate design wind speed, V_{ult} >160 mph (63 m/s).

❖ The purpose of this section is to correlate the wind zones of ASTM E 1996 with the new wind speed maps. It is needed to delineate the wind zones because ASTM E 1996 does not use V_{ult}.

1609.1.2.3 Garage doors. Garage door glazed opening protection for wind-borne debris shall meet the requirements of an *approved* impact-resisting standard or ANSI/DASMA 115.

❖ This provision references a standard for the wind-borne debris resistance testing of glazing installed in garage doors. Because ASTM E 1886 and ASTM E 1996 require interpretation regarding their application to garage doors, DASMA 115 is the primary standard referenced for this purpose.

1609.2 Definitions. For the purposes of Section 1609 and as used elsewhere in this code, the following terms are defined in Chapter 2.

❖ Definitions facilitate the understanding of code provisions and minimize potential confusion. To that end, this section lists definitions of terms associated with wind loads. Note that these definitions are found in Chapter 2. The use and application of defined terms, as well as undefined terms, are set forth in Section 201.

HURRICANE-PRONE REGIONS.

WIND-BORNE DEBRIS REGION.

WIND SPEED, V_{ult}.

WIND SPEED, V_{asd}.

1609.3 Basic wind speed. The ultimate design wind speed, V_{ult}, in mph, for the determination of the wind loads shall be determined by Figures 1609A, 1609B and 1609C. The ultimate design wind speed, V_{ult}, for use in the design of Risk Category II buildings and structures shall be obtained from Figure 1609A. The ultimate design wind speed, V_{ult}, for use in the design of Risk Category III and IV buildings and structures shall be obtained from Figure 1609B. The ultimate design wind speed, V_{ult}, for use in the design of Risk Category I buildings and structures shall be obtained from Figure 1609C. The ultimate design wind speed, V_{ult}, for the special wind regions indicated near mountainous terrain and near gorges shall be in accordance with local jurisdiction requirements. The ultimate design wind speeds, V_{ult}, determined by the local jurisdiction shall be in accordance with Section 26.5.1 of ASCE 7.

In nonhurricane-prone regions, when the ultimate design wind speed, V_{ult}, is estimated from regional climatic data, the ultimate design wind speed, V_{ult}, shall be determined in accordance with Section 26.5.3 of ASCE 7.

❖ This section establishes the wind speed that is to be used for design. The ultimate design wind speed maps identify special wind regions where the speeds vary substantially within a very short distance due to topographic effects, such as mountains and valleys. This section specifies appropriate recurrence interval criteria to be used for estimating the ultimate design wind speeds from regional climatic data in other than hurricane- prone regions. See Section 26.5.3 of ASCE 7 for issues to be addressed in the data analysis and the associated commentary for further details (also see commentary, Figures 1609A, 1609B and 1609C).

1609.3.1 Wind speed conversion. When required, the ultimate design wind speeds of Figures 1609A, 1609B and 1609C shall be converted to nominal design wind speeds, V_{asd}, using Table 1609.3.1 or Equation 16-33.

$$V_{asd} = V_{ult}\sqrt{0.6} \qquad \textbf{(Equation 16-33)}$$

where:

V_{asd} = nominal design wind speed applicable to methods specified in Exceptions 1 through 5 of Section 1609.1.1.

V_{ult} = ultimate design wind speeds determined from Figures 1609A, 1609B or 1609C.

❖ Because many code provisions are driven by the wind speed, it is necessary to include a mechanism that provides a comparable wind speed so that the provisions triggered by wind speed are not affected. The terms “ultimate design wind speed” and “nominal design wind speed” have been incorporated in numerous locations to help the code user distinguish between them. For example, in a case where the code previously imposed requirements where the basic wind speed exceeds 100 mph (45 m/s), it now imposes the requirements where V_{asd} exceeds 100 mph (45 m/s). A nominal design speed, V_{asd}, equal to 100 mph (45 m/s) corresponds to an ultimate design wind speed, V_{ult}, equal to 129 mph (58 m/s). The conversion between the two is accomplished using Equation 16-32 or Table 1609.3.1. This conversion equation is the result of the wind load being proportional to the square of the velocity pressure and the ASD wind load being 0.6 times the strength level wind load.

It should be noted that the term “basic wind speed” remains in ASCE 7, but it corresponds to the “ultimate design wind speed” in the code. For a comparison of ASCE 7-93 fastest-mile wind speeds and ASCE 7-05 3-second gust (ASD) wind speeds to ASCE 7-10 3-second gust wind speeds, refer to Table C26.5-6 in the ASCE 7-10.

TABLE 1609.3.1. See below.

❖ The table converts the mapped ultimate wind speed to nominal design wind speed for use where necessary, such as in the standards that are referenced in Section 1609.1.1.

1609.4 Exposure category. For each wind direction considered, an exposure category that adequately reflects the characteristics of ground surface irregularities shall be determined for the site at which the building or structure is to be constructed. Account shall be taken of variations in ground surface roughness that arise from natural topography and vegetation as well as from constructed features.

❖ The concept of exposure categories provides a means of accounting for the relative roughness of the boundary layer. The earth’s surface exerts a horizontal drag force on wind due to ground obstructions that retard the flow of air close to the ground. The reduction in the flow of air is a function of height above ground and terrain roughness. Wind speeds increase with height above ground, and the relationship between height above ground and wind speed is exponential. The rate of increase in wind speeds with height is a function of the terrain features. The rougher the terrain (such as large city centers), the shallower the slope of the wind speed profile. The

TABLE 1609.3.1
WIND SPEED CONVERSIONS[a, b, c]

V_{ult}	100	110	120	130	140	150	160	170	180	190	200
V_{asd}	78	85	93	101	108	116	124	132	139	147	155

For SI: 1 mile per hour = 0.44 m/s.
a. Linear interpolation is permitted.
b. V_{asd} = nominal design wind speed applicable to methods specified in Exceptions 1 through 5 of Section 1609.1.1.
c. V_{ult} = ultimate design wind speeds determined from Figures 1609A, 1609B, or 1609C.

smoother the terrain (open water), the steeper the slope of the wind speed profile.

The definitions of exposure and roughness categories are used to account for this roughness in the boundary layer and are intended to provide an adequate assessment of surface roughness for most situations. Exposure B is considered the roughest boundary layer condition. Exposure D is considered the smoothest boundary layer condition. Accordingly, calculated wind loads are less for Exposure B, which has more surface obstructions, as compared to Exposure D, with all other variables the same (see ASCE 7 commentary for guidance on performing a more detailed analysis of surface roughness).

1609.4.1 Wind directions and sectors. For each selected wind direction at which the wind loads are to be evaluated, the exposure of the building or structure shall be determined for the two upwind sectors extending 45 degrees (0.79 rad) either side of the selected wind direction. The exposures in these two sectors shall be determined in accordance with Sections 1609.4.2 and 1609.4.3 and the exposure resulting in the highest wind loads shall be used to represent winds from that direction.

❖ An exposure category must be determined for each direction in which wind loading is being considered. This section describes the method for doing so, and requires use of the more restrictive exposure category where the two sectors in a given wind direction would be classified in different exposure categories.

1609.4.2 Surface roughness categories. A ground surface roughness within each 45-degree (0.79 rad) sector shall be determined for a distance upwind of the site as defined in Section 1609.4.3 from the categories defined below, for the purpose of assigning an exposure category as defined in Section 1609.4.3.

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 feet (9144 mm). 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.

❖ This section defines three surface roughness categories that are used in evaluating each sector and, subsequently, in determining an exposure category. These surface roughnesses were previously included in the corresponding exposure category definition. The required upwind distance that must be considered varies according to the exposure category definition in Section 1609.4.3.

A significant philosophical change in determination of exposure categories occurred with the inclusion of shorelines in hurricane-prone regions in the definition of Exposure Category C (Surface Roughness C). Exposure Category D (Surface Roughness D) had been used for wind flowing over open water until further research determined that wave action at the water's surface in a hurricane, due to the intensity of the turbulence, produced substantial surface obstructions and friction that reduces the wind profile values to be more in line with Surface Roughness C as opposed to Surface Roughness D. Surface Roughness D would still apply to inland waterways and shorelines that are not in the hurricane-prone regions, such as coastal California, Oregon, Washington and Alaska.

1609.4.3 Exposure categories. An exposure category shall be determined in accordance with the following:

Exposure B. For buildings with a mean roof height of less than or equal to 30 feet (9144 mm), Exposure B shall apply where the ground surface roughness, as defined by Surface Roughness B, prevails in the upwind direction for a distance of at least 1,500 feet (457 m). For buildings with a mean roof height greater than 30 feet (9144 mm), Exposure B shall apply where Surface Roughness B prevails in the upwind direction for a distance of at least 2,600 feet (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 of at least 5,000 feet (1524 m) or 20 times the height of the building, 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 feet (183 m) or 20 times the building height, whichever is greater, from an exposure D condition as defined in the previous sentence.

❖ This section defines the three exposure categories that are used to determine various wind load parameters. Exposure categories are used along with basic wind speed elsewhere in the code as a threshold for wind requirements, such as establishing the need for structural observation in Section 1704.5.

Exposure B is the most common type of exposure category in the country. A study by the National Association of Home Builders (NAHB) indicated that perhaps up to 80 percent of all buildings were located in Exposure B.

1609.5 Roof systems. Roof systems shall be designed and constructed in accordance with Sections 1609.5.1 through 1609.5.3, as applicable.

❖ This section clarifies the design wind loads that are applied to roof decks and roof coverings.

1609.5.1 Roof deck. The roof deck shall be designed to withstand the wind pressures determined in accordance with ASCE 7.

❖ This section specifies the wind load criteria for the roof deck. The roof deck is a structural component of the building and must resist the applicable wind pressures of ASCE 7. This section is referenced by Section 1609.5.2 as the criteria for the wind design for roof coverings that are relatively impermeable.

1609.5.2 Roof coverings. Roof coverings shall comply with Section 1609.5.1.

Exception: Rigid tile roof coverings that are air permeable and installed over a roof deck complying with Section 1609.5.1 are permitted to be designed in accordance with Section 1609.5.3.

Asphalt shingles installed over a roof deck complying with Section 1609.5.1 shall comply with the wind-resistance requirements of Section 1507.2.7.1.

❖ This section establishes the wind design criteria for roof coverings. The exception references the use of Section 1609.5.3 for air-permeable rigid tile roof coverings. If the roof deck is relatively impermeable, wind pressures will act through it to the building frame system. The roof covering may or may not be subjected to the same wind pressures as the roof deck. If the roof covering is also relatively impermeable and fastened to the roof deck, the two components will react to and resist the same wind pressures. If the roof covering is not impermeable, the wind pressures will be able to develop on both the top of, and underneath, the roof covering. This "venting action" will negate some wind pressure on the roof covering.

1609.5.3 Rigid tile. Wind loads on rigid tile roof coverings shall be determined in accordance with the following equation:

$$M_a = q_h C_L b L L_a[1.0 - GC_p] \qquad \textbf{(Equation 16-34)}$$

For SI: $$M_a = \frac{L(q_h C_L b)L_a[1.0 - GC_p]}{1,000}$$

where:

b = Exposed width, feet (mm) of the roof tile.

C_L = Lift coefficient. The lift coefficient for concrete and clay tile shall be 0.2 or shall be determined by test in accordance with Section 1711.2.

GC_p = Roof pressure coefficient for each applicable roof zone determined from Chapter 30 of ASCE 7. Roof coefficients shall not be adjusted for internal pressure.

L = Length, feet (mm) of the roof tile.

L_a = Moment arm, feet (mm) from the axis of rotation to the point of uplift on the roof tile. The point of uplift shall be taken at 0.76L from the head of the tile and the middle of the exposed width. For roof tiles with nails or screws (with or without a tail clip), the axis of rotation shall be taken as the head of the tile for direct deck application or as the top edge of the batten for battened applications. For roof tiles fastened only by a nail or screw along the side of the tile, the axis of rotation shall be determined by testing. For roof tiles installed with battens and fastened only by a clip near the tail of the tile, the moment arm shall be determined about the top edge of the batten with consideration given for the point of rotation of the tiles based on straight bond or broken bond and the tile profile.

M_a = Aerodynamic uplift moment, feet-pounds (N-mm) acting to raise the tail of the tile.

q_h = Wind velocity pressure, psf (kN/m^2) determined from Section 27.3.2 of ASCE 7.

Concrete and clay roof tiles complying with the following limitations shall be designed to withstand the aerodynamic uplift moment as determined by this section.

1. The roof tiles shall be either loose laid on battens, mechanically fastened, mortar set or adhesive set.
2. The roof tiles shall be installed on solid sheathing which has been designed as components and cladding.
3. An underlayment shall be installed in accordance with Chapter 15.
4. The tile shall be single lapped interlocking with a minimum head lap of not less than 2 inches (51 mm).
5. The length of the tile shall be between 1.0 and 1.75 feet (305 mm and 533 mm).
6. The exposed width of the tile shall be between 0.67 and 1.25 feet (204 mm and 381 mm).
7. The maximum thickness of the tail of the tile shall not exceed 1.3 inches (33 mm).
8. Roof tiles using mortar set or adhesive set systems shall have at least two-thirds of the tile's area free of mortar or adhesive contact.

❖ This section includes the wind design method for clay or concrete rigid tile roofs. The method consists of the calculation of the aerodynamic uplift moment from the wind that acts to raise the end of the tile. This section includes the characteristics and the type of installation of the concrete or clay roof tile that are required for the use of the design method.

In certain types of installations, the roof covering is not exposed to the same wind loads as the roof deck. Concrete and clay roof tiles are typical of this type of installation and are not subject to wind loads that would be obtained from current wind-loading criteria. This is due to the gaps at tile joints allowing some equalization of pressure between the inner and outer face of the tiles, leading to reduced loads. A procedure has been developed through research for determining the uplift moment on loose-laid and mechanically fastened roof tiles when laid over sheathing with an underlayment. The procedure is based on practical measurements on real tiles to determine the effect of air being able to penetrate the roof covering.

1609.6 Alternate all-heights method. The alternate wind design provisions in this section are simplifications of the ASCE 7 Directional Procedure.

❖ In response to concerns from design engineers on the complexity of wind design procedures, member organizations of the National Council of Standard Engineering Associations (NCSEA) assembled this alternative method for determining wind loads. The procedure has been developed to give results equal to or more conservative than the Directional Procedure that is found in ASCE 7. The intention is to reduce the effort required in determining wind forces for the main wind-force-resisting system, as well as for components and cladding (C & C).

1609.6.1 Scope. As an alternative to ASCE 7 Chapters 27 and 30, the following provisions are permitted to be used to determine the wind effects on regularly shaped buildings, or other structures that are regularly shaped, which meet all of the following conditions:

1. The building or other structure is less than or equal to 75 feet (22 860 mm) in height with a height-to-least-width ratio of 4 or less, or the building or other structure has a fundamental frequency greater than or equal to 1 hertz.
2. The building or other structure is not sensitive to dynamic effects.
3. The building or other structure is not located on a site for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.
4. The building shall meet the requirements of a simple diaphragm building as defined in ASCE 7 Section 26.2, where wind loads are only transmitted to the main windforce-resisting system (MWFRS) at the diaphragms.
5. For open buildings, multispan gable roofs, stepped roofs, sawtooth roofs, domed roofs, roofs with slopes greater than 45 degrees (0.79 rad), solid free-standing walls and solid signs, and rooftop equipment, apply ASCE 7 provisions.

❖ While ASCE 7 already includes a simplified procedure, it necessarily includes numerous limitations. Similarly, this alternative design procedure is limited, though it was developed to apply to a broader range of buildings.

Note that Item 1 limits application to buildings with a frequency of at least 1 hertz. In other words, this can only be used for rigid structures (see definition of "Rigid buildings and other structures" in Section 26.2 of ASCE 7). Item 1 also allows any building up to 75 feet (22 860 mm) in height that has a height-to-least-width ratio of 4 or less to qualify without calculating the frequency. Buildings must also be regularly shaped, simple diaphragm buildings with envelopes classified as either enclosed or partially enclosed. Item 5 provides a partial list of structures that must be analyzed using the ASCE 7 provisions.

1609.6.1.1 Modifications. The following modifications shall be made to certain subsections in ASCE 7: in Section 1609.6.2, symbols and notations that are specific to this section are used in conjunction with the symbols and notations in ASCE 7 Section 26.3.

❖ These alternative provisions are an adaptation of the ASCE 7 analytical procedure. In providing the intended simplifications, this section alerts the code user that these are modifications to the ASCE 7 procedure.

1609.6.2 Symbols and notations. Coefficients and variables used in the alternative all-heights method equations are as follows:

C_{net} = Net-pressure coefficient based on K_d [(G) (C_p) - (GC_{pi})], in accordance with Table 1609.6.2.

G = Gust effect factor for rigid structures in accordance with ASCE 7 Section 26.9.1.

K_d = Wind directionality factor in accordance with ASCE 7 Table 26-6.

P_{net} = Design wind pressure to be used in determination of wind loads on buildings or other structures or their components and cladding, in psf (kN/m^2).

❖ The primary simplification is accomplished by generating a table of net pressure coefficients (C_{net}), combining a number of parameters in a simple, yet conservative manner. These are shown in the definition of notation C_{net}. Application of the tabulated net pressure coefficients reduces the number of steps required for performing a wind analysis, resulting in net wind forces that meet or exceed those calculated using the Directional Procedure of ASCE 7. A gust factor of 0.85 is used for the tabulated C_{net} values, as Section 26.9.2 of ASCE 7 permits for rigid structures.

TABLE 1609.6.2. See page 16-52.

❖ Net pressure coefficients are tabulated for both enclosed and partially enclosed structures (see Section 26.10 of ASCE 7).

For main windforce-resisting system roofs Table 1609.6.2 refers to Conditions 1 and 2 for "windward roof slopes," which correspond to the two values of C_p listed in Figure 27.4.1-1 of ASCE 7 (Note 3 at the bottom of that figure also refers to "both conditions").

The basis is $C_{net} = K_d$ [(G) (C_p) – (GC_{pi})] (see Section 1609.6.2) and the values obtained for C_p. When considering the wind loads on the "windward roof slope," see Figure 27.4.1-1 of ASCE 7. On the windward side of a sloping roof for "Gable, hip roof," there are two wind load conditions, one "upward" and the other "downward" as indicated by the two arrows. Two conditions on the windward roof slope provide two values of C_p, and thereby two values of C_{net} shown in Table 1609.6.2.

1609.6.3 Design equations. When using the alternative all-heights method, the MWFRS, and components and cladding of every structure shall be designed to resist the effects of wind pressures on the building envelope in accordance with Equation 16-35.

$$P_{net} = 0.00256V^2K_zC_{net}K_{zt} \qquad \textbf{(Equation 16-35)}$$

Design wind forces for the MWFRS shall not be less than 16 psf (0.77 kN/m^2) multiplied by the area of the structure projected on a plane normal to the assumed wind direction (see ASCE 7 Section 27.4.7 for criteria). Design net wind pressure for components and cladding shall not be less than 16 psf (0.77 kN/m^2) acting in either direction normal to the surface.

❖ This section provides the formula that is used for the design wind pressure. It also incorporates the ASCE 7 minimum wind pressure for main wind-force-resisting systems and components and cladding.

1609.6.4 Design procedure. The MWFRS and the components and cladding of every building or other structure shall be designed for the pressures calculated using Equation 16-35.

❖ Using Equation 16-35, the design pressures can be calculated for the main wind-force-resisting system. Similarly, the components and cladding design pressures are calculated for the various portions of the building envelope.

1609.6.4.1 Main windforce-resisting systems. The MWFRS shall be investigated for the torsional effects identified in ASCE 7 Figure 27.4.6.

❖ A reference is made to the ASCE 7 figure that illustrates the design wind load cases that must be considered, specifically making mention of the torsional load cases.

