SE-007 Design Loads for Residential Buildings

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CHAPTER 3

Design Loads for Residential Buildings

3.1 General

Loads are a primary consideration in any building design because they define the nature and magnitude of hazards or external forces that a building must resist to provide reasonable performance (that is, safety and serviceability) throughout the structure's useful life. The anticipated loads are influenced by a building's intended use (occupancy and function), configuration (size and shape), and location (climate and site conditions). Ultimately, the type and magnitude of design loads affect critical decisions such as material selection, construction details, and architectural configuration. To optimize the value (that is, performance versus economy) of the finished product, therefore, design loads must be applied realistically.

Although the buildings considered in this guide are primarily single-family detached and attached dwellings, the principles and concepts related to building loads also apply to other similar types of construction, such as low-rise apartment buildings. In general, the design loads recommended in this guide are based on applicable provisions of the ASCE 7 standard–*Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010). The ASCE 7 standard represents an acceptable practice for building loads in the United States and is recognized in U.S. building codes. For this reason, the reader is encouraged to become familiar with the provisions, commentary, and technical references contained in the ASCE 7 standard.

In general, the structural design of housing has not been treated as a unique engineering discipline or subjected to a special effort to develop better, more efficient design practices. For that reason, this part of the guide focuses on those aspects of ASCE 7 and other technical resources that are particularly relevant to the determination of design loads for residential structures. The guide provides supplemental design assistance to address aspects of residential construction for which current practice is either silent or in need of improvement. The guide's methods for determining design loads are complete yet tailored to typical residential conditions. As with any design function, the designer must ultimately understand and approve the loads for a given project as well as the overall design methodology, including all its inherent strengths and weaknesses. Because building codes from different jurisdictions can vary in their treatment of design loads, the designer should, as a matter of due diligence, identify variances from both local accepted practice and the applicable building code relative to design loads as presented in this guide, even though the variances may be considered technically sound.

Complete design of a home typically requires the evaluation of several different types of materials, as discussed in chapters 4 through 7. Some material specifications use the allowable stress design (ASD) approach while others use load and resistance factor design (LRFD). Chapter 4 uses the LRFD method for concrete design and the ASD method for masonry design. For wood design, chapters 5, 6, and 7 use ASD. For a single project, therefore, the designer may have to determine loads in accordance with both design formats. This chapter provides load combinations intended for each method. The determination of individual nominal loads is essentially unaffected. Special loads, such as ice loads and rain loads, are not addressed herein. The reader is referred to the ASCE 7 standard and applicable building code provisions regarding special loads.

3.2 Load Combinations

The load combinations in table 3.1 are recommended for use with design specifications based on ASD and LRFD. Load combinations provide the basic set of building load conditions that should be considered by the designer. They establish the proportioning of multiple transient loads that may assume point-in-time values when the load of interest attains its extreme design value. Load combinations are intended as a guide to the designer, who should exercise judgment in any particular application. The load combinations in table 3.1 are appropriate for use with the design loads determined in accordance with this chapter.

The principle used to proportion loads is a recognition that when one load attains its maximum lifetime value, the other loads assume arbitrary point-in-time values associated with the structure’s normal or sustained loading conditions. The advent of LRFD has drawn greater attention to this principle (Ellingwood et al., 1982; Galambos et al., 1982). The proportioning of loads in this chapter for ASD is consistent with design load specifications such as ASCE 7. ASD load combinations found in building codes typically have included some degree of proportioning (that is, D + W + 1/2S) and usually have made allowance for a special reduction for multiple transient loads. Some earlier codes also have permitted allowable material stress increases for load combinations involving wind and earthquake loads. None of these adjustments for ASD load combinations are recommended for use with table 3.1 because the load proportioning is considered sufficient. However, allowable material stress increases that are based upon the duration of the load (that is, wood members under wind loading) may be combined with load proportioning.

Note also that the wind load factor of 1.0 in table 3.1 used for LRFD is consistent with current wind design practice and now recognizes ultimate wind loads when the speeds illustrated in the ASCE 7-10 maps are used. The return period of the design wind speeds for residential buildings along the hurricane-prone coast is now

700 years, and this long return period provides a consistent risk basis for wind design across the country. Many elements of residential design continue to use ASD design level wind speeds, however, primarily because of how products have been tested, rated, and marketed to the industry. Some prescriptive design documents such as the Wood Frame Construction Manual (WFCM) continue to use ASD load combinations in the development of loads provided in the design tables of that document (AWC, 2012). The conversion of LRFD speeds to ASD speeds is ASD speed = LRFD speedx$\sqrt{0.6}$. The conversion of LRFD pressures to ASD pressures is ASD wind pressure = LRFD pressure x 0.6 (the ASD wind load factor). The load factor changes used in ASCE 7-10 are referenced in the 2012 editions of the building codes where ASCE 7-10 is referenced.