1609.6.4.2 Determination of K_z and K_{zt}. Velocity pressure exposure coefficient, K_z, shall be determined in accordance with ASCE 7 Section 27.3.1 and the topographic factor, K_{zt}, shall be determined in accordance with ASCE 7 Section 26.8.

1. For the windward side of a structure, K_{zt} and K_z shall be based on height *z*.
2. For leeward and sidewalls, and for windward and leeward roofs, K_{zt} and K_z shall be based on mean roof height *h*.

❖ For the velocity pressure exposure coefficient, the user is referred to the corresponding ASCE 7 section. For the topographic factor, the user is referred to the corresponding ASCE 7 section. Note that these are evaluated only at the mean roof height for leeward walls, sidewalls and roofs.

1609.6.4.3 Determination of net pressure coefficients, C_{net}. For the design of the MWFRS and for components and cladding, the sum of the internal and external net pressure shall be based on the net pressure coefficient, C_{net}.

1. The pressure coefficient, $C_{net,}$ for walls and roofs shall be determined from Table 1609.6.2.
2. Where C_{net} has more than one value, the more severe wind load condition shall be used for design.

❖ The tabulated C_{net} values represent the sum of external and internal pressure coefficients as shown in the notation defined in Section 1609.6.2.

1609.6.4.4 Application of wind pressures. When using the alternative all-heights method, wind pressures shall be applied simultaneously on, and in a direction normal to, all building envelope wall and roof surfaces.

❖ This section clarifies how the wind pressures are applied relative to the surfaces of the building envelope.

1609.6.4.4.1 Components and cladding. Wind pressure for each component or cladding element is applied as follows using C_{net} values based on the effective wind area, *A*, contained within the zones in areas of discontinuity of width and/or length "a," "2a" or "4a" at: corners of roofs and walls; edge strips for ridges, rakes and eaves; or field areas on walls or roofs as indicated in figures in tables in ASCE 7 as referenced in Table 1609.6.2 in accordance with the following:

1. Calculated pressures at local discontinuities acting over specific edge strips or corner boundary areas.
2. Include "field" (Zone 1, 2 or 4, as applicable) pressures applied to areas beyond the boundaries of the areas of discontinuity.
3. Where applicable, the calculated pressures at discontinuities (Zone 2 or 3) shall be combined with design pressures that apply specifically on rakes or eave overhangs.

❖ The C_{net} values for C & C pressures are separately tabulated for roof and walls. These portions of the structure are further divided into those that are in areas of discontinuities and those that are not. Effective wind area is another factor in selecting the correct C & C value. While the lateral-load-resisting system design may be controlled by earthquake forces, it is necessary to consider these C & C wind pressures, which can control the design of the component and attachments.

TABLE 1609.6.2
NET PRESSURE COEFFICIENTS, C_{net}[a, b]

STRUCTURE OR PART THEREOF	DESCRIPTION		C_{net} FACTOR			
			Enclosed		Partially enclosed	
	Walls:		+ Internal pressure	- Internal pressure	+ Internal pressure	- Internal pressure
	Windward wall		0.43	0.73	0.11	1.05
	Leeward wall		-0.51	-0.21	-0.83	0.11
	Sidewall		-0.66	-0.35	-0.97	-0.04
	Parapet wall	Windward	1.28		1.28	
		Leeward	-0.85		-0.85	
	Roofs:		Enclosed		Partially enclosed	
	Wind perpendicular to ridge		+ Internal pressure	- Internal pressure	+ Internal pressure	- Internal pressure
	Leeward roof or flat roof		-0.66	-0.35	-0.97	-0.04
	Windward roof slopes:					
	Slope < 2:12 (10°)	Condition 1	-1.09	-0.79	-1.41	-0.47
		Condition 2	-0.28	0.02	-0.60	0.34
	Slope = 4:12 (18°)	Condition 1	-0.73	-0.42	-1.04	-0.11
		Condition 2	-0.05	0.25	-0.37	0.57
	Slope = 5:12 (23°)	Condition 1	-0.58	-0.28	-0.90	0.04
		Condition 2	0.03	0.34	-0.29	0.65
1. Main windforce-resisting frames and systems	Slope = 6:12 (27°)	Condition 1	-0.47	-0.16	-0.78	0.15
		Condition 2	0.06	0.37	-0.25	0.68
	Slope = 7:12 (30°)	Condition 1	-0.37	-0.06	-0.68	0.25
		Condition 2	0.07	0.37	-0.25	0.69
	Slope = 9:12 (37°)	Condition 1	-0.27	0.04	-0.58	0.35
		Condition 2	0.14	0.44	-0.18	0.76
	Slope = 12:12 (45°)		0.14	0.44	-0.18	0.76
	Wind parallel to ridge and flat roofs		-1.09	-0.79	-1.41	-0.47
	Nonbuilding Structures: Chimneys, Tanks and Similar Structures:					
				h/D		
				1	7	25
	Square (Wind normal to face)			0.99	1.07	1.53
	Square (Wind on diagonal)			0.77	0.84	1.15
	Hexagonal or Octagonal			0.81	0.97	1.13
	Round			0.65	0.81	0.97
	Open signs and lattice frameworks			Ratio of solid to gross area		
				< 0.1	0.1 to 0.29	0.3 to 0.7
	Flat			1.45	1.30	1.16
	Round			0.87	0.94	1.08

(continued)

TABLE 1609.6.2—continued
NET PRESSURE COEFFICIENTS, C_{net}[a, b]

STRUCTURE OR PART THEREOF	DESCRIPTION		C_{net} FACTOR	
2. Components and cladding not in areas of discontinuity—roofs and overhangs	**Roof elements and slopes**		**Enclosed**	**Partially enclosed**
	Gable of hipped configurations (Zone 1)			
	Flat < Slope < 6:12 (27°) See ASCE 7 Figure 30.4-2B Zone 1			
	Positive	10 square feet or less	0.58	0.89
		100 square feet or more	0.41	0.72
	Negative	10 square feet or less	-1.00	-1.32
		100 square feet or more	-0.92	-1.23
	Overhang: Flat < Slope < 6:12 (27°) See ASCE 7 Figure 30.4-2A Zone 1			
	Negative	10 square feet or less	-1.45	
		100 square feet or more	-1.36	
		500 square feet or more	-0.94	
	6:12 (27°) < Slope < 12:12 (45°) See ASCE 7 Figure 30.4-2C Zone 1			
	Positive	10 square feet or less	0.92	1.23
		100 square feet or more	0.83	1.15
	Negative	10 square feet or less	-1.00	-1.32
		100 square feet or more	-0.83	-1.15
	Monosloped configurations (Zone 1)		**Enclosed**	**Partially enclosed**
	Flat < Slope < 7:12 (30°) See ASCE 7 Figure 30.4-5B Zone 1			
	Positive	10 square feet or less	0.49	0.81
		100 square feet or more	0.41	0.72
	Negative	10 square feet or less	-1.26	-1.57
		100 square feet or more	-1.09	-1.40
	Tall flat-topped roofs *h* > 60 feet		**Enclosed**	**Partially enclosed**
	Flat < Slope < 2:12 (10°) (Zone 1) See ASCE 7 Figure 30.8-1 Zone 1			
	Negative	10 square feet or less	-1.34	-1.66
		500 square feet or more	-0.92	-1.23
3. Components and cladding in areas of discontinuities—roofs and overhangs (continued)	Gable or hipped configurations at ridges, eaves and rakes (Zone 2)			
	Flat < Slope < 6:12 (27°) See ASCE 7 Figure 30.4-2B Zone 2			
	Positive	10 square feet or less	0.58	0.89
		100 square feet or more	0.41	0.72
	Negative	10 square feet or less	-1.68	-2.00
		100 square feet or more	-1.17	-1.49
	Overhang for Slope Flat < Slope < 6:12 (27°) See ASCE 7 Figure 30.4-2B Zone 2			
	Negative	10 square feet or less	-1.87	
		100 square feet or more	-1.87	
	6:12 (27°) < Slope < 12:12 (45°) Figure 30.4-2C		**Enclosed**	**Partially enclosed**
	Positive	10 square feet or less	0.92	1.23
		100 square feet or more	0.83	1.15
	Negative	10 square feet or less	-1.17	-1.49
		100 square feet or more	-1.00	-1.32
	Overhang for 6:12 (27°) < Slope < 12:12 (45°) See ASCE 7 Figure 30.4-2C Zone 2			
	Negative	10 square feet or less	-1.70	
		500 square feet or more	-1.53	

(continued)

TABLE 1609.6.2—continued
NET PRESSURE COEFFICIENTS, C_{net}[a, b]

STRUCTURE OR PART THEREOF	DESCRIPTION		C_{net} FACTOR	
3. Components and cladding in areas of discontinuities—roofs and overhangs	**Roof elements and slopes**		**Enclosed**	**Partially enclosed**
	Monosloped configurations at ridges, eaves and rakes (Zone 2)			
	Flat < Slope < 7:12 (30°) See ASCE 7 Figure 30.4-5B Zone 2			
	Positive	10 square feet or less	0.49	0.81
		100 square feet or more	0.41	0.72
	Negative	10 square feet or less	-1.51	-1.83
		100 square feet or more	-1.43	-1.74
	Tall flat topped roofs h > 60 feet		**Enclosed**	**Partially enclosed**
	Flat < Slope < 2:12 (10°) (Zone 2) See ASCE 7 Figure 30.8-1 Zone 2			
	Negative	10 square feet or less	-2.11	-2.42
		500 square feet or more	-1.51	-1.83
	Gable or hipped configurations at corners (Zone 3) See ASCE 7 Figure 30.4-2B Zone 3			
	Flat < Slope < 6:12 (27°)		**Enclosed**	**Partially enclosed**
	Positive	10 square feet or less	0.58	0.89
		100 square feet or more	0.41	0.72
	Negative	10 square feet or less	-2.53	-2.85
		100 square feet or more	-1.85	-2.17
	Overhang for Slope Flat < Slope < 6:12 (27°) See ASCE 7 Figure 30.4-2B Zone 3			
	Negative	10 square feet or less	-3.15	
		100 square feet or more	-2.13	
	6:12 (27°) < 12:12 (45°) See ASCE 7 Figure 30.4-2C Zone 3			
	Positive	10 square feet or less	0.92	1.23
		100 square feet or more	0.83	1.15
	Negative	10 square feet or less	-1.17	-1.49
		100 square feet or more	-1.00	-1.32
	Overhang for 6:12 (27°) < Slope < 12:12 (45°)		**Enclosed**	**Partially enclosed**
	Negative	10 square feet or less	-1.70	
		100 square feet or more	-1.53	
	Monosloped Configurations at corners (Zone 3) See ASCE 7 Figure 30.4-5B Zone 3			
	Flat < Slope < 7:12 (30°)			
	Positive	10 square feet or less	0.49	0.81
		100 square feet or more	0.41	0.72
	Negative	10 square feet or less	-2.62	-2.93
		100 square feet or more	-1.85	-2.17
	Tall flat topped roofs h > 60 feet		**Enclosed**	**Partially enclosed**
	Flat < Slope < 2:12 (10°) (Zone 3) See ASCE 7 Figure 30.8-1 Zone 3			
	Negative	10 square feet or less	-2.87	-3.19
		500 square feet or more	-2.11	-2.42
4. Components and cladding not in areas of discontinuity—walls and parapets (continued)	Wall Elements: h = 60 feet (Zone 4) Figure 30.4-1		**Enclosed**	**Partially enclosed**
	Positive	10 square feet or less	1.00	1.32
		500 square feet or more	0.75	1.06
	Negative	10 square feet or less	-1.09	-1.40
		500 square feet or more	-0.83	-1.15
	Wall Elements: h > 60 feet (Zone 4) See ASCE 7 Figure 30.8-1 Zone 4			
	Positive	20 square feet or less	0.92	1.23
		500 square feet or more	0.66	0.98

(continued)

TABLE 1609.6.2—continued
NET PRESSURE COEFFICIENTS, C_{net} [a, b]

STRUCTURE OR PART THEREOF	DESCRIPTION		C_{net} FACTOR	
4. Components and cladding not in areas of discontinuity-walls and parapets	Negative	20 square feet or less	-0.92	-1.23
		500 square feet or more	-0.75	-1.06
	Parapet Walls			
	Positive		2.87	3.19
	Negative		-1.68	-2.00
5. Components and cladding in areas of discontinuity—walls and parapets	Wall elements: $h \leq 60$ feet (Zone 5) Figure 30.4-1		**Enclosed**	**Partially enclosed**
	Positive	10 square feet or less	1.00	1.32
		500 square feet or more	0.75	1.06
	Negative	10 square feet or less	-1.34	-1.66
		500 square feet or more	-0.83	-1.15
	Wall elements: $h > 60$ feet (Zone 5) See ASCE 7 Figure 30.8-1 Zone 4			
	Positive	20 square feet or less	0.92	1.23
		500 square feet or more	0.66	0.98
	Negative	20 square feet or less	-1.68	-2.00
		500 square feet or more	-1.00	-1.32
	Parapet walls			
	Positive		3.64	3.95
	Negative		-2.45	-2.76

For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929m^2, 1 degree = 0.0175 rad.

a. Linear interpolation between values in the table is permitted.

b. Some C_{net} values have been grouped together. Less conservative results may be obtained by applying ASCE 7 provisions.

SECTION 1610 SOIL LATERAL LOADS

1610.1 General. Foundation walls and retaining walls shall be designed to resist lateral soil loads. Soil loads specified in Table 1610.1 shall be used as the minimum design lateral soil loads unless determined otherwise by a geotechnical investigation in accordance with Section 1803. Foundation walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top shall be permitted to be designed for active pressure. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure load. Design lateral pressure shall be increased if soils at the site are expansive. Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 1805.4.2 and 1805.4.3.

Exception: Foundation walls extending not more than 8 feet (2438 mm) below grade and laterally supported at the top by flexible diaphragms shall be permitted to be designed for active pressure.

❖ Nominal loads attributable to lateral earth pressures are determined in accordance with this section. This section provides lateral loads for various soil types. This section requires that foundation and retaining walls be designed to be capable of resisting the lateral soil loads specified by Table 1610.1 where a specific soil investigation has not been performed. Consideration must be given to additional lateral soil pressures due to surcharge loads that result from sloping backfill, driveways or parking spaces that are close to a foundation wall, as well as the foundation of an adjacent structure.

TABLE 1610.1. See page 16-56.

❖ The table lists at-rest and active soil pressures for a number of different types of moist soils. The basis of the soil classification into the various types listed is ASTM D 2487. Soils identified by Note b in Table 1610.1 have unpredictable characteristics. These are called expansive soils. Because of their ability to absorb water, they shrink and swell to a higher degree than other soils. As expansive soils swell, they are capable of exerting large forces on soil-retaining structures; thus, these types of soils are not to be used as backfill.

SECTION 1611 RAIN LOADS

1611.1 Design rain loads. Each portion of a roof shall be designed to sustain the load of 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. The design rainfall shall be based on the 100-year hourly rainfall rate indicated in Figure 1611.1 or on other rainfall rates determined from *approved* local weather data.

$R = 5.2(d_s + d_h)$ **(Equation 16-36)**

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

where:

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

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

R = Rain load on the undeflected roof, in psf (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.

❖ The nominal rain load, *R*, is determined in accordance with this section. It represents the weight of accumulated rainwater, assuming a blockage of the primary roof drainage system. The design of the roof drainage systems must be in accordance with Chapter 11 of the *International Plumbing Code*® (IPC®). The primary roof drainage system can include roof drains, leaders, conductors and horizontal storm drains within the structure. Drainage system design is based on a specified design rainfall intensity, as well as the roof area it drains. The criteria for sizing the components of the drainage system are provided in Section 1106 of the IPC. Where the building is configured such that water will not collect on the roof there is no requirement for a secondary drainage system [see Figure 1611.1(1)]. Likewise, there would be no rain load required in the design of the roof.

It is not uncommon to find that roof drains have become blocked by debris, leading to ponding of rainwater where the roof construction is conducive to retaining water. While the objective of providing roof drainage is typically to prevent the accumulation of water, the code also recognizes controlled drainage systems that are engineered to retain rainwater (see Section 1611.3). The important point is that wherever the potential exists for the accumulation of rainwater on a roof, whether it is intentional or otherwise, the roof must be designed for this load. Furthermore, Section 1101.7 of the IPC requires the maximum depth of water to be determined, assuming all primary roof drainage to be blocked. The water will rise above the primary roof drain until it reaches the elevation of the roof edge, scuppers or another serviceable drain. At the design rainfall intensity, this depth will be based on the flow rate of the secondary drainage system. This depth, referred to as the hydraulic head, can be determined from Table 1611.1(2) for various types of drains and flow rates. Its use is illustrated in the example on page 16-65. Section 1108 of

TABLE 1610.1
LATERAL SOIL LOAD

DESCRIPTION OF BACKFILL MATERIAL[c]	UNIFIED SOIL CLASSIFICATION	DESIGN LATERAL SOIL LOAD[a] (pound per square foot per foot of depth)	
		Active pressure	At-rest pressure
Well-graded, clean gravels; gravel-sand mixes	GW	30	60
Poorly graded clean gravels; gravel-sand mixes	GP	30	60
Silty gravels, poorly graded gravel-sand mixes	GM	40	60
Clayey gravels, poorly graded gravel-and-clay mixes	GC	45	60
Well-graded, clean sands; gravelly sand mixes	SW	30	60
Poorly graded clean sands; sand-gravel mixes	SP	30	60
Silty sands, poorly graded sand-silt mixes	SM	45	60
Sand-silt clay mix with plastic fines	SM-SC	45	100
Clayey sands, poorly graded sand-clay mixes	SC	60	100
Inorganic silts and clayey silts	ML	45	100
Mixture of inorganic silt and clay	ML-CL	60	100
Inorganic clays of low to medium plasticity	CL	60	100
Organic silts and silt clays, low plasticity	OL	Note b	Note b
Inorganic clayey silts, elastic silts	MH	Note b	Note b
Inorganic clays of high plasticity	CH	Note b	Note b
Organic clays and silty clays	OH	Note b	Note b

For SI: 1 pound per square foot per foot of depth = 0.157 kPa/m, 1 foot = 304.8 mm.

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.

b. Unsuitable as backfill material.

c. The definition and classification of soil materials shall be in accordance with ASTM D 2487.

the IPC specifically requires a secondary roof drainage system where the building construction extends above the roof at the perimeter. This applies to parapet walls, stepped buildings or any other construction that would allow rainwater to pond on the roof. The sizing of a secondary drainage system is identical to the process used for the primary system. Instead of using a “piped” secondary system, designers may prefer to install scuppers to allow rainwater to overflow the roof. Examples of both types of secondary systems are shown in Figure 1611.1(3). Also note that the IPC requires a secondary system to be completely separate and to discharge above grade. Since the secondary system serves as an emergency backup, requiring it to discharge above grade provides a means of signaling that there is a blockage of the primary drainage system.

Some roof failures have been attributed to the increased loads from ponding water. This section requires the roof to be capable of resisting the maximum water depth that can occur if the primary means of roof drainage becomes blocked. Blockages are typically caused by debris at the inlet to the primary roof drains, but they can occur anywhere along the primary piping system, such as an under-slab pipe collapse. Computation of rain load, *R*, is in accordance with Equation 16-36. The coefficient of that equation is merely the conversion of the unit weight of water to an equivalent unit load per inch of water depth as Figure 1611.1(4) illustrates. Two variables are considered to determine rain load: the depth of the water on the undeflected roof, as measured from the low point elevation to the inlet elevation of the secondary drain; and the additional depth of water at the secondary drainage flow, respectively referred to as static head and hydraulic head. The sum of these depths is the design depth for computing rain load, *R*, as indicated in Equation 16-36. An example of the

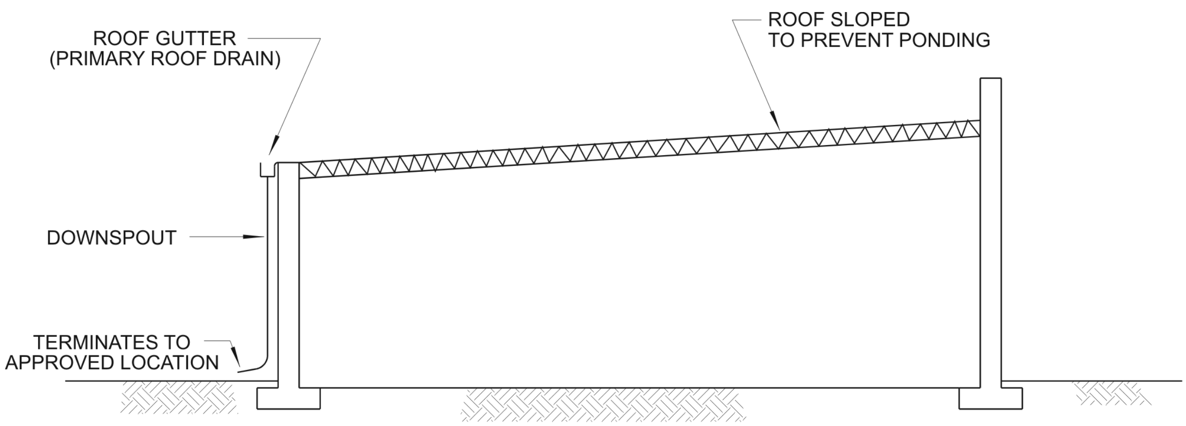


Figure 1611.1(1)
SECONDARY ROOF DRAINAGE NOT REQUIRED

DRAINAGE SYSTEM	FLOW RATE (gpm) Depth of water above drain inlet (hydraulic head) (inches)									
	1	2	2.5	3	3.5	4	4.5	5	7	8
4-inch-diameter drain	80	170	180							
6-inch-diameter drain	100	190	270	380	540					
8-inch-diameter drain	125	230	340	560	850	1,100	1,170			
6-inch-wide, open-top scupper	18	50	*	90	*	140	**	194	321	393
24-inch-wide, open-top scupper	72	200	*	360	*	560	*	776	1,284	1,572
6-inch-wide, 4-inch-high, closed-top scupper	18	50	*	90	*	140	*	177	231	253
24-inch-wide, 4-inch-high, closed-top scupper	72	200	*	360	*	560	*	708	924	1,012
6-inch-wide, 6-inch-high, closed-top scupper	18	50	*	90	*	140	*	194	303	343
24-inch-wide, 6-inch-high, closed-top scupper	72	200	*	360	*	560	*	776	1,212	1,372

For SI: 1 inch = 25.4 mm, 1 gallon per minute = 3.785 L/m.
Source: Factory Mutual Engineering Corp. Loss Prevention Data 1-54.

Table 1611.1(2)
FLOW RATE, IN GALLONS PER MINUTE, OF VARIOUS ROOF DRAINS AT VARIOUS WATER DEPTHS AT DRAIN INLETS (INCHES)

computation of rain load is provided in the following example [also see Figure 1611.1(5)].

EXAMPLE
Rain Load on Roof with Overflow Scuppers

Given:

Primary roof drain and overflow scupper shown in Figure 1611.1(5).

Static head, d_s = 7 inches (178 mm)

Tributary area, A = 5,400 square feet (502 m^2)

Rainfall rate, i = 2.5 inches/hour
= 0.208333 feet/hour (0.0635 m/hr)

Determine:

Hydraulic head, d_h

Rain load, R

Calculate required flow rate, Q, at scupper in gallons per minute (gpm).

$Q = A \times i$ = 5,400 square feet (502 m^2) × 0.2083 feet/hour

= 1125 cubic feet/hour (31.9 m^3/hr)

= 18.75 cubic feet/minute (0.531 m^3/min)

= 140.25 gpm (531 L/m)

Look up hydraulic head using Table 1611.1(2).

For 6 inches wide 6 inches (152 mm) high scupper, find 140 gpm. (530 L/m)

d_h = 4 inches (102 mm)

Determine rain load, R

Total head = $(d_s + d_h)$ = 11 inches (279 mm)

$R = 5.2\,(d_s + d_h)$

R = 57.2 psf (2.74 kN/m^2)

FIGURE 1611.1. See page 16-60.

❖ Including these IPC maps in the code provides the structural designer with design criteria that are essential for determining the rain load. Figure 1611.1 consists of five maps for various regions of the country. This figure provides the rainfall rates for a storm of 1-hour duration that has a 100-year return period. Rainfall rates indicate the maximum rate of rainfall within the given period of time occurring at the stated frequency. For example, the map indicates a rainfall rate of 2.1 inches (53 mm) per hour for Burlington, Vermont. Thus, it is predicted that it will rain 2.1 inches (53 mm) within 1 hour once every 100 years. The rainfall rates are calculated by a statistical analysis of weather records. Because the statistics are based on previous or historical weather conditions, it is conceivable to have two 100-year storms in one week's time. The probability of this occurring, however, is very low.

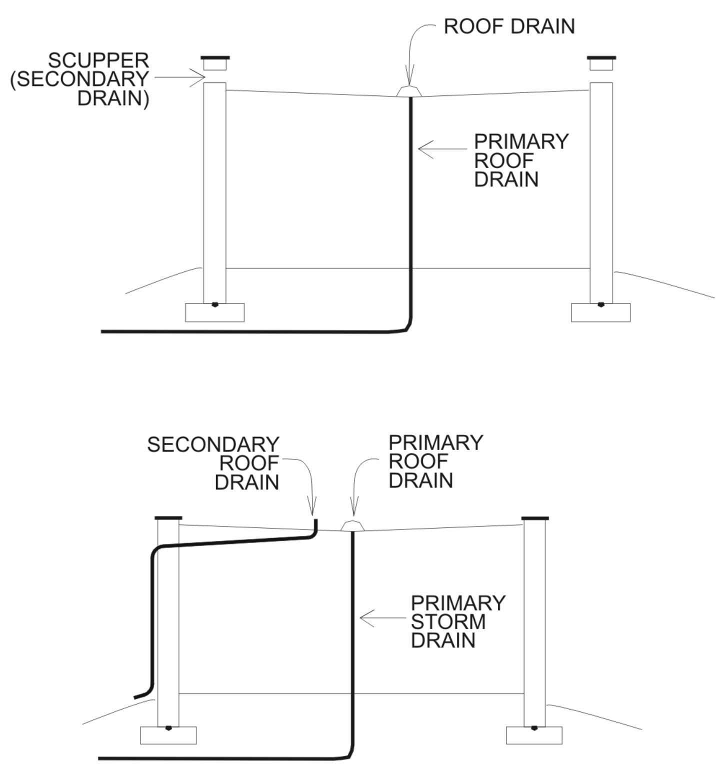
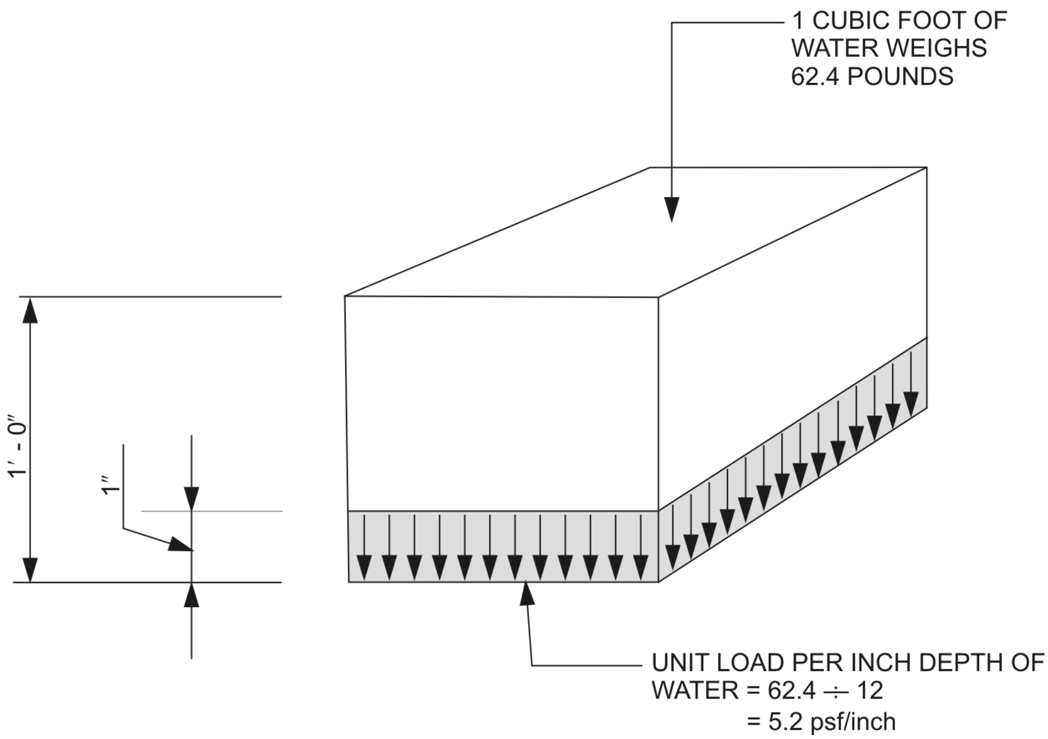
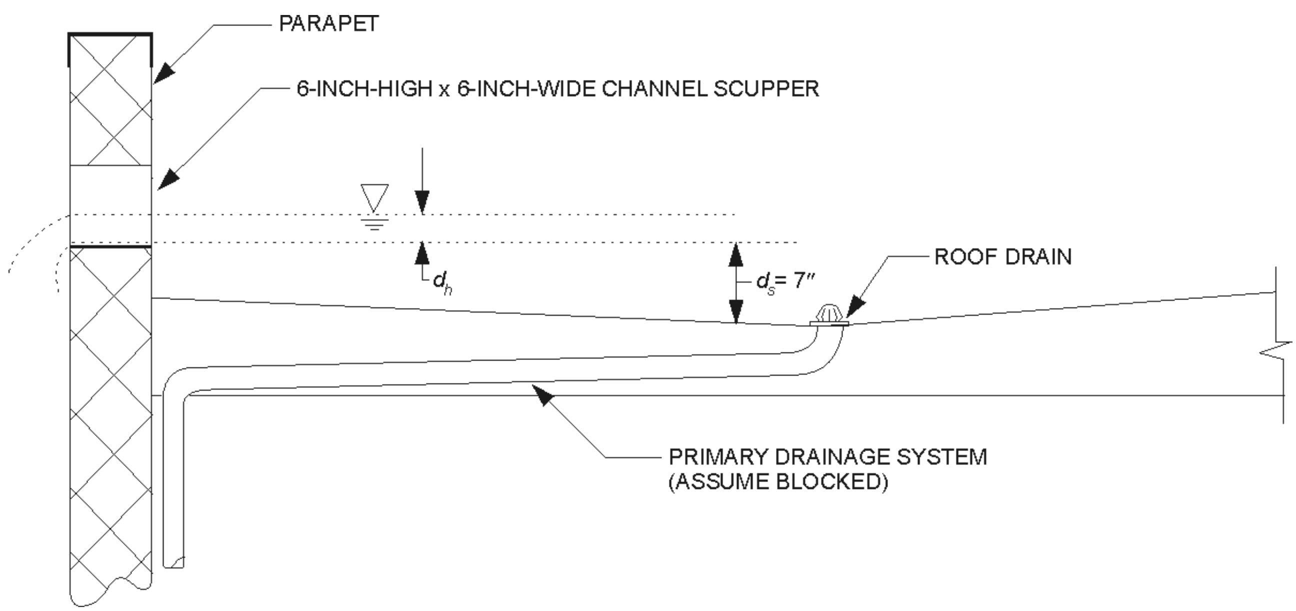


Figure 1611.1(3)
SEPARATE PRIMARY AND SECONDARY ROOF DRAINS



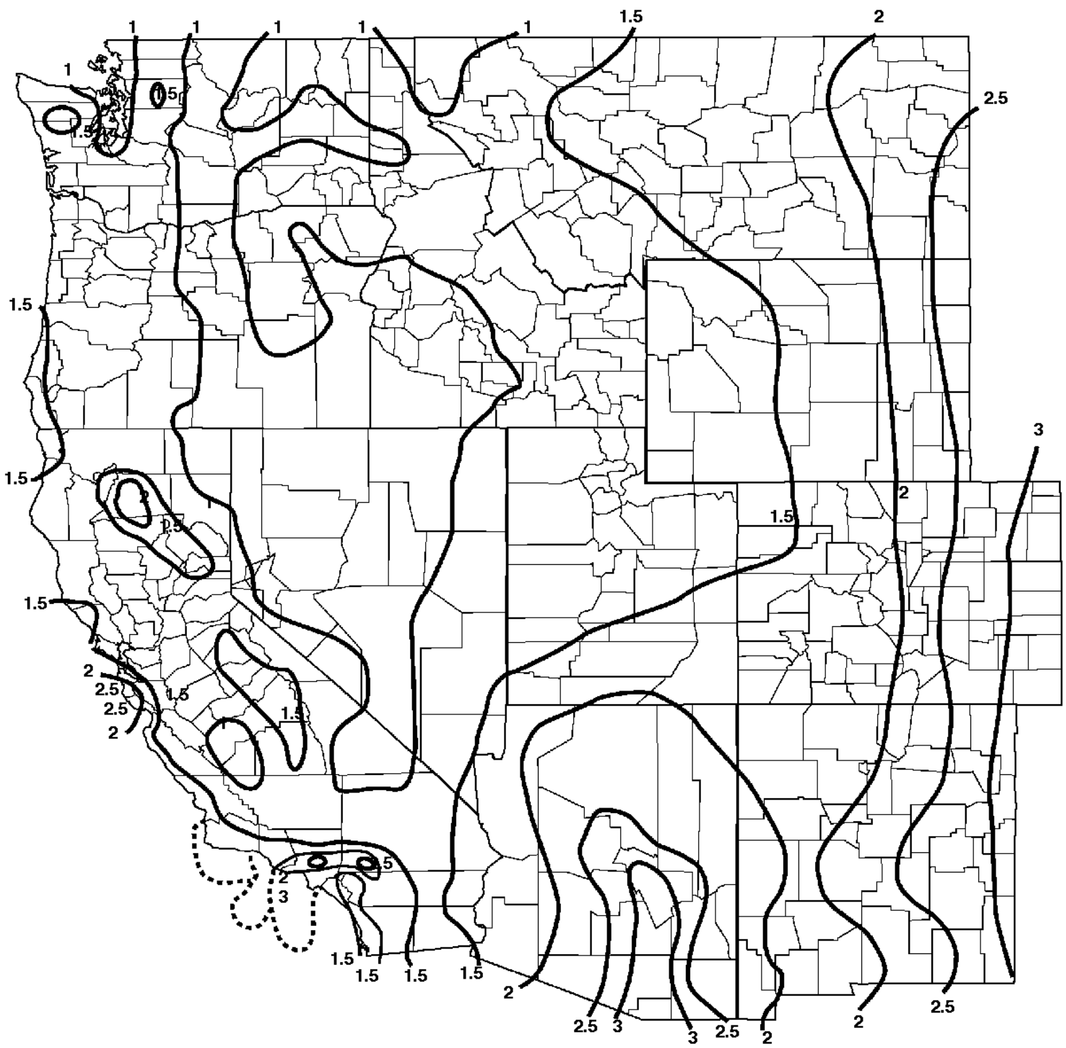
For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm,
1 cubic foot = 0.02832 m^2, 1 pound = 0.454 kg,
1 pound per square foot = 47.88 Pa.

Figure 1611.1(4)
EQUATION 16-35 COEFFICIENT



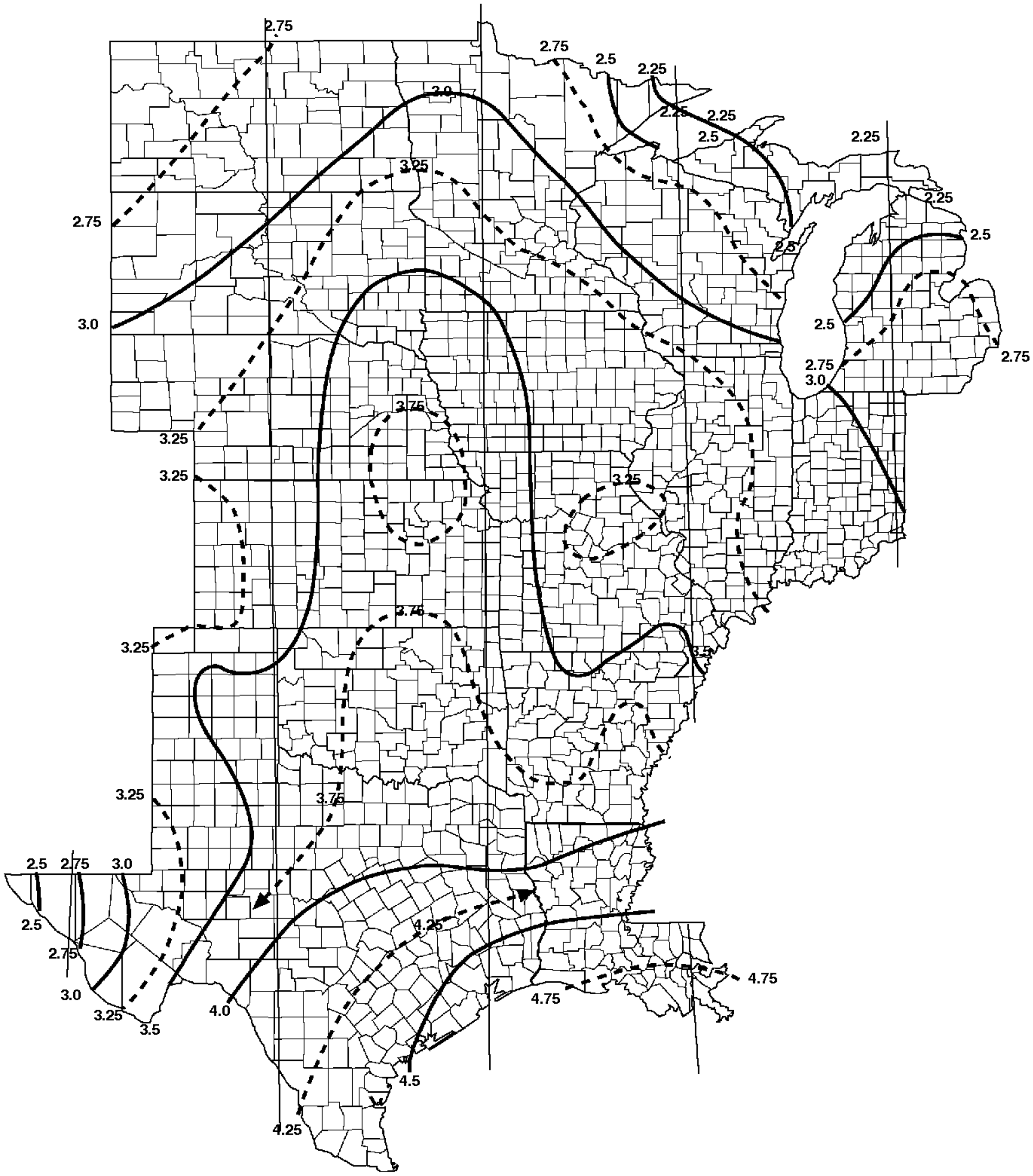
For SI: 1 inch = 25.4 mm.