The load combinations in table 3.1 are simplified and tailored to specific application in residential construction and the design of typical components and systems in a home. These or similar load combinations often are used in practice as shortcuts to those load combinations that govern the design result. This guide makes effective use of the shortcuts and demonstrates them in the examples provided later in the chapter. The shortcuts are intended only for the design of residential light-frame construction.

TABLE 3.1 ***Typical Load Combinations Used for the Design of Components and Systems***[1]

Component or System	ASD Load Combinations	LRFD Load Combinations
Foundation wall (gravity and soil lateral loads)	$D + H$ $D + H + 0.75\ (L_r \text{ or } S) + 0.75L^2$	$1.2D + 1.6H$ $1.2D + 1.6H + 1.6L^2 + 0.5(L_r + S)$ $1.2D + 1.6H + 1.6(L_r \text{ or } S) + L^2$
Headers, girders, joists, interior load-bearing walls and columns, footings (gravity loads)	$D + 0.75\ L^2 + 0.75\ (L_r \text{ or } S)$ $D + 0.75\ (L_r \text{ or } S) + 0.75\ L^2$	$1.2D + 1.6L^2 + 0.5\ (L_r \text{ or } S)$ $1.2D + 1.6(L_r \text{ or } S) + L^2$
Exterior load-bearing walls and columns (gravity and transverse lateral load) [3]	Same as immediately above, plus $0.6D + 0.6W$ $D + 0.7E + 0.75L^2 + 0.75S^4$	Same as immediately above, plus $1.2D + 1.0W$ $1.2D + 1.0E + L^2 + 0.2S^4$
Roof rafters, trusses, and beams; roof and wall sheathing (gravity and wind loads)	$D + (L_r \text{ or } S)$ $0.6D + 0.6W_u{}^5$ $0.6D + 0.6W$	$1.2D + 1.6(L_r \text{ or } S)$ $0.9D + 1.0W_u{}^5$ $1.2D + 1.0W$
Floor diaphragms and shear walls (in-plane lateral and overturning loads) [6]	$0.6D + (0.6W \text{ or } 0.7E)$	$0.9D + (1.0W \text{ or } 1.0E)$

Notes:

[1]The load combinations and factors are intended to apply to nominal design loads defined as follows: D = estimated mean dead weight of the construction; E = design earthquake load; H = design lateral pressure for soil condition/type; L = design floor live load; L_r = maximum roof live load anticipated from construction/maintenance;; S = design roof snow load; and W = design wind load. The design or nominal loads should be determined in accordance with this chapter.

[2]Attic loads may be included in the floor live load, but a 10-psf attic load typically is used only to size ceiling joists adequately for access purposes. If the attic is intended for storage, however, the attic live load (or some portion) should also be considered for the design of other elements in the load path.

[3]The transverse wind load for stud design is based on a localized component and cladding wind pressure; D + W provides an adequate and simple design check representative of worst-case combined axial and transverse loading. Axial forces from snow loads and roof live loads should usually not be considered simultaneously with an extreme wind load because they are mutually exclusive on residential sloped roofs. Further, in most areas of the United States, design winds are produced by either hurricanes or thunderstorms; therefore, these wind events and snow are mutually exclusive because they occur at different times of the year.

[4]For walls supporting heavy cladding loads (such as brick veneer), an analysis of earthquake lateral loads and combined axial loads should be considered; however, this load combination rarely governs the design of light-frame construction.

[5]W_u is wind uplift load from negative (that is, suction) pressures on the roof. Wind uplift loads must be resisted by continuous load path connections to the foundation or until offset by D.

[6]The 0.6 reduction factor on D is intended to apply to the calculation of net overturning stresses and forces. For wind, the analysis of overturning should also consider roof uplift forces unless a separate load path is designed to transfer those forces.

3.3 Dead Loads

Dead loads consist of the permanent construction material loads comprising the roof, floor, wall, and foundation systems, including claddings, finishes, and fixed equipment. The values for dead loads in table 3.2 are for commonly used materials and constructions in light-frame residential buildings. Dead loads are given as nominal or ASD-level loads. Table 3.3 provides values for common material densities and may be useful in calculating dead loads more accurately. The design examples in section 3.12 demonstrate the straightforward process of calculating dead loads.