Figure 1611.1(5)
RAIN LOAD EXAMPLE



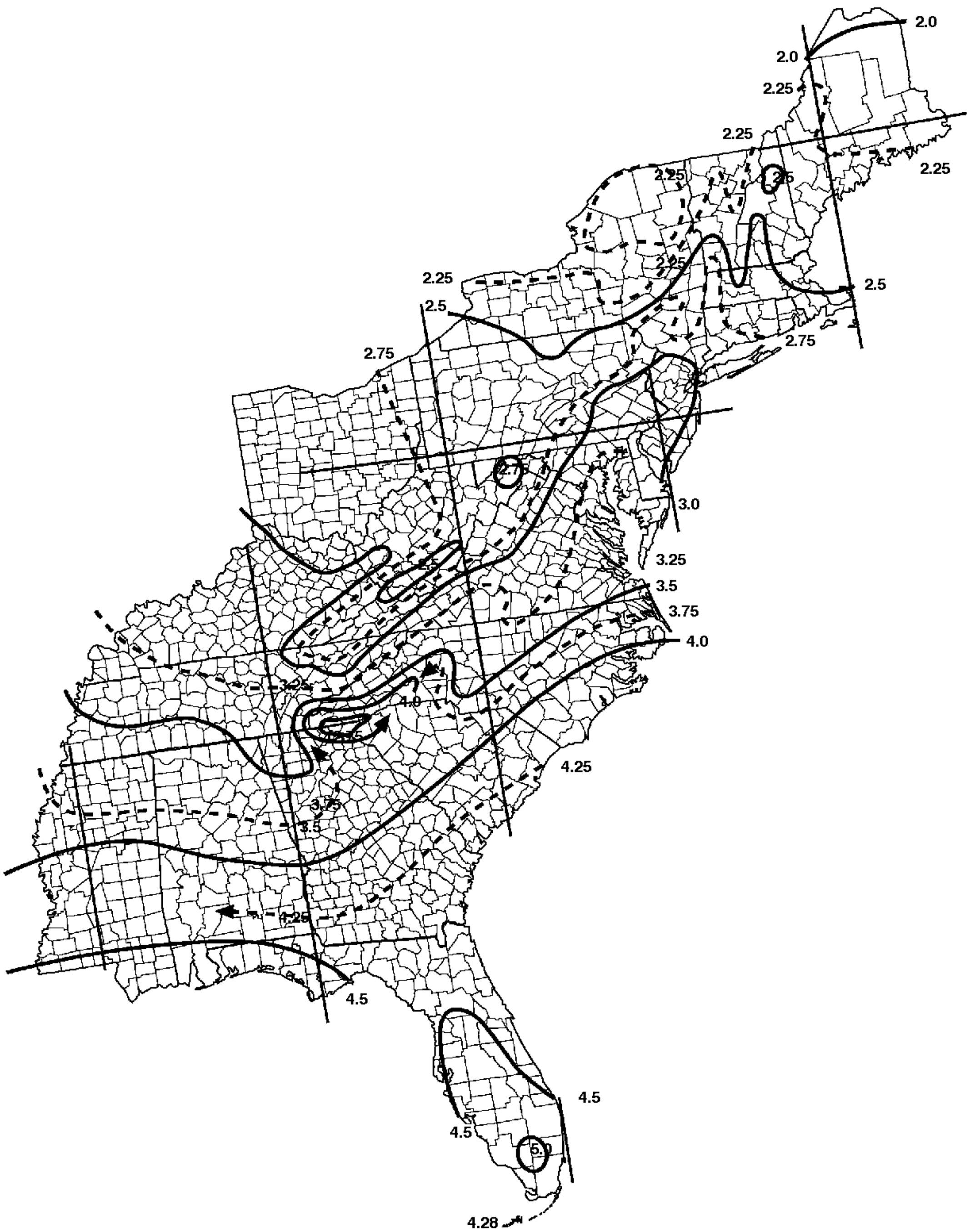
[P] FIGURE 1611.1
100-YEAR, 1-HOUR RAINFALL (INCHES) WESTERN UNITED STATES

For SI: 1 inch = 25.4 mm.
Source: National Weather Service, National Oceanic and Atmospheric Administration, Washington, DC.



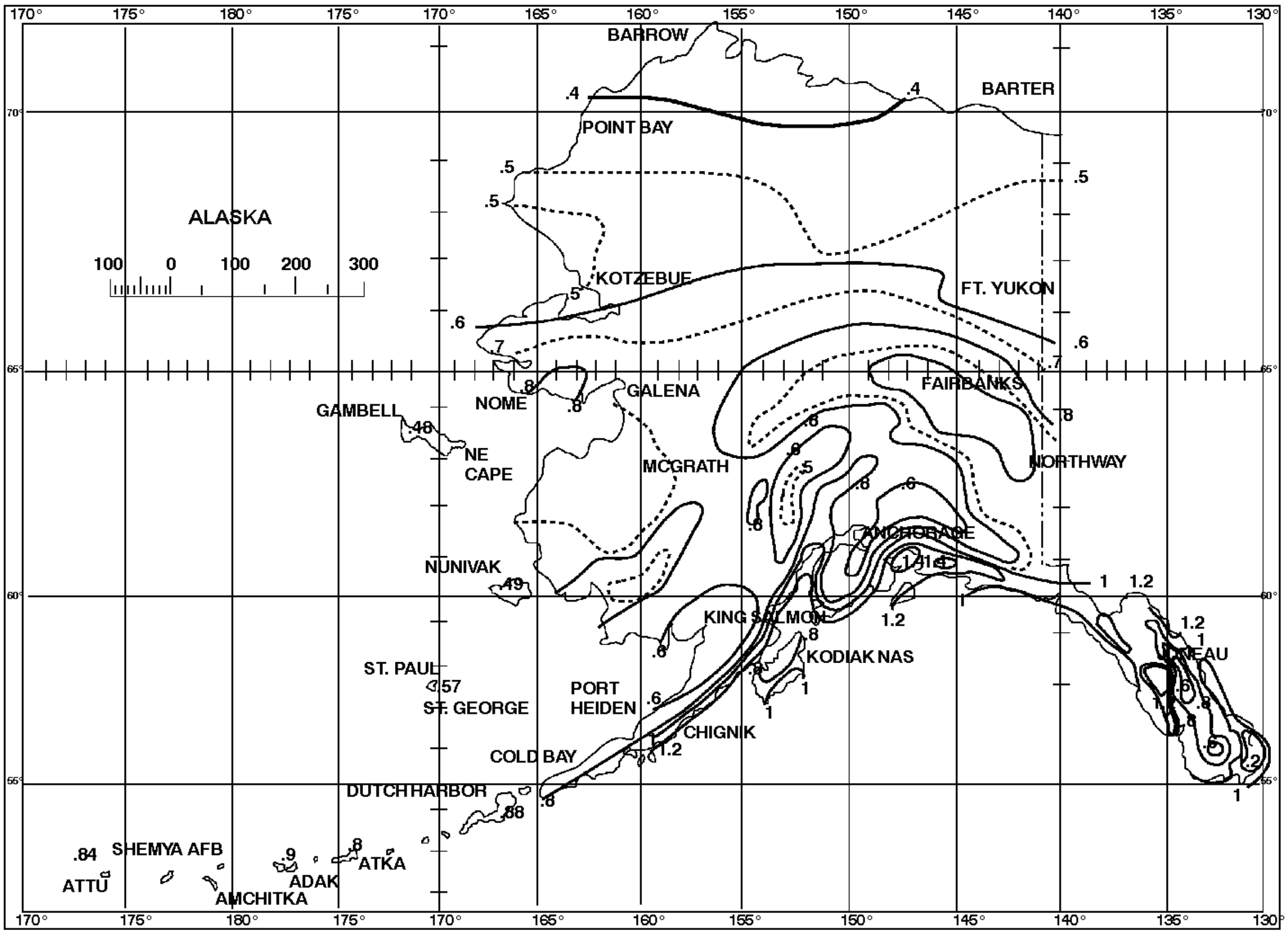
[P] FIGURE 1611.1—continued
100-YEAR, 1-HOUR RAINFALL (INCHES) CENTRAL UNITED STATES

For SI: 1 inch = 25.4 mm.
Source: National Weather Service, National Oceanic and Atmospheric Administration, Washington, DC.



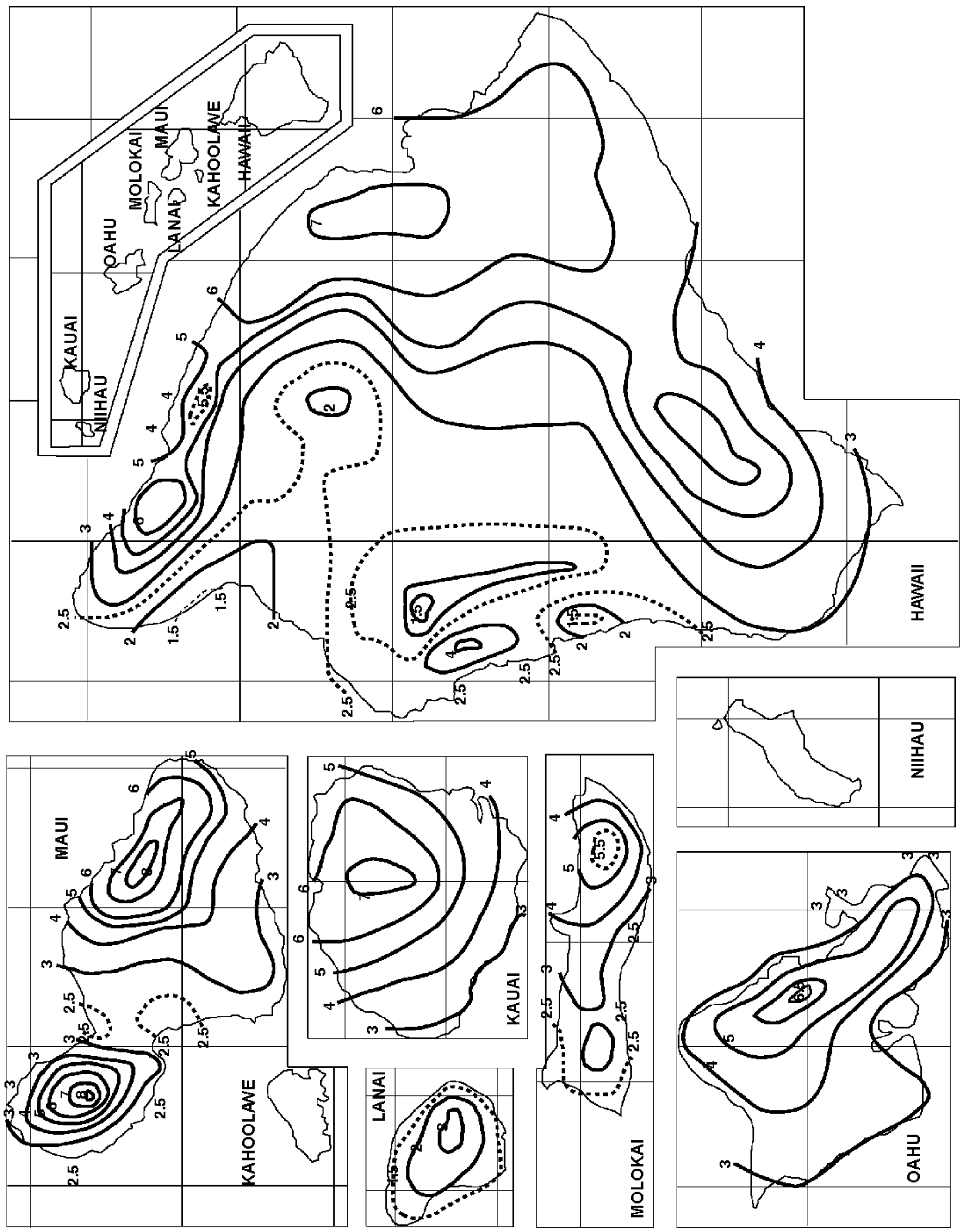
[P] FIGURE 1611.1—continued
100-YEAR, 1-HOUR RAINFALL (INCHES) EASTERN UNITED STATES

For SI: 1 inch = 25.4 mm.
Source: National Weather Service, National Oceanic and Atmospheric Administration, Washington, DC.



[P] FIGURE 1611.1—continued
100-YEAR, 1-HOUR RAINFALL (INCHES) ALASKA

For SI: 1 inch = 25.4 mm.
Source: National Weather Service, National Oceanic and Atmospheric Administration, Washington, DC.



[P] FIGURE 1611.1—continued
100-YEAR, 1-HOUR RAINFALL (INCHES) HAWAII

For SI: 1 inch = 25.4 mm.
Source: National Weather Service, National Oceanic and Atmospheric Administration, Washington, DC.

1611.2 Ponding instability. Susceptible bays of roofs shall be evaluated for ponding instability in accordance with Section 8.4 of ASCE 7.

❖ In roofs lacking sufficient framing stiffness, a condition known as "ponding instability" can occur where increasingly larger deflections caused by the continued accumulation of rainwater are large enough to overload the structure and result in a roof collapse. This must be countered by providing adequate stiffness in order to prevent increasingly larger deflections due to the buildup of rainwater. Another means to minimize the accumulation of rainwater is to camber the roof framing. This section requires a check for ponding instability at susceptible bays (see definition in Section 202). A ponding instability check is to be made assuming the primary roof drains are blocked. The determination of ponding instability is typically done by an iterative structural analysis where the incremental deflection is determined and the resulting increased rain load from the deflection is added to the original rain load.

1611.3 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 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 1611.1. Such roofs shall also be checked for ponding instability in accordance with Section 1611.2.

❖ Controlled drainage is the limitation of the drainage flow rate to a rate that is less than the rainfall rate such that the depth of the water intentionally builds up on the roof during a design rainfall. Controlled flow roof drain systems must be designed in accordance with Section 1111 of the IPC. A secondary roof drain system is needed to limit the buildup of water to a specific depth for roof design. The depth of water on the roof is also to include the depth of the water above the inlet of the secondary drain when the design flow rate is reached. Consideration of the effect of the accumulated rainwater is identical to assuming a blockage in the primary drainage system as discussed under Section 1611.1.

SECTION 1612
FLOOD LOADS

1612.1 General. Within *flood hazard areas* as established in Section 1612.3, all new construction of buildings, structures and portions of buildings and structures, including substantial improvement and restoration of substantial damage to buildings and structures, shall be designed and constructed to resist the effects of flood hazards and flood loads. For buildings that are located in more than one *flood hazard area*, the provisions associated with the most restrictive *flood hazard area* shall apply.

❖ This section addresses requirements for all buildings and structures in flood hazard areas. These areas are commonly referred to as "flood plains" and are shown on a community's Flood Insurance Rate Map (FIRM) prepared by FEMA or on another adopted flood hazard map. Code users should be aware that floods often affect areas outside the flood hazard area boundaries shown on FIRMs and can exceed the base flood elevation.

Through the adoption of the code, communities meet a significant portion of the flood plain management regulation requirements necessary to participate in the National Flood Insurance Program (NFIP). To participate in the NFIP, a jurisdiction must adopt regulations that at least meet the requirements of federal regulations in Section 60.3 of 44 CFR. This requirement can be satisfied by adopting the *International Building Code*® (IBC®) (including Appendix G) and the *International Residential Code*® (IRC®). [The *International Existing Building Code*® (IEBC®) also has provisions that are consistent with the NFIP.] If Appendix G (Flood-resistant Construction) is not enforced, its provisions must be captured in a companion flood plain management ordinance adopted by the community. The NFIP requires communities to regulate all development in flood hazard areas. Section 201.2 of Appendix G defines "Development" as "any man-made change to improved or unimproved real estate, including but not limited to buildings or other structures; temporary or permanent storage of materials; mining; dredging; filling; grading; paving; excavations; operations and other land-disturbing activities."

The NFIP was established to reduce flood losses, to better indemnify individuals from flood losses and to reduce federal expenditures for disaster assistance. A community that has been determined to have flood hazard areas elects to participate in the NFIP to protect health, safety and property, so that its citizens can purchase federally backed flood insurance. FEMA administers the NFIP, which includes monitoring community compliance with the flood plain management requirements of the NFIP.

New buildings and structures, and substantial improvements to existing buildings and structures, are to be designed and constructed to resist flood forces to minimize damage. Flood forces include flotation, lateral (hydrostatic) pressures, moving water (hydrodynamic) pressures, wave impact and debris impact. Flood-related hazards may include erosion and scour.

Many states and communities have elected to regulate flood plain development to a higher standard than the minimum required to participate in the NFIP. Communities considering using the code and other *International Codes*® to meet the flood plain management requirements of the NFIP are advised to consult with their state NFIP coordinator or the appropriate FEMA regional office.

If located in flood hazard areas, buildings and structures that are damaged by any cause are to be examined by the building official to determine if the damage constitutes substantial damage, in which the cost to repair or restore the building or structure to its predamaged condition equals or exceeds 50 percent of its market value before the damage occurred. All substantial improvements and repairs of buildings and structures that are substantially damaged must meet the flood-resistant provisions of the code. For additional guidance, see FEMA P-758, *Substantial Improvement/Substantial Damage Desk Reference*.

Some buildings are proposed to be located such that either they are in more than one flood zone or only a portion of the building is in a flood zone. Where this occurs, the entire building or structure is required to be designed and constructed according to the requirements of the more restrictive flood zone. For example, if a building is partially in a flood hazard area subject to high-velocity wave action (V zone), then the entire building must meet the requirements for that area. Similarly, if a building is partially in a flood hazard area and partially out of the mapped flood plain, then the entire building must be flood resistant.

For additional guidance on how to use the *International Codes®* to participate in the NFIP, see *Reducing Flood Losses through the International Codes: Meeting the Requirements of the National Flood Insurance Program*.

1612.2 Definitions. The following terms are defined in Chapter 2:

BASE FLOOD.

BASE FLOOD ELEVATION.

BASEMENT.

DESIGN FLOOD.

DESIGN FLOOD ELEVATION.

DRY FLOODPROOFING.

EXISTING CONSTRUCTION.

EXISTING STRUCTURE.

FLOOD or FLOODING.

FLOOD DAMAGE-RESISTANT MATERIALS.

FLOOD HAZARD AREA.

FLOOD HAZARD AREA SUBJECT TO HIGH-VELOCITY WAVE ACTION.

FLOOD INSURANCE RATE MAP (FIRM).

FLOOD INSURANCE STUDY.

FLOODWAY.

LOWEST FLOOR.

SPECIAL FLOOD HAZARD AREA.

START OF CONSTRUCTION.

SUBSTANTIAL DAMAGE.

SUBSTANTIAL IMPROVEMENT.

❖ Definitions facilitate the understanding of code provisions and minimize potential confusion. To that end, this section lists definitions of terms associated with flood requirements. Note that these definitions are found in Chapter 2. The use and application of defined terms, as well as undefined terms, are set forth in Section 201.

1612.3 Establishment of flood hazard areas. To establish *flood hazard areas*, the applicable governing authority shall adopt a flood hazard map and supporting data. The flood hazard map shall include, at a minimum, areas of special flood hazard as identified by the Federal Emergency Management Agency in an engineering report entitled "The Flood Insurance Study for **[INSERT NAME OF JURISDICTION]**," dated **[INSERT DATE OF ISSUANCE]**, as amended or revised with the accompanying Flood Insurance Rate Map (FIRM) and Flood Boundary and Floodway Map (FBFM) and related supporting data along with any revisions thereto. The adopted flood hazard map and supporting data are hereby adopted by reference and declared to be part of this section.

❖ Flood maps and studies are prepared by FEMA and are to be used as a community's official map, unless the community chooses to adopt a map that shows more extensive flood hazard areas. Most communities have multiple flood map panels, all of which should be listed in the adopting ordinance by panel number and date so that the appropriate effective map is used.

From time to time, FEMA's flood plain maps and studies may be revised and republished. In recent years, revised FIRMs have been produced in a digital format (referred to as Digital FIRMs or DFIRMs). Communities that prefer to cite the digital data should obtain a legal opinion. DFIRMs are registered to the primary coordinate system of the state or community. FEMA advises that the horizontal location of flood hazard areas relative to specific sites should be determined using the coordinate grid rather than planimetric base map features such as streets.

When maps are revised and flood hazard areas are changed, FEMA involves the community and provides a formal opportunity to review the documents. Once the revisions are finalized, FEMA requires the community to adopt the new maps. Communities may be able to minimize having to adopt each revision by referencing the date of the original map and study and all future revisions. This is a method by which subsequent revisions to flood maps and studies may be adopted administratively without requiring legislative action on the part of the community. Communities will need to determine whether this "adoption by reference" approach is allowed under their state's enabling authority and due process requirements. If not allowed, communities are to follow their state's requirements, which typically require public notices, hearings and specific adoption of revised maps by the community's legislative body.

1612.3.1 Design flood elevations. Where design flood elevations are not included in the *flood hazard areas* established in Section 1612.3, or where floodways are not designated, the *building official* is authorized to require the applicant to:

1. Obtain and reasonably utilize any design flood elevation and floodway data available from a federal, state or other source; or
2. Determine the design flood elevation and/or floodway in accordance with accepted hydrologic and hydraulic engineering practices used to define special flood hazard areas. Determinations shall be undertaken by a *registered design professional* who shall document that the technical methods used reflect currently accepted engineering practice.

❖ The purpose of this provision is to clarify how design flood elevations are to be determined for those flood hazard areas shown on community flood hazard maps that do not have the flood elevation already specified. Section 107.2 requires that the construction documents submitted are to be accompanied by a site plan, which includes flood hazard areas, floodways or design flood elevations, as applicable. While flood elevations are often available, a large percentage of areas that are mapped as special flood hazard areas by the NFIP do not have flood elevations or do not have floodway designations. This section clarifies the authority of the building official to require use of data, which may be obtained from other sources, or to require the applicant to develop flood hazard data, and is based on the NFIP regulation in 44 CFR §60.3(b)(4).

1612.3.2 Determination of impacts. In riverine *flood hazard areas* where design flood elevations are specified but floodways have not been designated, the applicant shall provide a floodway analysis that demonstrates that the proposed work will not increase the design flood elevation more than 1 foot (305 mm) at any point within the jurisdiction of the applicable governing authority.

❖ This section requires a floodway analysis to determine impacts. Development in riverine flood plains can increase flood levels and loads on other properties, especially if it occurs in areas known as "floodways" that must be reserved to convey flood flows. Commercial software for these analyses is readily available and FEMA provides software and technical guidance at http://www.fema.gov/plan/prevent/fhm/frm_soft.shtm.

This section provides consistency with the NFIP, which requires applicants to demonstrate whether their proposed work will increase flood levels in floodplains where the NFIP's FIRM shows base flood elevations, but floodways are not shown. A small percentage of flood plains where FEMA has specified base flood elevations do not have designated floodways.

1612.4 Design and construction. The design and construction of buildings and structures located in *flood hazard areas*, including flood hazard areas subject to high-velocity wave action, shall be in accordance with Chapter 5 of ASCE 7 and with ASCE 24.

❖ FEMA uses multiple designations for flood hazard areas shown on each FIRM, including A, AO, AH, A1-30, AE, A99, AR, AR/A1-30, AR/AE, AR/A0, AR/AH, AR/A, VO or V1-30, VE and V. Along many open coasts and lake shorelines where wind-driven waves are predicted, the flood hazard area is commonly referred to as the "V zone." Flood hazard areas that are inland of areas subject to high-velocity wave action and flood hazard areas along rivers and streams are commonly referred to as "A zones." Due to waves, the flood loads in areas subject to high-velocity wave action differ from those in other flood hazard areas. Some recent FIRMS in coastal communities show the "Limit of Moderate Wave Action," which is the inland extent of the 1.5-foot (457 mm) wave height. In ASCE 7 and ASCE 24, as well as in common usage, the area between the V zone and Limit of Moderate Wave Action is called "Coastal A Zone."

ASCE 24 outlines in detail the specific requirements that are to be applied to buildings and structures in all flood hazard areas. Communities that have AR zones and A99 zones shown on their FIRMs should consult with the appropriate state agency or FEMA regional office for guidance on the requirements that apply. AR zones are areas that result from the decertification of a previously accredited flood protection system (such as a levee) that is determined to be in the process of being restored to provide base flood protection. A99 zones are areas subject to inundation by the 1-percent-annual-chance flood event, but which will ultimately be protected upon completion of an under-construction federal flood protection system.

1612.5 Flood hazard documentation. The following documentation shall be prepared and sealed by a *registered design professional* and submitted to the *building official*:

1. For construction in *flood hazard areas* not subject to high-velocity wave action:
   - 1.1. The elevation of the lowest floor, including the basement, as required by the lowest floor elevation inspection in Section 110.3.3.
   - 1.2. For fully enclosed areas below the design flood elevation where provisions to allow for the automatic entry and exit of floodwaters do not meet the minimum requirements in Section 2.6.2.1 of ASCE 24, *construction documents* shall include a statement that the design will provide for equalization of hydrostatic flood forces in accordance with Section 2.6.2.2 of ASCE 24.

1.3. For dry floodproofed nonresidential buildings, *construction documents* shall include a statement that the dry floodproofing is designed in accordance with ASCE 24.

2. For construction in flood hazard areas subject to high-velocity wave action:

2.1. The elevation of the bottom of the lowest horizontal structural member as required by the lowest floor elevation inspection in Section 110.3.3.

2.2. *Construction documents* shall include a statement that the building is designed in accordance with ASCE 24, including that the pile or column foundation and building or structure to be attached thereto is designed to be anchored to resist flotation, collapse and lateral movement due to the effects of wind and flood loads acting simultaneously on all building components, and other load requirements of Chapter 16.

2.3. For breakaway walls designed to have a resistance of more than 20 psf (0.96 kN/m^2) determined using allowable stress design, *construction documents* shall include a statement that the breakaway wall is designed in accordance with ASCE 24.

❖ The NFIP requires that certain documentation be submitted in order to demonstrate compliance with provisions of the code that cannot be easily verified during a site inspection. The most common, described in Item 1.1, is the documentation of the lowest floor elevation. It provides documents that the lowest floor is at or above the required minimum elevation. Elevation is one of the most important aspects of flood-resistant construction and is a significant factor used to determine flood insurance premium rates. FEMA Form 81-31, *Elevation Certificate*, which includes illustrations and instructions, is recommended (download from www.fema.gov/business/nfip/elvinst.shtm). Building owners need elevation certificates to obtain NFIP flood insurance, and insurance agents use the certificates to compute the proper flood insurance premium rates.

The criteria for the minimum number and size of flood openings to allow free inflow and outflow of floodwaters under all types of flood conditions are set in ASCE 24. The statement described in Item 1.2 is required in the construction documents if the engineered openings are used (see Section 2.6.2.2 of ASCE 24); this statement is not required if nonengineered (prescriptive) openings meet the criteria of Section 2.6.2.1 of ASCE 24. For further guidance, refer to FEMA TB #1, *Openings in Foundation Walls and Walls of Enclosures Below Elevated Buildings in Special Flood Hazard Areas*.

The statement described in Item 1.3 is to be included in the construction documents for nonresidential buildings that are designed to be dry floodproofed. It is important to note that dry floodproofing is allowed only for nonresidential buildings and structures that are located in flood hazard areas not subject to high-velocity wave action. The registered design professional who seals the construction documents is indicating that, based upon development or review of the structural design, specifications and plans for construction, the design and methods of construction are in accordance with accepted standards of practice in ASCE 24 to meet the following provisions: (1) the structure, together with attendant utilities and sanitary facilities, is water tight to the floodproofed design elevation indicated with walls that are substantially impermeable to the passage of water and (2) all structural components are capable of resisting hydrostatic and hydrodynamic flood forces, including the effects of buoyancy and anticipated debris impact forces. The use of FEMA Form 81-65, *Floodproofing Certificate*, is recommended (download from http://www.fema.gov/plan/prevent/floodplain/nfipkeywordsfloodproofing_certificate.shtm).

This certificate is used by insurance agents to determine NFIP flood insurance premium rates for dry floodproofed nonresidential buildings. For further guidance, refer to FEMA TB #3, *Nonresidential Floodproofing—Requirements and Certification for Buildings Located in Special Flood Hazard Areas*.

Certain documentation must be submitted in order to demonstrate compliance with provisions of the code that cannot be verified readily during a site inspection. The most common, in Item 2.1, provides evidence that the bottom of the lowest horizontal members of buildings constructed in flood hazard areas subject to high-velocity wave action (V zones) are elevated to or above the minimum required height. Buildings located in flood hazard areas subject to high-velocity wave action and winds are expected to experience significant flood and wind loads simultaneously. FEMA and coastal communities report significant damage to buildings that are not built to current code. The statement described in Item 2.2 is included in the construction documents to indicate that the design meets the flood load provisions of ASCE 24 and other loads required by this chapter.

The documentation described in Item 2.3 is used only for specific situations in which properly elevated buildings in flood hazard areas subject to high-velocity wave action have enclosures beneath them, and then only if the walls of the enclosures are designed to resist more than 20 psf (0.96 kN/m^2) determined using ASD. Because breakaway walls will fail under flood conditions, building materials can become water-borne debris that may damage adjacent buildings. Refer to FEMA TB #5, *Free-of-Obstruction Requirements for Buildings Located in Coastal High-hazard Areas,* and FEMA TB #9, *Design and Construction Guidance for Breakaway Walls Below Elevated Buildings in Coastal High Hazard Areas*.

SECTION 1613
EARTHQUAKE LOADS

1613.1 Scope. Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7, excluding Chapter 14 and Appendix 11A. The *seismic design category* for a structure is permitted to be determined in accordance with Section 1613 or ASCE 7.

Exceptions:

1. Detached one- and two-family dwellings, assigned to *Seismic Design Category* A, B or C, or located where the mapped short-period spectral response acceleration, S_S, is less than 0.4 g.
2. The seismic force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.
3. Agricultural storage structures intended only for incidental human occupancy.
4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 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.

❖ These code provisions provide the requirements essential to determining a building's seismic design category. The balance of the earthquake load provisions are contained in the ASCE 7 load standard. The chapters that are noted as excluded from the ASCE 7 referenced standard are those that can create conflicts with Chapters 17 through 23. The balance of ASCE 7 earthquake load provisions are as follows:

Chapter	Subject
11	Seismic Design Criteria
12	Seismic Design Requirements for Building-Structures
13	Seismic Design Requirements for Nonstructural Components
15	Seismic Design Requirements for Non-building Structures
16	Seismic Response History Procedures
17	Seismic Design Requirements for Seismically Isolated Structures
18	Seismic Design Requirements for Structures with Damping Systems
19	Soil Structure Interaction for Seismic Design
20	Site Classification Procedure for Seismic Design
21	Site-specific Ground Motion Procedures for Seismic Design
22	Seismic Ground Motion and Long Period Transition Maps
23	Seismic Design Reference Documents

These seismic requirements are based, for the most part, on the NEHRP *Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. A new set of NEHRP recommended provisions has been prepared by the BSSC approximately every three years since the first edition in 1985. The code uses the NEHRP recommended provisions as the technical basis for seismic design requirements because of the nationwide input into the development of these design criteria. The NEHRP recommended provisions present up-to-date criteria for the design and construction of buildings subject to earthquake ground motions that are applicable anywhere in the nation. The requirements are intended to minimize the hazard to life for all buildings, increase the expected performance of higher occupancy buildings as compared to ordinary buildings and improve the capability of essential facilities to function during and after an earthquake. These minimum criteria are considered to be prudent and economically justified for the protection of life safety in buildings subject to earthquakes. Achieving the intended performance, however, depends on a number of factors, including the type of structural framing type, configuration, construction materials and as-built details of construction.

Detailed descriptions of changes made for the 2009 NEHRP *Recommended Provisions for Seismic Regulations for New Building and Other Structures* are available at www.bssconline.org under the explanation of changes made for the 2009 edition of the Provisions.

There are four exceptions to seismic design requirements included in this section. The following discussion addresses each of the exceptions.

Exception 1 exempts detached one- and two-family dwellings under two conditions. The first applies to structures assigned to Seismic Design Category A, B or C (meaning that both S_{ips} < 0.5g and S_{DI} < 0.2g). The second is for structures having a value for mapped short-period spectral response acceleration, S_s, less than 0.4g. The latter condition is derived from the ASCE 7 seismic provisions. Since it is based solely on the mapped value for short periods, it may allow structures to qualify more directly for this exception than will the first condition. In other words, the value of S_s should be checked first. If the structure qualifies based on S_s, then it is not necessary to check the seismic design category.

Exception 2 exempts conventional light-frame wood construction from the seismic requirements in this chapter. It should be noted that the limitations for conventional light-frame wood construction are included in Section 2308.2. There is no limitation on the use of the structure, except that Risk Category IV structures are not permitted to be constructed using the conventional light-frame wood construction provisions. Similarly, irregular portions of Seismic Design Category D and E structures are not permitted to be constructed using conventional light-frame wood construction (see commentary, Section 2308.12). The conventional light-frame wood construction provisions are deemed to provide equivalent seismic resistance as compared to construction designed in accordance with the requirements of this chapter based on the history of such conventional construction.

In Exception 3, agricultural buildings are exempt because they present a minimal life-safety hazard due to the low probability of human occupancy.

Exception 4 corresponds to an ASCE 7 exception regarding structures that are covered under other regulations, making it unnecessary to comply with the earthquake load requirements of the code.

1613.2 Definitions. The following terms are defined in Chapter 2:

❖ Definitions facilitate the understanding of code provisions and minimize potential confusion. To that end, this section lists definitions of terms associated with earthquake loads. Note that these definitions are found in Chapter 2. The use and application of defined terms, as well as undefined terms, are set forth in Section 201.

DESIGN EARTHQUAKE GROUND MOTION.

MECHANICAL SYSTEMS.

ORTHOGONAL.

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATION.

SEISMIC DESIGN CATEGORY.

SEISMIC FORCE-RESISTING SYSTEM.

SITE CLASS.

SITE COEFFICIENTS.

1613.3 Seismic ground motion values. Seismic ground motion values shall be determined in accordance with this section.

❖ The design earthquake ground motion levels determined in this section may result in damage, both structural and nonstructural, from the high stresses that occur because of the dynamic nature of seismic events. For most structures, damage from the design earthquake ground motion would be repairable, but might be so costly as to make it economically undesirable. For essential facilities, it is expected that damage from the design earthquake ground motion would not be so severe as to prevent continued occupancy and function of the facility. For ground motions greater than the design levels, the intent is that there be a low likelihood of structural collapse.

1613.3.1 Mapped acceleration parameters. The parameters S_s and S_1 shall be determined from the 0.2 and 1-second spectral response accelerations shown on Figures 1613.3.1(1) through 1613.3.1(6). 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. 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.

❖ The mapped maximum considered earthquake spectral response accelerations at 0.2-second period (S_s) and 1-second period (S_1) for a particular site are to be determined from Figures 1613.3.1(1) through (6). Where a site is between contours, as would usually be the case, straight-line interpolation or the value of the higher contour may be used. Areas that are considered to have a low seismic risk based solely on the mapped ground motions are placed directly into Seismic Design Category A.

The mapped maximum considered earthquake spectral response accelerations for a site may also be obtained using a seismic parameter program that has been developed by the United States Geological Survey (USGS) in cooperation with BSSC and FEMA. This program is available on the USGS earthquake hazards web site. The data are interpolated for a specific latitude-longitude or zip code, which the user enters. Output for an entry uses the built-in database to interpolate for the specific site. Caution should be used when using a zip code. In regions with highly variable mapped ground motions, the design parameters within a zip code may vary considerably from the value at the centroid of the zip code area. The code/NEHRP maximum considered earthquake (MCE) output for a site are the two spectral values required for design. The user may also use the program to calculate an MCE response spectrum, with or without site coefficients. Site coefficients can be calculated and included in calculations by simply selecting the site class; the program then calculates the site coefficient.

FIGURES 1613.3.1(1) through 1613.3.1(6). See page 16-74 through 16-81.

❖ These code figures provide the 5-percent damped spectral response accelerations at 0.2-second period (S_s), as well as at 1-second period (S_i) for Site Class B soil profiles. The code has incorporated updated earthquake ground motion maps that reflect the 2008 maps developed by the USGS National Seismic Hazard Mapping Project as well as technical changes adopted for the 2009 NEHRP *Recommended Seismic Provisions for New Buildings and Other Structures* (FEMA P750). In the NEHRP update process, the title for these maps was revised from “Maximum Considered Earthquake (MCE) Ground Motions” to “Risk-Targeted Earthquake (RTE) Ground Motions.”

The seismic hazard maps incorporate new information on earthquake sources and ground motion prediction equations, including the new Next Generation Attenuation (NGA) relations. The ground motion maps further incorporate technical changes that reflect the use of: (1) risk-targeted ground motions; (2) maximum direction ground motions and (3) near-source 84th percentile ground motions.

Precise design values can be obtained from a USGS web site (http://earthquake.usgs.gov/research/hazmaps/design/index.php) using the longitude and latitude of the building site, obtained from GPS mapping programs or web sites.

1613.3.2 Site class definitions. 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 of ASCE 7. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the building official or geotechnical data determines Site Class E or F soils are present at the site.

❖ Each site is to be classified as one of six site classes (A through F), as defined in ASCE 7, based on one of three soil properties measured over the top 100 feet (30 480 mm) of the site. If the top 100 feet (30 480 mm) are not homogeneous, the ASCET provisions address how to determine average properties. Site Class A is hard rock typically found in the eastern United States. Site Class B is softer rock, typical of the western parts of the country. Site Class C, D or E indicates progressively softer soils. From an earthquake-resistance perspective, rock is the best material for most structures to be founded on. Site Class F indicates soil so poor that site-specific geotechnical investigation and dynamic site-response analysis are needed to determine appropriate site coefficients.

The three soil properties forming the basis of site classification are: shear wave velocity, standard penetration resistance or blow count (determined in accordance with ASTM D 1586) and undrained shear strength (determined in accordance with ASTM D 2166 or ASTM D 2850). Site Class A and B designations must be based on shear wave velocity measurements or estimates.

Where soil property measurements to a depth of 100 feet (30 480 mm) are not feasible, the registered design professional performing the geotechnical investigation may estimate appropriate soil properties based on known geologic conditions.

When soil properties are not known in sufficient detail to determine the site class, the default site class is D, unless the building official determines that Site Class E or F soil may exist at the site.

1613.3.3 Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. The maximum considered earthquake spectral response acceleration for short periods, S_{MS}, and at 1-second period, S_{M1}, adjusted for *site class* effects shall be determined by Equations 16-37 and 16-38, respectively:

$$S_{MS} = F_a S_s \quad \textbf{(Equation 16-37)}$$

$$S_{M1} = F_v S_1 \quad \textbf{(Equation 16-38)}$$

where:

F_a = Site coefficient defined in Table 1613.3.3(1).

F_v = Site coefficient defined in Table 1613.3.3(2).

S_S = The mapped spectral accelerations for short periods as determined in Section 1613.3.1.

S_1 = The mapped spectral accelerations for a 1-second period as determined in Section 1613.3.1.

❖ Table 1613.3.3(1) defines an acceleration-related or short-period site coefficient, F_a, as a function of site class and the mapped spectral response acceleration at 0.2-second period, S_s. Table 1613.3.3(2) similarly defines a velocity-related or long-period site coefficient, F_v, as a function of site class and the mapped spectral response acceleration at 1-second period, S_1. The acceleration-related or short-period site coefficient F_a times S_s is S_{MS}, the 5-percent damped soil-modified maximum considered earthquake spectral response acceleration at short periods. The velocity-related or long-period site coefficient F_v times S_1 is S_{M1}, the 5-percent damped soil-modified maximum considered earthquake spectral response acceleration at 1-second period. Such modification by site coefficients is necessary because the mapped quantities are for Site Class B soils. Softer soils (Site Classes C through E) would typically amplify, and stiffer soils (Site Class A) would deamplify ground motion referenced to Site Class B.