TABLE 3.2 Dead Loads for Common Residential Construction[1]

Roof Construction Light-frame wood roof with wood structural panel sheathing and 1/2-inch gypsum board ceiling (2 psf) with asphalt shingle roofing (3 psf) - with conventional clay/tile roofing - with lightweight tile - with metal roofing - with wood shakes - with tar and gravel	15 psf 27 psf 21 psf 14 psf 15 psf 18 psf
Floor Construction Light-frame 2x12 wood floor with 3/4-inch wood structural panel sheathing and 1/2-inch gypsum board ceiling (without 1/2-inch gypsum board, subtract 2 psf from all values) with carpet, vinyl, or similar floor covering - with wood flooring - with ceramic tile - with slate	10 psf[2] 12 psf 15 psf 19 psf
Wall Construction Light-frame 2x4 wood wall with 1/2-inch wood structural panel sheathing and 1/2-inch gypsum board finish (for 2x6, add 1 psf to all values) - with vinyl or aluminum siding - with lap wood siding - with 7/8-inch portland cement stucco siding - with thin-coat stucco on insulation board - with 3-1/2-inch brick veneer Interior partition walls (2x4 with 1/2-inch gypsum board applied to both sides)	6 psf 7 psf 8 psf 15 psf 9 psf 45 psf 6 psf

Foundation Construction	Masonry[3] Hollow	Masonry[3] Solid or Full Grout	Concrete
6-inch-thick wall	28 psf	60 psf	75 psf
8-inch-thick wall	36 psf	80 psf	100 psf
10-inch-thick wall	44 psf	100 psf	123 psf
12-inch-thick wall	50 psf	125 psf	145 psf
6-inch x 12-inch concrete footing			73 plf
6-inch x 16-inch concrete footing			97 plf
8-inch x 24-inch concrete footing			193 plf

psf = pounds per square foot

Notes:
[1]For unit conversions, see appendix B.
[2]Value also used for roof rafter construction (that is, cathedral ceiling).
[3]For partially grouted masonry, interpolate between hollow and solid grout in accordance with the fraction of masonry cores that are grouted.

TABLE 3.3 Densities for Common Residential Construction Materials[1]

Aluminum	170 pcf
Copper	556 pcf
Steel	492 pcf
Concrete (normal weight with light reinforcement)	145–150 pcf
Masonry, grout	140 pcf
Masonry, brick	100–130 pcf
Masonry, concrete	85–135 pcf
Glass	160 pcf
Wood (approximately 10 percent moisture content)[2]	
- spruce-pine-fir (G = 0.42)	29 pcf
- spruce-pine-fir, south (G = 0.36)	25 pcf
- southern yellow pine (G = 0.55)	38 pcf
- Douglas fir–larch (G = 0.5)	34 pcf
- hem-fir (G = 0.43)	30 pcf
- mixed oak (G = 0.68)	47 pcf
Water	62.4 pcf
Structural wood panels	
- plywood	36 pcf
- oriented strand board	36 pcf
Gypsum board	50 pcf
Stone	
- Granite	96 pcf
- Sandstone	82 pcf
Sand, dry	90 pcf
Gravel, dry	104 pcf

pcf = pounds per cubic foot

Notes:
[1]For unit conversions, see appendix B.
[2]The equilibrium moisture content of lumber is usually not more than 10 percent in protected building construction. The specific gravity, G, is the decimal fraction of dry wood density relative to that of water; therefore, at a 10 percent moisture content, the density of wood is 1.1(G)(62.4 lbs/ft^3). The values given are representative of average densities and may easily vary by as much as 15 percent, depending on lumber grade and other factors.

3.4 Live Loads

Live loads are produced by the use and occupancy of a building. Loads include those from human occupants, furnishings, nonfixed equipment, storage, and construction and maintenance activities. Table 3.4 provides recommended design live loads for residential buildings. Live loads also are given as nominal or ASD-level loads. Example 3.1 in section 3.10 demonstrates use of those loads and the load combinations specified in table 3.1, along with other factors discussed in this section. As required to adequately define the loading condition, loads are presented in terms of uniform area loads (in pounds per square foot: psf), concentrated loads (in pounds: lbs), and uniform line loads (in pounds per linear foot: plf). The uniform and concentrated live loads should not be applied simultaneously in a structural evaluation. Concentrated loads should be applied to a small area or surface consistent with the application and should be located or directed to give the maximum load effect possible in end-use conditions. For example, the stair concentrated load of 300 pounds should be applied to the center of the stair tread between supports. The concentrated wheel load of a vehicle on a garage slab or floor should be applied to all areas or members subject to a wheel or jack load, typically using a loaded area of about 20 square inches.