As one would expect, both the short-period site coefficient, F_a, and the long-period site coefficient, F_v, are equal to unity for the benchmark Site Class B, irrespective of seismicity of the site. For Site Class A, both coefficients are smaller than unity, indicating reduction of benchmark Site Class B ground motion caused by the stiffer soils. For Site Classes C through E, both site coefficients, with the exception of F_a for Site Class E where $S_s \geq 1.00$, are larger than 1, indicating amplification of benchmark Site Class B ground motion on softer soils. For the same site class, each site coefficient is typically larger in areas of low seismicity than in areas of high seismicity. The basis of this lies in observations that low-magnitude subsurface rock motion is amplified to a larger extent by overlying softer soils than is high-magnitude rock motion. The site coefficients typically become larger for progressively softer soils. The only exception is provided by the short-period site coefficient in areas of high seismicity (S_s > 0.75), which remain unchanged or even decrease as the site class changes from D to E. The basis for this also lies in observations that very soft soils are not capable of amplifying the short-period components of subsurface rock motion; deamplification, in fact, takes place when the subsurface rock motion is high in magnitude.

TABLE 1613.3.3(1). See page 16-72.

❖ F_a, the acceleration-related or short-period site coefficient, is defined in this table as a function of site class and the seismicity at the site in the form of the mapped spectral response acceleration at 0.2-second period, S_s.

TABLE 1613.3.3(2). See below.

❖ F_v, the velocity-related or long-period site coefficient, is defined in this table as a function of site class and the seismicity at the site in the form of the mapped spectral response acceleration at 1-second period, S_1.

1613.3.4 Design spectral response acceleration parameters. Five-percent damped design spectral response acceleration at short periods, $S_{DS,}$ and at 1-second period, $S_{DS,}$ shall be determined from Equations 16-39 and 16-40, respectively:

$$S_{DS} = \frac{2}{3}S_{MS} \quad \textbf{(Equation 16-39)}$$

$$S_{D1} = \frac{2}{3}S_{M1} \quad \textbf{(Equation 16-40)}$$

where:

S_{MS} = The maximum considered earthquake spectral response accelerations for short period as determined in Section 1613.3.3.

S_{M1} = The maximum considered earthquake spectral response accelerations for 1-second period as determined in Section 1613.3.3.

❖ The design spectral response acceleration at 0.2-second period, S_{DS}, is two-thirds of the S_{MS} value calculated in accordance with Section 1613.3.3. The design spectral response acceleration at 1-second period, S_{D1}, is two-thirds of the S_{M1} value calculated in accordance with Section 1613.3.3. Two-thirds is the reciprocal of 1.5; thus, the design ground motion is 1/1.5 times the soil-modified maximum considered earthquake ground motion. This is in recognition of the inherent margin contained in the NEHRP provisions that would make collapse unlikely under 1.5 times the design-level ground motion. The idea is to avoid collapse when a structure is subjected to the soil-modified maximum considered earthquake ground motion.

1613.3.5 Determination of seismic design category. Structures classified as *Risk Category* I, II or III that are located where the mapped spectral response acceleration parameter at 1-second period, S_1, is greater than or equal to 0.75 shall be assigned to *Seismic Design Category* E. Structures classified as *Risk Category* IV that are located where the mapped spectral response acceleration parameter at 1-second 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 1613.3.4 or the site-specific procedures of ASCE 7. Each building and structure shall be assigned to the more severe *seismic design category* in accordance with Table 1613.3.5(1) or 1613.3.5(2), irrespective of the fundamental period of vibration of the structure, *T*.

❖ The seismic design category classification provides a relative scale of earthquake risk to structures. The seismic design category considers not only the seismicity of the site in terms of the mapped spectral response accelerations, but also the site soil profile and the nature of the structure's risk category (see the step-by-step description of this process under the

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a [a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIOD				
	$S_s \le 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \ge 1.25$
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	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_s.
b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_V [a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT 1-SECOND 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	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, S_1.
b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

definition of “Seismic design category”). It is important to note that there are two tables that are used in order to establish the most restrictive seismic design category classification.

There are two conditions that allow the seismic design category to be determined based upon mapped spectral accelerations, making it unnecessary to go through the process described above. The first is Section 1613.3.1, which identifies areas that have a low seismic risk and are classified as Seismic Design Category A without the need to go through the usual steps. The second instance is an area that is close to a major active fault where S_1 is greater than or equal to 0.75g. These areas are deemed to be areas of considerable seismic risk, and the seismic design category is classified as E or F depending on the structure’s risk category.

The seismic design category classification is a key criterion in using and understanding the seismic requirements because the analysis method, general design, structural material detailing and the structure’s component and system design requirements are determined, at least in part, by the seismic design category. Some of the special inspection requirements in Section 1705.11 and structural observation requirements in Section 1704.5 are dependent on the seismic design category classification, as well.

TABLE 1613.3.5(1). See below.

❖ This table defines seismic design category as a function of risk category and the short-period (0.2 second) spectral response acceleration at the site of a structure. As the value of S_{DS} increases, the structure is assigned a higher seismic design category and the earthquake design requirements become more stringent.

TABLE 1613.3.5(2). See below.

❖ This table defines seismic design category as a function of risk category and the long-period (1-second) spectral response acceleration at the site of a structure. As the value of S_{M1} increases, the structure is assigned a higher seismic design category and the earthquake design requirements become more stringent.

1613.3.5.1 Alternative seismic design category determination. Where S_1 is less than 0.75, the *seismic design category* is permitted to be determined from Table 1613.3.5(1) alone when all of the following apply:

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, Ta, in each of the two orthogonal directions determined in accordance with Section 12.8.2.1 of ASCE 7, is less than 0.8 T_s determined in accordance with Section 11.4.5 of ASCE 7.
2. In each of the two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.
3. Equation 12.8-2 of ASCE 7 is used to determine the seismic response coefficient, C_s.
4. The diaphragms are rigid as defined in Section 12.3.1 of ASCE 7 or, for diaphragms that are flexible, the distances between vertical elements of the seismic force-resisting system do not exceed 40 feet (12 192 mm).

❖ This section permits the seismic design category of structures meeting the four listed conditions to be based solely on the short-period design spectral coefficient, S_{DS}. This is similar to the approach taken in mapping seismic design categories for use in the IRC, as well as for the simplified earthquake analysis procedure.

TABLE 1613.3.5(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATIONS

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

TABLE 1613.3.5(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

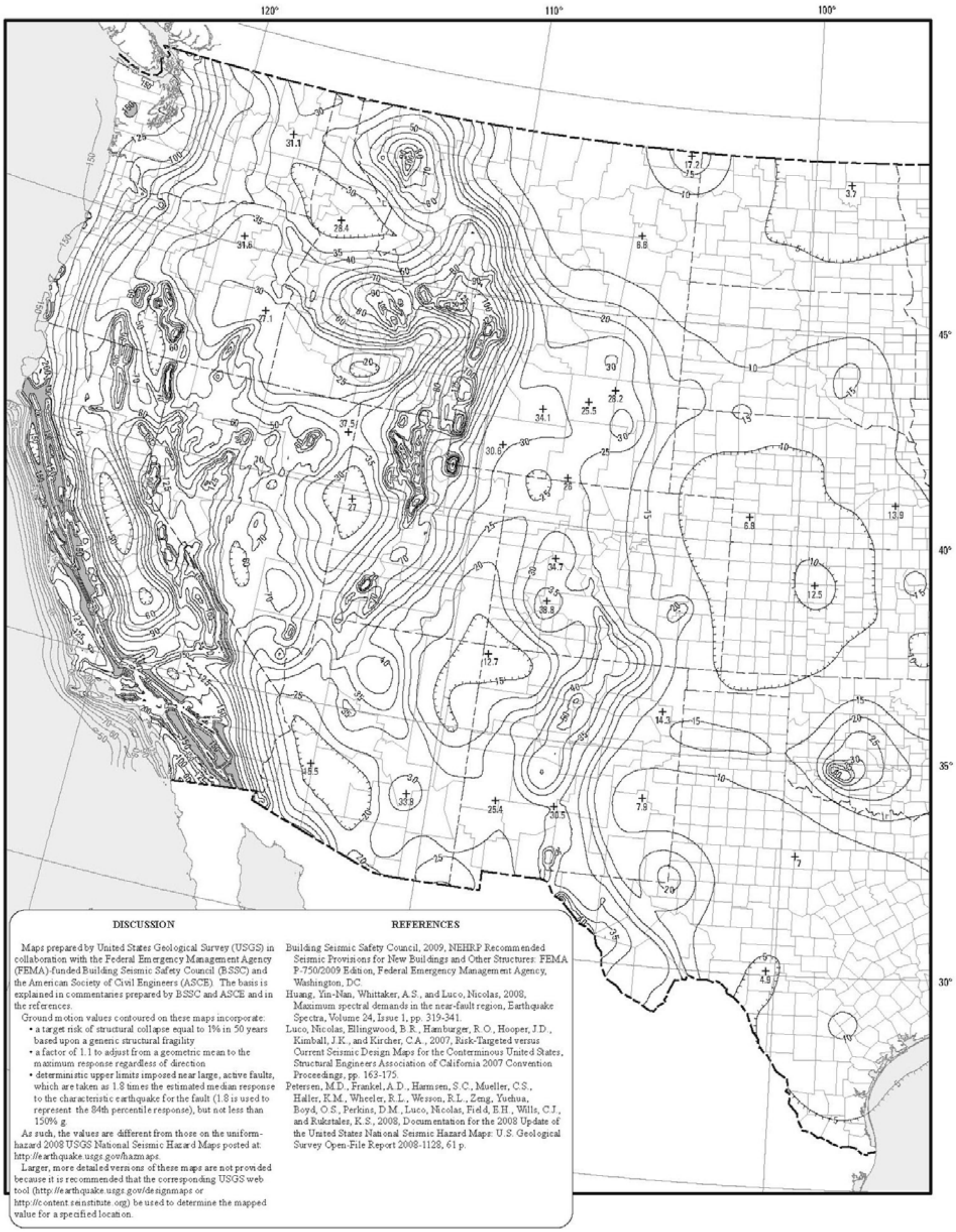


FIGURE 1613.3.1(1)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

(continued)

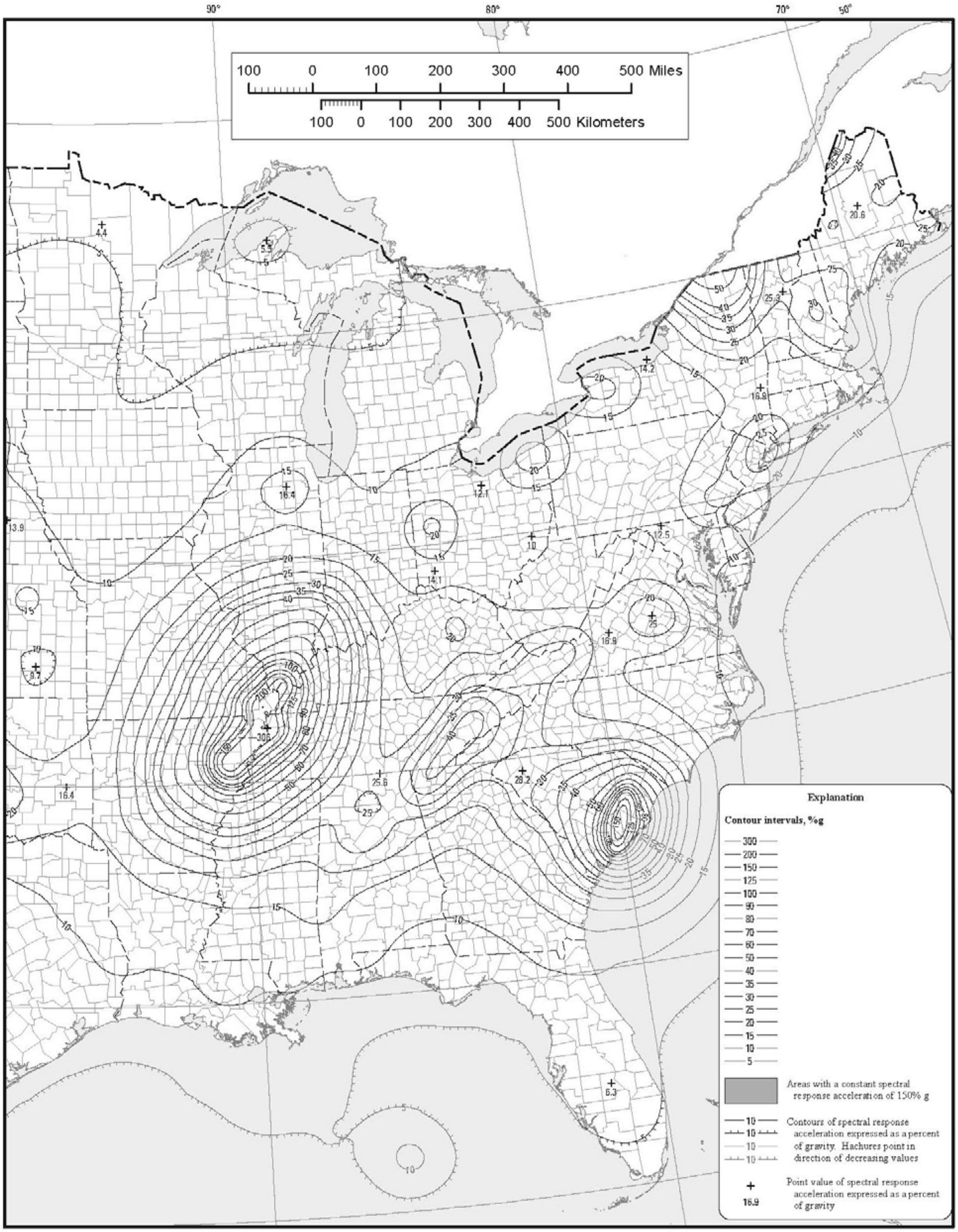


FIGURE 1613.3.1(1)—continued
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

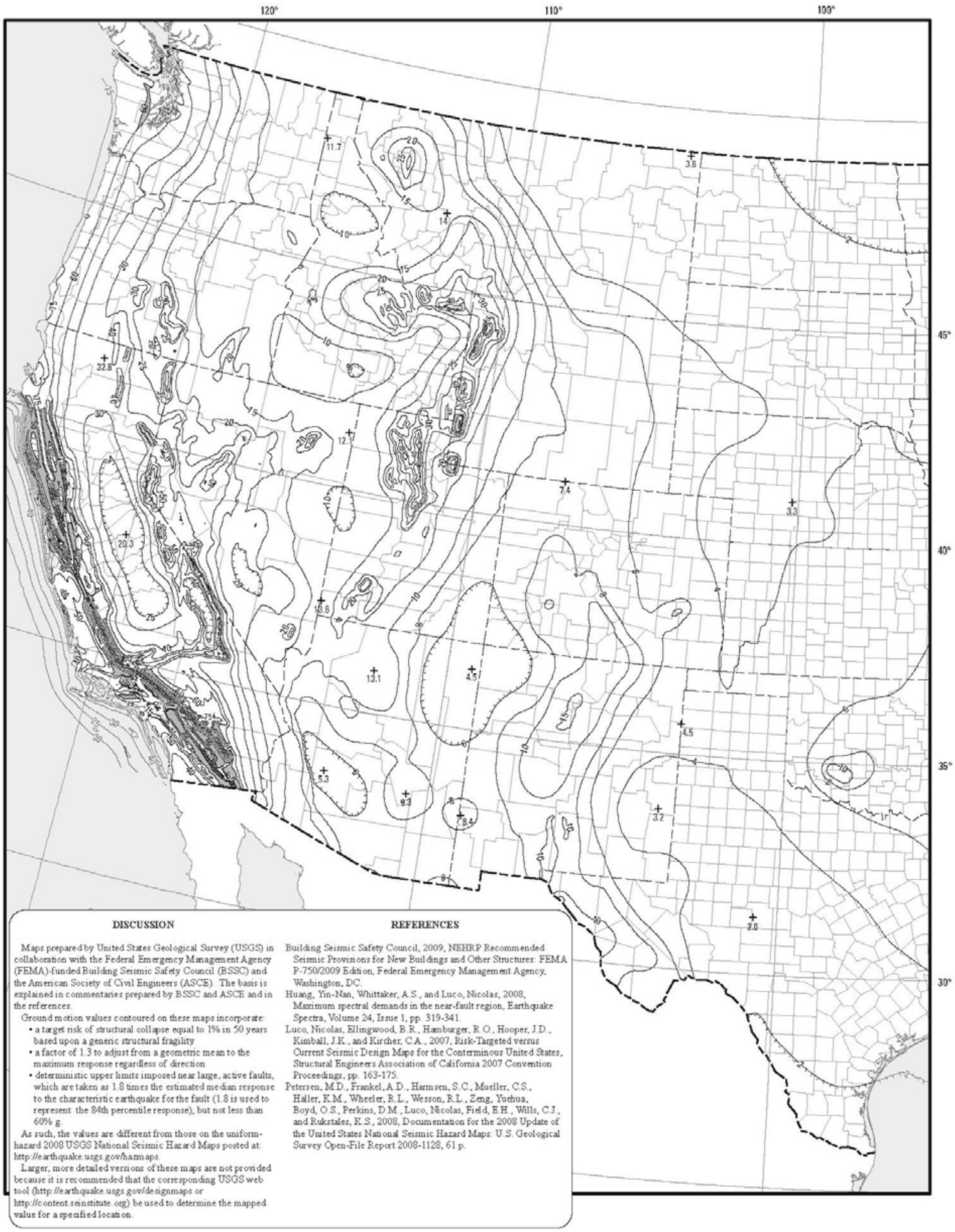


FIGURE 1613.3.1(2)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

(continued)