TABLE 3.4 Live Loads for Residential Construction[1]

Application	Uniform Load	Concentrated Load
Roof[2]		
Slope ≥ 4:12	15 psf	250 lbs
Flat to 4:12 slope	20 psf	250 lbs
Attic[3]		
Without storage	10 psf	250 lbs
With storage	20 psf	250 lbs
Floors		
Bedroom areas[3,4]	30 psf	300 lbs
Other areas	40 psf	300 lbs
Garages	50 psf	2,000 lbs (passenger cars, vans, light trucks)
Decks and balconies	40 psf[7]	
Stairs	40 psf	300 lbs
Guards and handrails	50 plf[5]	200 lbs
Guard in-fill components	50 psf[6]	
Grab bars	N/A	250 lbs

lbs = pounds; plf = pounds per linear foot; psf = pounds per square foot

Notes:

[1]Live load values should be verified relative to the locally applicable building code.

[2]Roof live loads are intended to provide a minimum load for roof design in consideration of maintenance and construction activities. They should not be considered in combination with other transient loads (for example, floor live load, wind load) when designing walls, floors, and foundations. A 15-psf roof live load is recommended for residential roof slopes greater than 4:12; refer to ASCE 7-10 for an alternate approach.

[3]Loft sleeping and attic storage loads should be considered only in areas with a clear height greater than about 3.5 feet. The concept of a “clear height” limitation on live loads is logical, but it may not be universally recognized.

[4]Some codes require 40 psf for all floor areas.

[5] ASCE 7-10 indicates that this load does not have to be considered for one- and two-family dwellings.

[6] The applied normal load on an area is not to exceed 12 in. by 12 in.

[7] ASCE 7 requirements may be more stringent.

The floor live load on any given floor area may be reduced in accordance with equation 3.4-1 (Harris, Corotis, and Bova, 1981). Live load reductions also are allowed for multiple floors in ASCE 7-10. The equation applies to floor and support members, such as beams or columns (see table 3-5), which experience floor loads from a total tributary floor area greater than 200 square feet. This equation also is in chapter 4 of ASCE 7-10, which covers live load design.

Equation 3.4-1

$$L = L_o\left[0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right]$$

where

L = reduced design live load per ft^2 of area supported by the member
K_{LL} = live load element factor
L_o = unreduced design live load per ft^2 of area supported by the member
A_T = the tributary area in ft^2

L shall not be less than $0.50L_0$ for members supporting one floor and not less than $0.40L_0$ for members supporting two or more floors.

TABLE 3.5 Live Load Element Factor, K_{LL}

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

*In lieu of the preceding values, K_{LL} may be calculated.

Note also that the nominal design floor live load in table 3.4 includes both a sustained and a transient load component. The sustained component is that load typically present at any given time and includes the load associated with normal human occupancy and furnishings. For residential buildings, the mean sustained live load is about 6 psf but typically varies from 4 to 8 psf (Chalk and Corotis, 1978). The mean transient live load for dwellings also is about 6 psf but may be as high as 13 psf. A total design live load of 30 to 40 psf is therefore fairly conservative.

3.5 Soil Lateral Loads

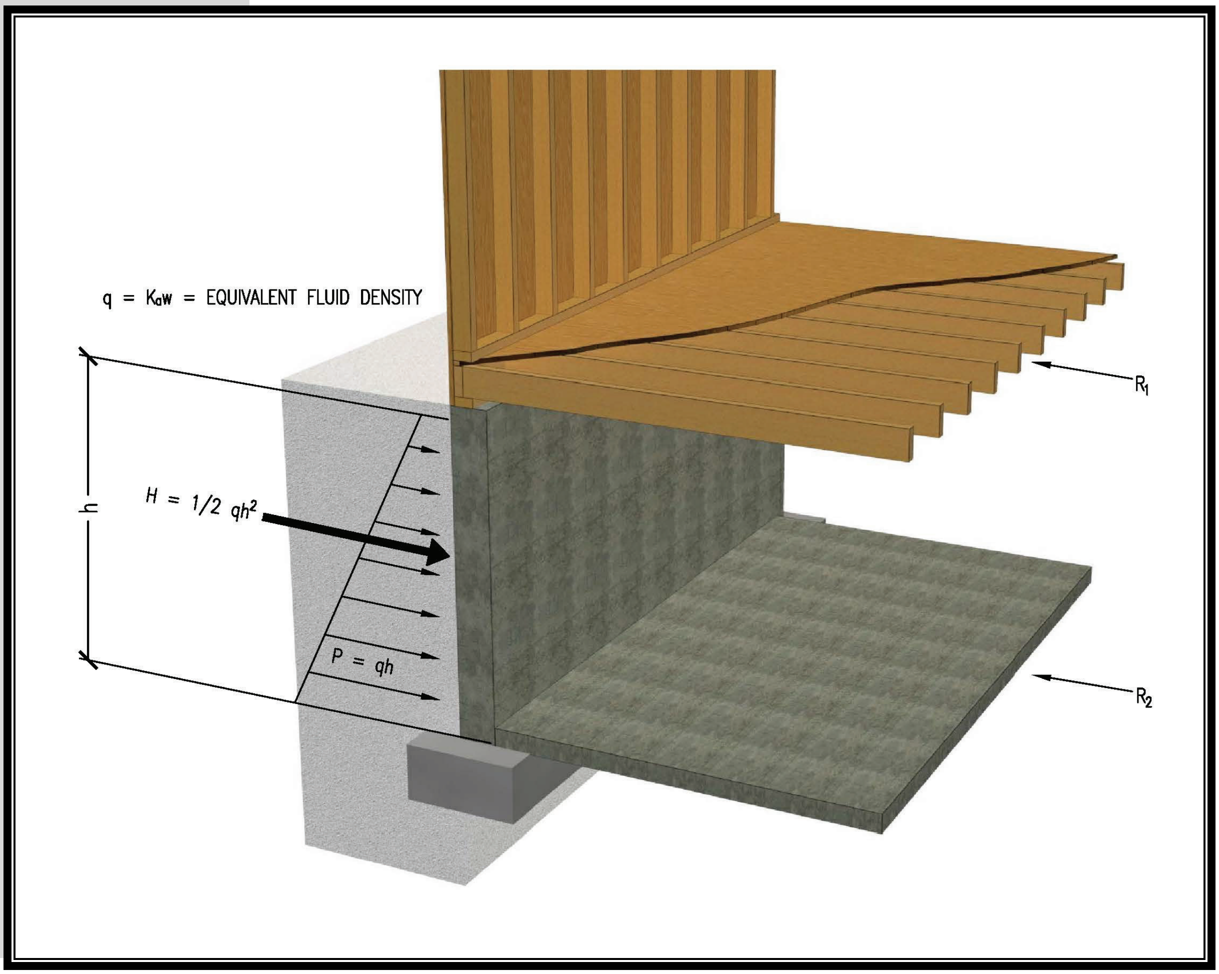
The lateral pressure exerted by earth backfill against a residential foundation wall (basement wall) can be calculated with reasonable accuracy on the basis of theory, but only for conditions that rarely occur in practice (Peck, Hanson, and Thornburn, 1974; University of Alberta, 1992). Theoretical analyses usually are based on homogeneous materials that demonstrate consistent compaction and behavioral properties. Such conditions rarely are experienced in typical residential construction projects.

The most common method of determining lateral soil loads on residential foundations follows Rankine's (1857) theory of earth pressure and uses what is known as the Equivalent Fluid Density (EFD) method. As shown in figure 3.1, pressure distribution is assumed to be triangular and to increase with depth.

In the EFD method, the soil unit weight w is multiplied by an empirical coefficient K_a to account for the soil is not actually fluid and the pressure distribution is not necessarily triangular. The coefficient K_a is known as the active Rankine pressure coefficient. The EFD is determined as shown in equation 3.5-1.

Equation 3.5-1 $$q = K_a w$$

FIGURE 3.1 ***Triangular Pressure Distribution on a Basement Foundation Wall***



For the triangular pressure distribution shown in figure 3.1, the pressure, P in psf, at depth, h in feet, is determined by equation 3.5-2, and the resultant force, H in lbs, at depth, h in feet, is determined by equation 3.5-3. The factor q is the EFD as discussed above.

$$P = qh$$

Equation 3.5-2

The total active soil force (pounds per linear foot of wall length) is—

Equation 3.5-3

$$H = \frac{1}{2}(qh)(h) = \frac{1}{2}qh^2$$

where
h = the depth of the unbalanced fill on a foundation wall
H = the resultant force (plf) applied at a height of h/3 from the base of the unbalanced fill because the pressure distribution is assumed to be triangular