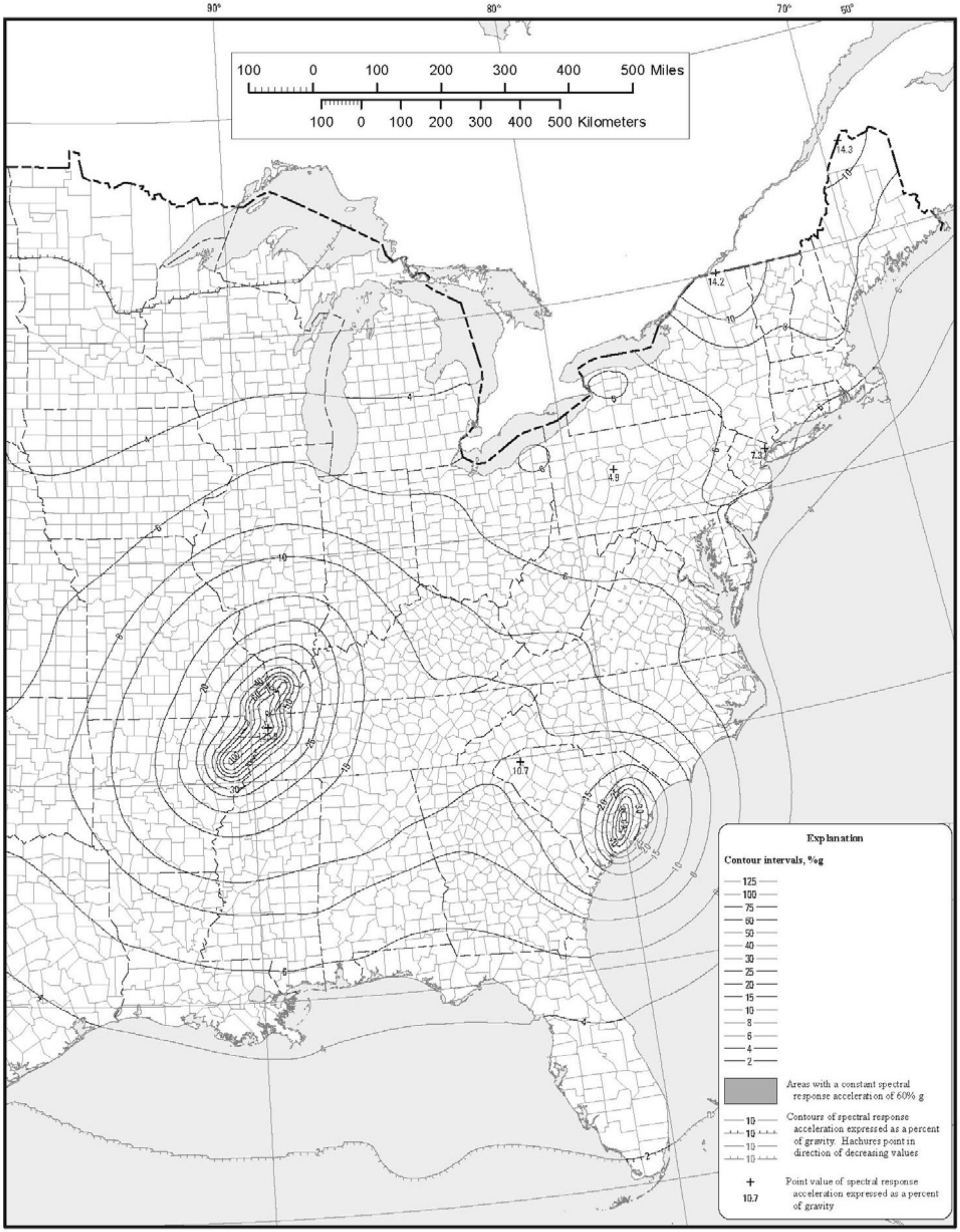
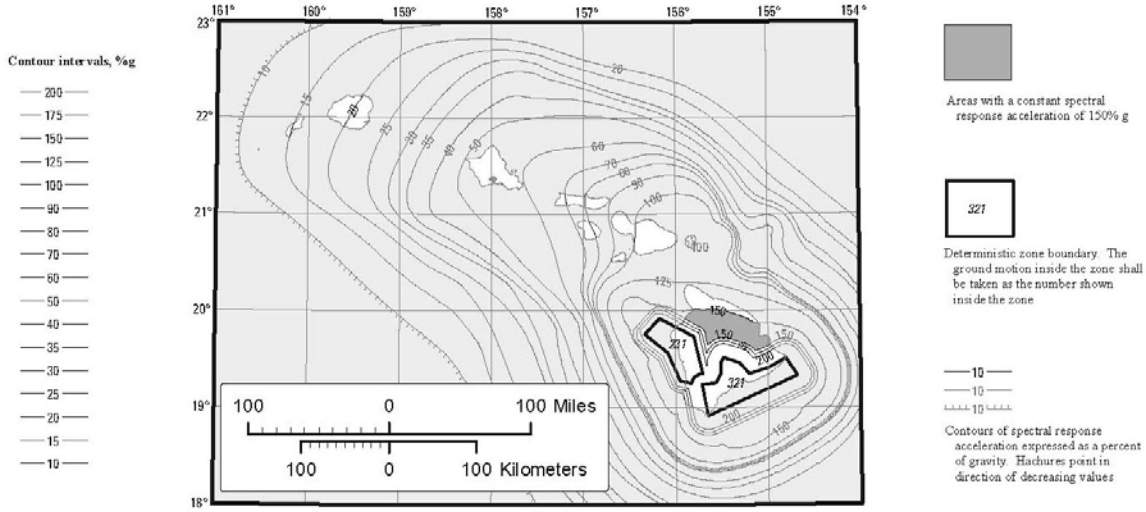
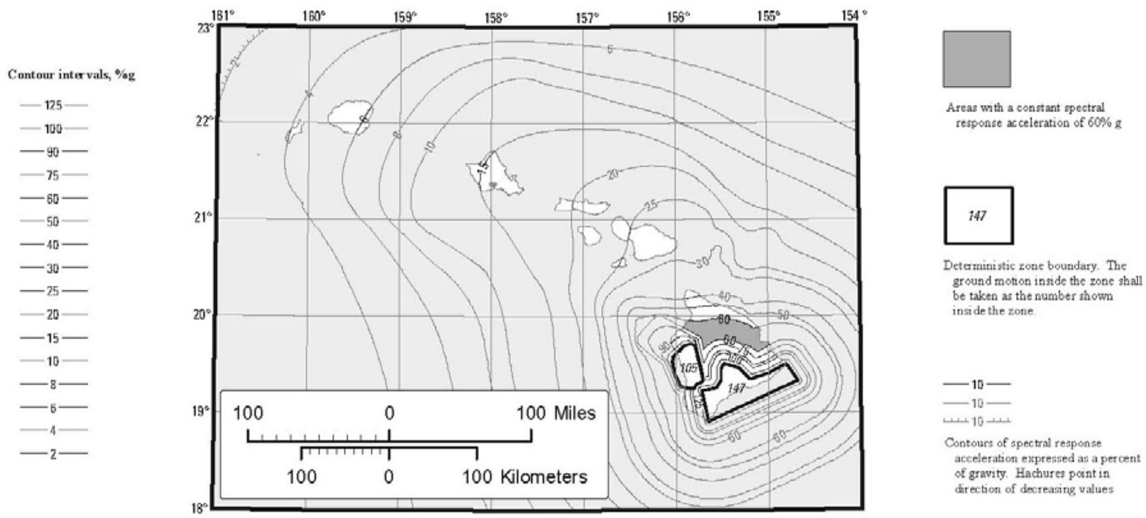


FIGURE 1613.3.1(2)—continued
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



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.

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FIGURE 1613.3.1(3)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR HAWAII OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

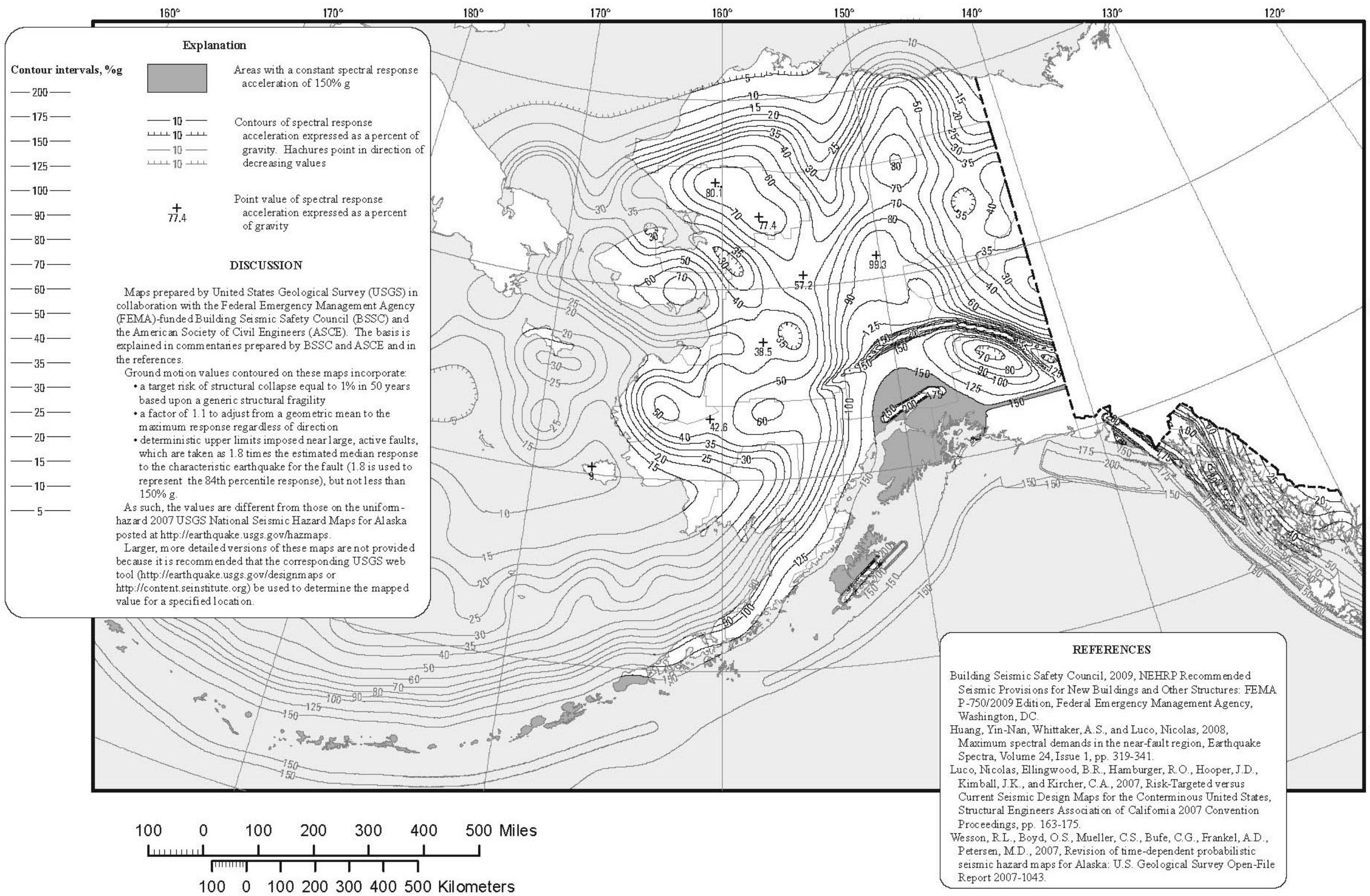


FIGURE 1613.3.1(4)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR ALASKA OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

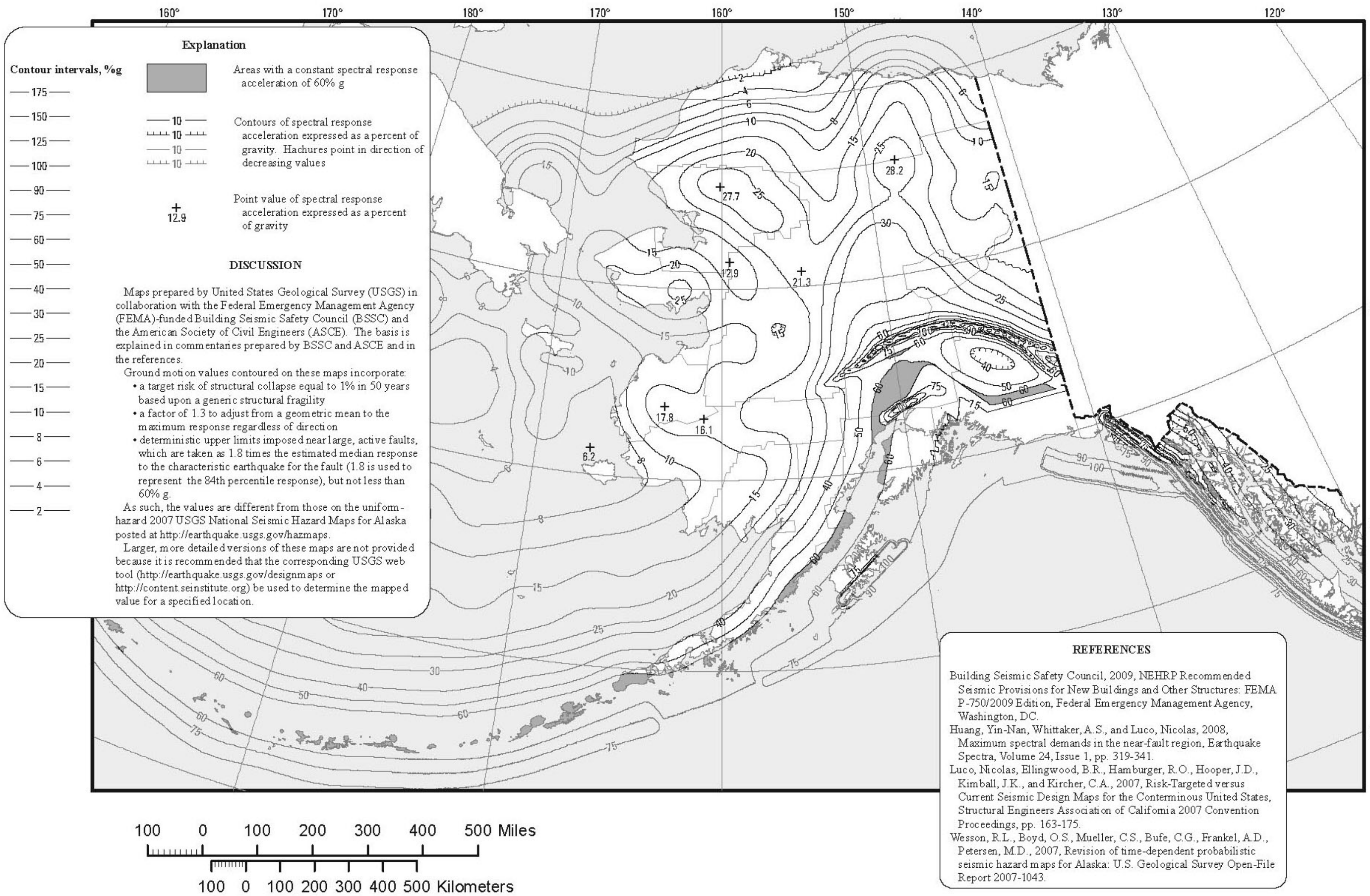
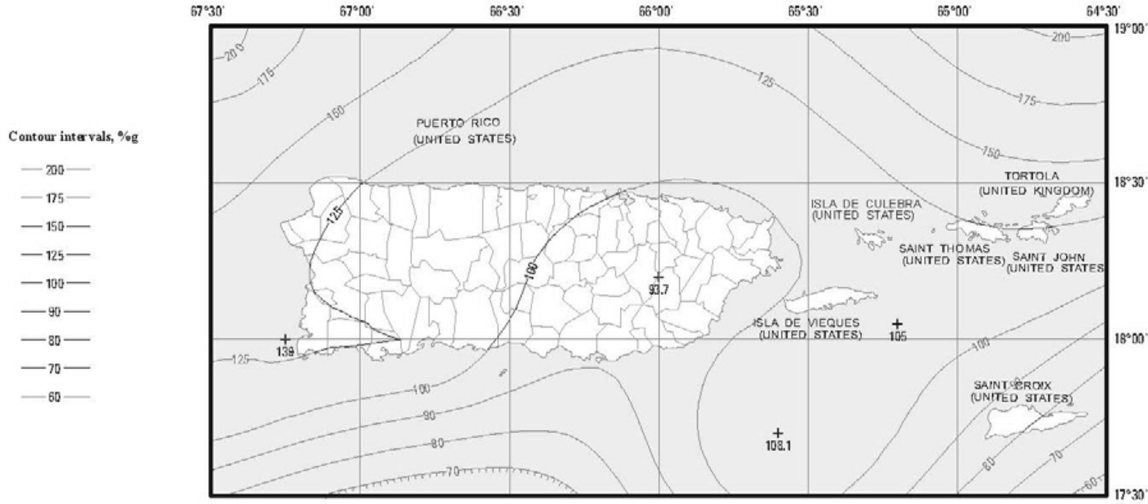
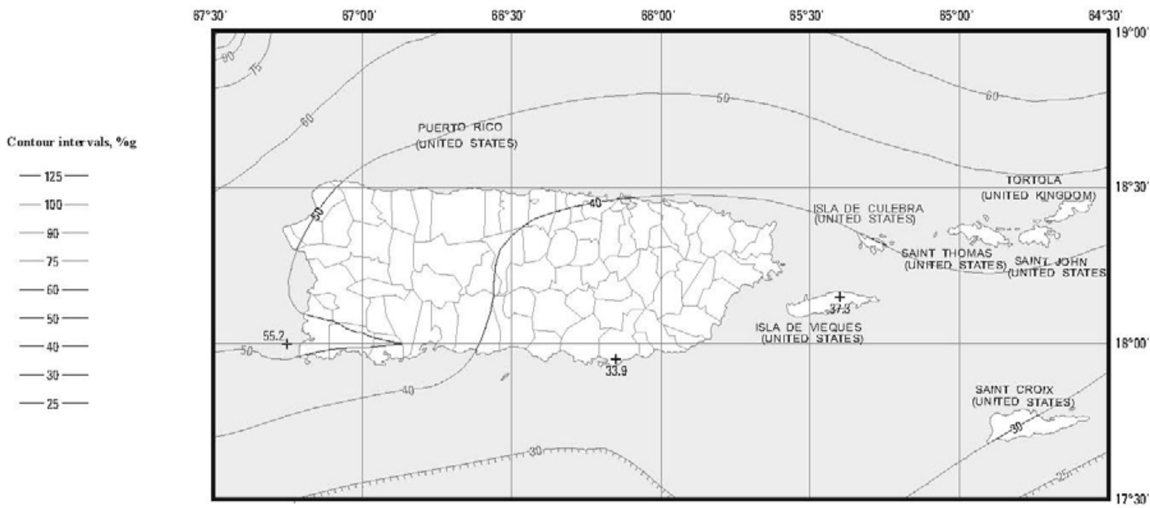


FIGURE 1613.3.1(5)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR ALASKA OF 1.0-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

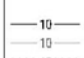


0.2 Second Spectral Response Acceleration (5% of Critical Damping)



1.0 Second Spectral Response Acceleration (5% of Critical Damping)

Explanation



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

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.

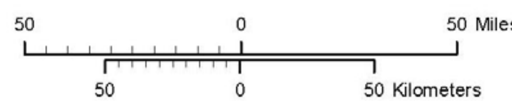


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FIGURE 1613.3.1(6)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR PUERTO RICO AND THE UNITED STATES VIRGIN ISLANDS OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

1613.3.5.2 Simplified design procedure. Where the alternate simplified design procedure of ASCE 7 is used, the *seismic design category* shall be determined in accordance with ASCE 7.

❖ A simplified earthquake procedure was first introduced under the 1997 UBC and continued on with minor changes through the first two editions of the code. The 2005 edition of ASCE 7 introduced the first stand- alone version of a simplified earthquake analysis for simple bearing wall or building frame systems that comply with the 12 limitations listed in Section 12.14.1.1 of ASCE 7. For structures that qualify to use this method of analysis, the simplified procedure permits the determination of the seismic design category to be based solely on the short-period design spectral coefficient, S_{DS}.

1613.4 Alternatives to ASCE 7. The provisions of Section 1613.4 shall be permitted as alternatives to the relevant provisions of ASCE 7.

❖ This section is intended to provide permitted alternatives to the corresponding ASCE 7 requirements. As such, they should be viewed as options that are available to the design professional in demonstrating compliance with the earthquake provisions.

1613.4.1 Additional seismic force-resisting systems for seismically isolated structures. Add the following exception to the end of Section 17.5.4.2 of ASCE 7:

> **Exception:** For isolated structures designed in accordance with this standard, the Structural System Limitations and the Building Height Limitations in Table 12.2-1 for ordinary steel concentrically braced frames (OCBFs) as defined in Chapter 11 and ordinary moment frames (OMFs) as defined in Chapter 11 are permitted to be taken as 160 feet (48 768 mm) for structures assigned to *Seismic Design Category* D, E or F, provided that the following conditions are satisfied:
>
> 1. The value of R_I as defined in Chapter 17 is taken as 1.
> 2. For OMFs and OCBFs, design is in accordance with AISC 341.

❖ The ASCE 7 provisions include requirements for structures that utilize seismic base isolation. It is worth noting that use of seismic isolation is never required by the code. This method is an optional method of design for earthquake loading that has been recognized under legacy model codes, such as the UBC. The modification adds an exception to the ASCE 7 requirements for structural systems above the isolation system. It corrects an oversight that unnecessarily changes prior practice by restricting the use of certain seismic-force-resisting systems that have been used successfully. As modified, it would allow ordinary concentric braced frames and ordinary moment frame systems with heights up to 160 feet (48 768 mm) when the stated conditions are met.

SECTION 1614 ATMOSPHERIC ICE LOADS

1614.1 General. Ice-sensitive structures shall be designed for atmospheric ice loads in accordance with Chapter 10 of ASCE 7.