The EFD method is subject to judgment as to the appropriate value of the coefficient K_a. The values of K_a in table 3.6 are recommended for the determination of lateral pressures on residential foundations for various types of backfill materials placed with light compaction and good drainage. Given the long-time use of a 30 pounds per cubic foot (pcf) EFD in residential foundation wall prescriptive design tables (ICC, 2012), the values in table 3.6 may be considered somewhat conservative for typical conditions. A relatively conservative safety factor of 3 to 4 is typically applied to the design of unreinforced or nominally reinforced masonry or concrete foundation walls (ACI, 2011). Therefore, at imminent failure of a foundation wall, the 30 psf design EFD would correspond to an active soil lateral pressure determined by using an EFD of about 90 to 120 pcf or more. The design examples in chapter 4 demonstrate the calculation of soil loads.

TABLE 3.6 ***Values of K_a, Soil Unit Weight, and Equivalent Fluid Density by Soil Type***[1,2,3]

Type of Soil[4] (Unified Soil Classification)	Active Pressure Coefficient (K_a)	Soil Unit Weight (pcf)	Equivalent Fluid Density (pcf)
Sand or gravel (GW, GP, GM, SW, SP)	0.26	115	30
Silty sand, silt, and sandy silt (GC, SM)	0.35	100	35
Clay-silt, silty clay (SM-SC, SC, ML, ML-CL)	0.45	100	45
Clay[5] (CL, MH, CH)	0.60	100	60

Notes:
[1] Values are applicable to well-drained foundations with less than 10 feet of backfill placed with light compaction or natural settlement, as is common in residential construction. The values do not apply to foundation walls in flood-prone environments; in such cases, an equivalent fluid density value of 80 to 90 pcf would be more appropriate (HUD, 1977).
[2] Values are based on the *Standard Handbook for Civil Engineers*, 3rd ed. (Merritt, 1983), and on research on soil pressures reported in *Thin Wall Foundation Testing*, Department of Civil Engineering, University of Alberta, (March 1992). The designer should note that the values for soil equivalent fluid density differ from those recommended in ASCE 7-10 but are nonetheless compatible with current residential building codes, design practice, and the stated references.
[3] These values do not consider the significantly higher loads that can result from expansive clays and the lateral expansion of moist, frozen soil. Such conditions should be avoided by eliminating expansive clays adjacent to the foundation wall and providing for adequate surface and foundation drainage.
[4] Organic silts and clays and expansive clays are unsuitable for backfill material.
[5] Backfill in the form of clay soils (non-expansive) should be used with caution on foundation walls with unbalanced fill heights greater than 3 to 4 feet and on cantilevered foundation walls with unbalanced fill heights greater than 2 to 3 feet.

Depending on the type and depth of backfill material and the manner of its placement (see table 3.7), common practice in residential construction is to allow the backfill soil to consolidate naturally by providing an additional 3 to 6 inches of fill material. The additional backfill ensures that surface water drainage away from the foundation remains adequate (that is, the grade slopes away from the building). It also helps avoid heavy compaction that could cause undesirable loads on the foundation wall during and after construction. If soils are heavily compacted at the ground surface or compacted in lifts to standard Proctor densities greater than approximately 85 percent of optimum (ASTM, 2012), the standard 30 pcf EFD assumption may be inadequate. In cases in which the backfill supports exterior slabs, patios, stairs, or other items, however, some amount of compaction is required unless the structures are supported on a separate foundation bearing on undisturbed ground.

Some remediation may be necessary in areas that contain marine clay or other expansive soils. In very moist conditions, these soils can place significant lateral loads against foundation walls. The soils may need to be replaced with soil of lower clay content or the moisture levels must be stabilized to reduce excessive lateral pressures.

TABLE 3.7 Lateral Soil Load

Description of Backfill Material[3]	Unified Soil Classification	Design Lateral Soil Load[1] (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-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; clayey silts	ML	45	100
Inorganic silt-clay mixes	ML-CL	60	100
Inorganic clays of low to medium plasticity	CL	60	100
Organic silts and silt clays of low plasticity	OL	[2]	[2]
Inorganic clayey silts; elastic silts	MH	[2]	[2]
Inorganic clays of high plasticity	CH	[2]	[2]

Notes:
[1] Design lateral soil loads are given for moist conditions for the specified soils at their optimum densities. Actual field conditions shall govern.
[2] Unsuitable as backfill material
[3] The definition and classification of soil materials is in accordance with ASTM D2487.

3.6 Wind Loads

3.6.1 General

Wind produces dynamic loads on a structure at highly variable magnitudes. The variation in pressures at different locations on a building is complex to the point that pressures may become too analytically intensive for precise consideration in design. Wind load specifications attempt to simplify the design problem by considering basic static pressure zones on a building representative of peak loads that are likely to be experienced. The peak pressures in one zone for a given wind direction may not, however, occur simultaneously with peak pressures in other zones. For some pressure zones, the peak pressure depends on a narrow range of wind directions; therefore, the wind directionality effect must also be factored into determining risk-consistent wind loads on buildings. Characteristics of the building site and the surrounding area, such as exposure and topography, also play a large role in determining the peak pressures on the structure and should be carefully considered. In fact, most modern wind load specifications account for wind directionality and other effects in determining nominal design loads in some simplified form (ASCE, 2010). This section further simplifies wind load design specifications to provide an easy yet effective approach for designing typical residential buildings.

Because they vary substantially over the surface of a building, wind loads are considered at two different scales. On a large scale, the loads produced on the overall building are resisted by a system of structural elements working together to transfer the wind loads acting on the entire structure to the ground, a system known as the main wind force-resisting system (MWFRS). The MWFRS of a home includes the shear walls and diaphragms that create the lateral force-resisting system (LFRS) as well as the structural systems, such as trusses, that experience loads from external and internal pressures generated on the building. The wind loads applied to the MWFRS account for the area-averaging effects of time-varying wind pressures on the surface or surfaces of the building.

Wind pressures are greater on certain localized surface areas of the building, particularly near abrupt changes in building geometry (for example, eaves, ridges, and corners). Those higher wind pressures can occur on smaller areas, particularly affecting the loads carried by components and cladding (for example, sheathing, windows, doors, purlins, and studs). The components and cladding (C&C) transfer localized time-varying loads to the MWFRS, at which point the loads average out both spatially and temporally since, at a given time, some components may be at near peak loads while others are at substantially less than peak.

In light-framed wood structural systems, the distinction between MWFRS and C&C is not as clear-cut as in other buildings. In some cases, structural components may act as MWFRS and as C&C, depending on situations. The designer must consider which elements of the building must be treated as C&C, part of the MWFRS, or both. As indicated, parts of the MWFRS that collect and transfer lateral loads in shear walls and floors or roof diaphragms consist of wall studs, sheathing, and trusses, and these elements as a system must be designed for MWFRS lateral loads; but the studs, sheathing, and truss chords must be designed for the direct loading from wind as C&C. Thus, the stud size and connection to top and bottom plates must be designed for C&C pressures, yet the entire wall system, especially the sheathing thickness and the nailing attachment of the sheathing to the studs, must be designed to resist the shear forces created by the lateral loads.

The next section presents a simplified method for determining both MWFRS and C&C wind loads. Because the loads in section 3.6.2 are determined for specific applications, the calculation of MWFRS and C&C wind loads is implicit in the values provided. Design example 3.2 in section 3.12 demonstrates the calculation of wind loads by applying the simplified method in the following section to several design conditions associated with wind loads and the load combinations presented in table 3.1.

3.6.2 Determination of Wind Loads on Residential Buildings

The following method for the design of residential buildings is based on a simplification of the ASCE 7-10 wind provisions (ASCE, 2010); therefore, the wind loads are not exact duplicates. Lateral loads and roof uplift loads are determined by using a projected area approach. Other wind loads are determined for specific components or assemblies that comprise the exterior building envelope. Determining design wind loads on a residential building and its components requires five steps.

Step 1: Determine site design wind speed and basic velocity pressure

From the wind map in figure 3.2 (refer to ASCE 7-10 for a more detailed map for risk category II buildings), select a design wind speed for the site (ASCE, 2010), or, alternatively, find a location-specific wind speed from the local building code office or by using www.atcouncil.org/windspeed. The wind speed map in ASCE 7-10 (figure 3.2) includes the most accurate data and analysis available regarding design wind speeds in the United States. The ASCE 7-10 wind speeds are higher than those used in older design wind maps. The difference results solely from using ultimate wind speeds developed for use with 700-year return periods for risk category II buildings that include residential uses. The speeds correspond to approximately a 7 percent probability of exceedance in 50 years. The design 3-second peak gust wind speeds are 110 to 115 miles per hour (mph) in most of the United States; however, along the hurricane-prone Gulf and Atlantic coasts, the design wind speeds range from 115 to 180 mph. The wind speeds are standardized for exposure C conditions at 33 feet (10 meters). Tornadoes have not been considered in the design wind speeds presented in figure 3.2. Design loads for tornadoes are still in the development stage, and discussion of the latest knowledge is provided in section 3.10.