❖ This section provides charging text in the code in conjunction with technical provisions in ASCE 7 for computing atmospheric ice loads. This section relies on the determination of which structures are ice-sensitive structures in order to determine the need to comply with the applicable provisions of ASCE 7. An “Ice-sensitive structure” is defined in Section 202 and provides a technical basis for determining which structures are ice-sensitive structures.

SECTION 1615 STRUCTURAL INTEGRITY

1615.1 General. High-rise buildings that are assigned to *Risk Category* III or IV shall comply with the requirements of this section. Frame structures shall comply with the requirements of Section 1615.3. Bearing wall structures shall comply with the requirements of Section 1615.4.

❖ The requirements in the codes and standards together with the common structural design and construction practices prevalent in the United States have provided the overwhelming majority of structures with adequate levels of reliability and safety. Many building simply do not have integrity issues. Low-rise buildings do not represent the same risk as taller, high-rise buildings. The structural integrity provisions reflect this relative risk. By limiting this requirement to Risk Category III and IV buildings that are also high-rise buildings, the provision does not affect buildings that are commonly built and do not exhibit integrity issues.

1615.2 Definitions. The following words and terms are defined in Chapter 2:

❖ Definitions facilitate the understanding of code provisions and minimize potential confusion. To that end, this section lists definitions of terms associated with these provisions for structural integrity. Note that these definitions are found in Chapter 2. The use and application of defined terms, as well as undefined terms, are set forth in Section 201.

BEARING WALL STRUCTURE.

FRAME STRUCTURE.

1615.3 Frame structures. Frame structures shall comply with the requirements of this section.

❖ These provisions enhance the overall structural integrity and resistance of frame structures by establishing minimum requirements for tying together the primary structural elements.

1615.3.1 Concrete frame structures. Frame structures constructed primarily of reinforced or prestressed concrete, either cast-in-place or precast, or a combination of these, shall conform to the requirements of ACI 318 Sections 7.13, 13.3.8.5, 13.3.8.6, 16.5, 18.12.6, 18.12.7 and 18.12.8 as applicable. Where ACI 318 requires that nonprestressed reinforcing or prestressing steel pass through the region bounded by the longitudinal column reinforcement, that reinforcing or prestressing steel shall have a minimum nominal tensile strength equal to two-thirds of the required one-way vertical strength of the connection of the floor or roof system to the column in each direction of beam or slab reinforcement passing through the column.

Exception: Where concrete slabs with continuous reinforcement having an area not less than 0.0015 times the concrete area in each of two orthogonal directions are present and are either monolithic with or equivalently bonded to beams, girders or columns, the longitudinal reinforcing or prestressing steel passing through the column reinforcement shall have a nominal tensile strength of one-third of the required one-way vertical strength of the connection of the floor or roof system to the column in each direction of beam or slab reinforcement passing through the column.

❖ This section refers to ACI 318 requirements for structural integrity that are applicable to concrete frames. These provisions establish minimum requirements for tying together the primary structural elements. These ACI 318 structural integrity requirements are already incorporated in the code by the general reference to ACI 318 in Section 1901.2 for the design and construction of structural concrete. The only additional requirement for concrete frame structures is the minimum nominal tensile strength that is established for reinforcement that must pass through the longitudinal column reinforcement (see Section 7.13.2.5 of ACI 318). The exception reduces this minimum nominal tensile strength for slab construction that meets certain conditions.

1615.3.2 Structural steel, open web steel joist or joist girder, or composite steel and concrete frame structures. Frame structures constructed with a structural steel frame or a frame composed of open web steel joists, joist girders with or without other structural steel elements or a frame composed of composite steel or composite steel joists and reinforced concrete elements shall conform to the requirements of this section.

❖ The provisions contained in this section have been adapted from requirements contained in the ACI 318 standard for many years. By adapting those requirements to structural steel, open web steel joist or joist girder, or composite steel and concrete frame structures, their overall structural integrity is enhanced.

1615.3.2.1 Columns. Each column splice shall have the minimum design strength in tension to transfer the design dead and live load tributary to the column between the splice and the splice or base immediately below.

❖ The additional requirement for the tensile strength of column splices enhances the column's performance in unforeseen events.

1615.3.2.2 Beams. End connections of all beams and girders shall have a minimum nominal axial tensile strength equal to the required vertical shear strength for *allowable stress design* (ASD) or two-thirds of the required shear strength for *load and resistance factor design* (LRFD) but not less than 10 kips (45 kN). For the purpose of this section, the shear force and the axial tensile force need not be considered to act simultaneously.

Exception: Where beams, girders, open web joist and joist girders support a concrete slab or concrete slab on metal deck that is attached to the beam or girder with not less than $^{3}/_{8}$-inch-diameter (9.5 mm) headed shear studs, at a spacing of not more than 12 inches (305 mm) on center, averaged over the length of the member, or other attachment having equivalent shear strength, and the slab contains continuous distributed reinforcement in each of two orthogonal directions with an area not less than 0.0015 times the concrete area, the nominal axial tension strength of the end connection shall be permitted to be taken as half the required vertical shear strength for ASD or one-third of the required shear strength for LRFD, but not less than 10 kips (45 kN).

❖ Providing the required tensile strength for all beam and girder connections provides some ability to carry, transfer and/or redistribute load in the event there is loss of support. The exception allows a reduced tensile strength in the beam and girder connection where a concrete slab is utilized.

1615.4 Bearing wall structures. Bearing wall structures shall have vertical ties in all load-bearing walls and longitudinal ties, transverse ties and perimeter ties at each floor level in accordance with this section and as shown in Figure 1615.4.

❖ These provisions enhance the overall structural integrity and resistance of bearing wall structures by establishing minimum requirements for tying together the primary structural elements.

FIGURE 1615.4. See page 16-84.

❖ This figure illustrates the ties that are required in Sections 1614.4.1 through 1614.4.2.4.

1615.4.1 Concrete wall structures. Precast bearing wall structures constructed solely of reinforced or prestressed concrete, or combinations of these shall conform to the requirements of Sections 7.13, 13.3.8.5 and 16.5 of ACI 318.

❖ This section refers to the ACI 318 requirements that are applicable to concrete wall structures. These provisions establish minimum requirements for tying together the primary structural elements. These ACI 318 structural integrity requirements are already incorporated in the code by the general reference to ACI 318 in Section 1901.2 for the design and construction of structural concrete.

1615.4.2 Other bearing wall structures. Ties in bearing wall structures other than those covered in Section 1615.4.1 shall conform to this section.

❖ The provisions contained in this section have been adapted from requirements contained within the ACI

318 standard for many years. By adapting those requirements to other bearing wall structures, these provisions enhance their overall structural integrity.

1615.4.2.1 Longitudinal ties. Longitudinal ties shall consist of continuous reinforcement in slabs; continuous or spliced decks or sheathing; continuous or spliced members framing to, within or across walls; or connections of continuous framing members to walls. Longitudinal ties shall extend across interior load-bearing walls and shall connect to exterior load-bearing walls and shall be spaced at not greater than 10 feet (3038 mm) on center. Ties shall have a minimum nominal tensile strength, T_T, given by Equation 16-41. For ASD the minimum nominal tensile strength shall be permitted to be taken as 1.5 times the allowable tensile stress times the area of the tie.

$T_T = w\,LS \le \alpha_T S$ **(Equation 16-41)**

where:

L = The span of the horizontal element in the direction of the tie, between bearing walls, feet (m).

w = The weight per unit area of the floor or roof in the span being tied to or across the wall, psf (N/m^2).

S = The spacing between ties, feet (m).

α_T = A coefficient with a value of 1,500 pounds per foot (2.25 kN/m) for masonry bearing wall structures and a value of 375 pounds per foot (0.6 kN/m) for structures with bearing walls of cold-formed steel light-frame construction.

❖ Requirements for tying together horizontal elements in the longitudinal direction (spanning between bearing walls—noted as “L” in Figure 1615.4) provides for transfer and/or redistribution of loads in the event there is loss of support.

1615.4.2.2 Transverse ties. Transverse ties shall consist of continuous reinforcement in slabs; continuous or spliced decks or sheathing; continuous or spliced members framing to, within or across walls; or connections of continuous framing members to walls. Transverse ties shall be placed no farther apart than the spacing of load-bearing walls. Transverse ties shall have minimum nominal tensile strength T_T, given by Equation 16-46. For ASD the minimum nominal tensile strength shall be permitted to be taken as 1.5 times the allowable tensile stress times the area of the tie.

❖ Requirements for ties in the transverse direction at bearing walls (noted as “T” in Figure 1615.4) provides for transfer and/or redistribution of loads in the event there is loss of support.

1615.4.2.3 Perimeter ties. Perimeter ties shall consist of continuous reinforcement in slabs; continuous or spliced decks or sheathing; continuous or spliced members framing to, within

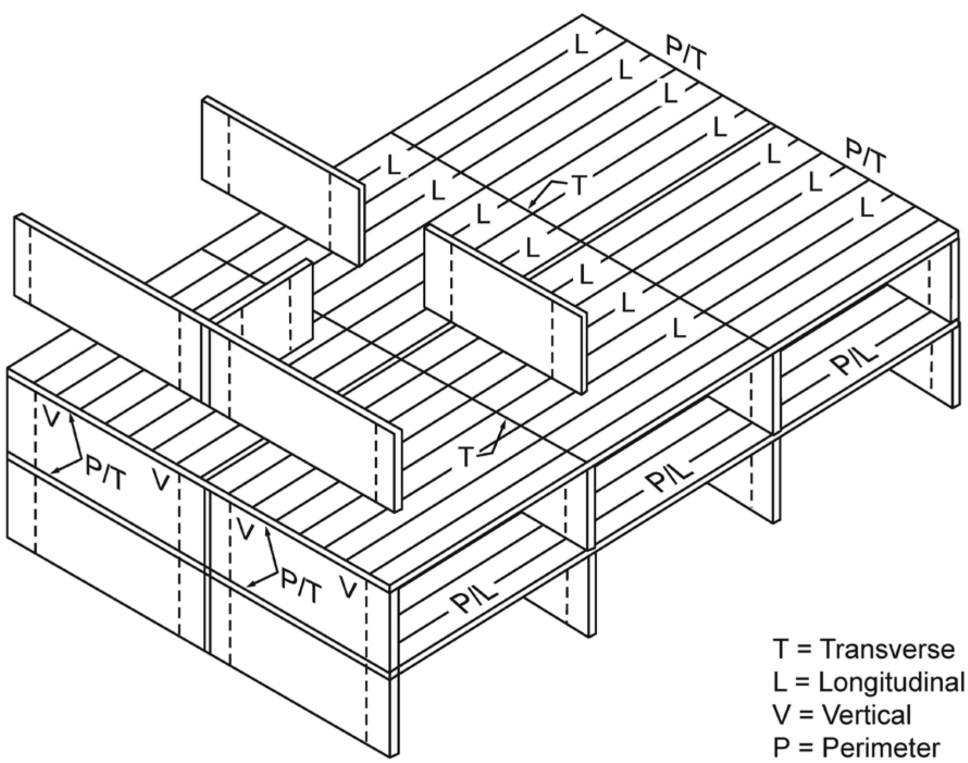

FIGURE 1615.4
LONGITUDINAL, PERIMETER, TRANSVERSE AND VERTICAL TIES

or across walls; or connections of continuous framing members to walls. Ties around the perimeter of each floor and roof shall be located within 4 feet (1219 mm) of the edge and shall provide a nominal strength in tension not less than T_p, given by Equation 16-42. For ASD the minimum nominal tensile strength shall be permitted to be taken as 1.5 times the allowable tensile stress times the area of the tie.

$T_p = 200w \leq \beta_T$ **(Equation 16-42)**

For SI: $T_p = 90.7w \leq \beta_T$

where:

w = As defined in Section 1615.4.2.1.

β_T = A coefficient with a value of 16,000 pounds (7200 kN) for structures with masonry bearing walls and a value of 4,000 pounds (1300 kN) for structures with bearing walls of cold-formed steel light-frame construction.

❖ Requirements for tying together horizontal elements at the perimeter in both the longitudinal and transverse directions (noted as "P/L" and "P/T" in Figure 1615.4) provides for transfer and/or redistribution of load in the event there is loss of support.

1615.4.2.4 Vertical ties. Vertical ties shall consist of continuous or spliced reinforcing, continuous or spliced members, wall sheathing or other engineered systems. Vertical tension ties shall be provided in bearing walls and shall be continuous over the height of the building. The minimum nominal tensile strength for vertical ties within a bearing wall shall be equal to the weight of the wall within that *story* plus the weight of the diaphragm tributary to the wall in the *story* below. No fewer than two ties shall be provided for each wall. The strength of each tie need not exceed 3,000 pounds per foot (450 kN/m) of wall tributary to the tie for walls of masonry construction or 750 pounds per foot (140 kN/m) of wall tributary to the tie for walls of cold-formed steel light-frame construction.

❖ The additional requirement for continuous vertical ties enhances the performance of bearing walls in unforeseen events (noted as "V" in Figure 1615.4).

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Chapter 17: Special Inspections and Tests

General Comments

In this chapter, the code sets minimum quality standards for the acceptance of materials used in building construction. It also establishes requirements for special inspections, structural observations and load testing.

Section 1701 contains the scope statement and general statement for new and used materials.

The terms primarily related to this chapter are listed in Section 1702 and their definitions are provided in Chapter 2.

Section 1703 addresses the approval process and labeling by approved agencies. Special inspections, contractor responsibility and structural observation are specified in Section 1704.

Section 1704 also includes the detailed requirements pertaining to the statement of special inspections.

Section 1705 contains detailed special inspection and verification requirements for various building elements based on the type of construction involved. Included are special inspection and verification requirements for steel, concrete, masonry, wood, soils, deep foundations, wind, seismic, fire resistance, Exterior Insulation and Finish Systems (EIFS) and smoke control. Structural testing and qualification for seismic resistance is also addressed in Section 1705.

The general requirements for determining the design strengths of materials are in Section 1706.

Section 1707 provides for an alternative test procedure in the absence of approved standards.

Provisions for a test load are addressed in Section 1708.

Section 1709 includes requirements for field load testing of a structure.

Preconstruction load testing of materials and methods of construction that are not capable of being designed by an approved analysis is covered by Section 1710.

Section 1711 includes specific material and test standards for joist hangers and concrete and clay tile roof covering.

Chapter 17 provides information regarding the evaluation, inspection and approval process for any material or system proposed for use as a component of a structure. These are general requirements that expand on the requirements of Chapter 1 relating to the roles and responsibilities of the building official regarding approval of building components. Additionally, the chapter includes general requirements relating to the roles and responsibilities of the owner, contractor, special inspectors and architects or engineers.

Purpose

This chapter provides procedures and criteria for: testing materials or assemblies; labeling materials; systems and assemblies and special inspection and verification of structural assemblies.

SECTION 1701 GENERAL

1701.1 Scope. The provisions of this chapter shall govern the quality, workmanship and requirements for materials covered. Materials of construction and tests shall conform to the applicable standards listed in this code.

❖ This chapter gives provisions for quality, workmanship, testing and labeling of all materials covered within. In general, all construction materials and tests must conform to the standards, or portions thereof, that are referenced in the code. This chapter provides requirements for materials and tests when there are no applicable standards; specific tests and standards are referenced in other chapters of the code. Additionally, this chapter provides basic requirements for labeling construction materials and assemblies, and for special inspection and verification of structural systems and components.

1701.2 New materials. New building materials, equipment, appliances, systems or methods of construction not provided for in this code, and any material of questioned suitability proposed for use in the construction of a building or structure, shall be subjected to the tests prescribed in this chapter and in the *approved* rules to determine character, quality and limitations of use.

❖ Testing is required to be performed on materials that are not specifically provided for in the code. For example, suppose a manufacturer of a sandwich panel consisting of aluminum skins and a foam plastic core wishes to use this panel as an exterior weather covering. The material does not conform to any of the standards referenced in Chapter 14, so an appropriate test protocol must be developed. The same provision for acceptance of alternative materials is already given in Section 104.11. That section provides a strong, definitive statement for performance requirements for alternative materials, requiring the proposed alternative to be equivalent to that prescribed by the code in quality, strength, effectiveness, durability and safety. Section 1701.2 simply reasserts that alternative materials (new materials) may be used, as long as the performance characteristics and quality can be established.

1701.3 Used materials. The use of second-hand materials that meet the minimum requirements of this code for new materials shall be permitted.

❖ Materials and assemblies may be reused, provided that they meet the requirements of the code for new materials (see Section 104.9.1 of the code regarding reuse of materials and equipment). Caution should be exercised in approving a used material for reuse. The applicable material standards must be consulted to determine if certain reuses are prohibited and to determine the characteristics of the used material that must be carefully checked before reuse is approved.

One example is a high-strength structural steel bolt. Reuse of the bolt is restricted by Research Council on Structural Connections (RCSC), *Specification for Structural Joints Using ASTM A 325 or A 490 Bolts*. Even a piece of structural steel, such as a wide flange, would need to be carefully checked to determine that dimensional tolerances for a new piece of structural steel are met (see ASTM A 6 and A 36).

SECTION 1702 DEFINITIONS

1702.1 Definitions. The following terms are defined in Chapter 2:

❖ Definitions facilitate the understanding of code provisions and minimize potential confusion. To that end, this section lists definitions of terms associated with special inspections and testing. Note that these definitions are found in Chapter 2. The use and application of defined terms, as well as undefined terms, are set forth in Section 201.

APPROVED AGENCY.

APPROVED FABRICATOR.

CERTIFICATE OF COMPLIANCE.

DESIGNATED SEISMIC SYSTEM.

FABRICATED ITEM.

INSPECTION CERTIFICATE.

INTUMESCENT FIRE-RESISTANT COATINGS.

MAIN WINDFORCE-RESISTING SYSTEM.

MASTIC FIRE-RESISTANT COATINGS.

SPECIAL INSPECTION.

Continuous special inspection.

Periodic special inspection.

SPECIAL INSPECTOR.

SPRAYED FIRE-RESISTANT MATERIALS.

STRUCTURAL OBSERVATION.

SECTION 1703 APPROVALS

1703.1 Approved agency. An *approved agency* shall provide all information as necessary for the *building official* to determine that the agency meets the applicable requirements.

❖ This section specifies the information that an approved agency must provide to the building official to enable him or her to determine if the agency meets the applicable requirements.

1703.1.1 Independence. An *approved agency* shall be objective, competent and independent from the contractor responsible for the work being inspected. The agency shall also disclose possible conflicts of interest so that objectivity can be confirmed.

❖ As part of the basis for a building official's approval of a particular inspection agency, the agency must demonstrate its objectivity and competence. The judgement of objectivity is linked to the financial and fiduciary independence of the agency. The competence of the agency is judged by its experience and organization, and the experience of its personnel.

For example, suppose that ACME Agency is the inspection agency employed by Builder's, Inc. for factory-built fireplaces. During an investigation of the agency, it is discovered that ACME and Builder's are subsidiaries of the same parent company, Conglomerate, Inc. The inspection agency and manufacturer clearly have a relationship that is undesirable from the standpoint of independence.

1703.1.2 Equipment. An *approved agency* shall have adequate equipment to perform required tests. The equipment shall be periodically calibrated.

❖ As part of judging the ability of a testing or inspection agency, the building official should determine that the agency has the proper equipment to perform the required tests or inspections.

1703.1.3 Personnel. An *approved agency* shall employ experienced personnel educated in conducting, supervising and evaluating tests and/or inspections.

❖ The competence of an inspection or testing agency is also based on the experience and background of its personnel. For example, if 10 engineering graduates form an agency, the building official should question whether or not this newly formed agency is sufficiently experienced to perform the tests.

If the services being provided by the inspection or test agency come within the purview of the professional registration laws of the state in which the building is being constructed, the building official should request evidence that the personnel are qualified to perform the work in accordance with this professional registration law, as well.