Once the nominal design wind speed in terms of peak 3-second gust is determined, the designer can select the basic velocity pressure, in accordance with table 3.8. The basic velocity pressure is a reference wind pressure to which coefficients are applied to determine the surface pressures on a building. Velocity pressures in table 3.8 are based on typical conditions for residential construction, namely, suburban terrain (exposure B) and relatively flat or rolling terrain without topographic wind speed-up effects.

FIGURE 3.2a ***Basic Design Wind Speed Map from ASCE 7-10***

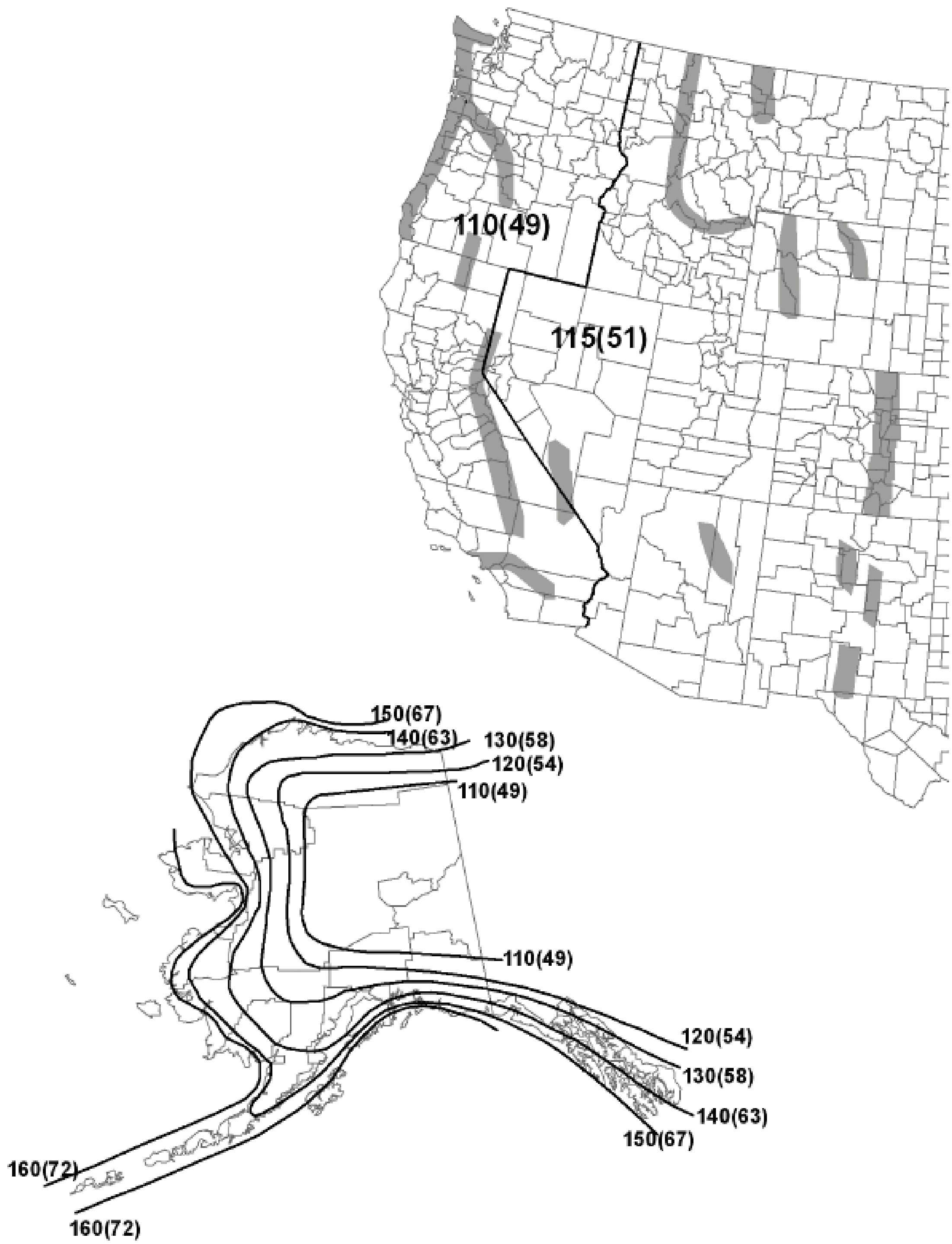


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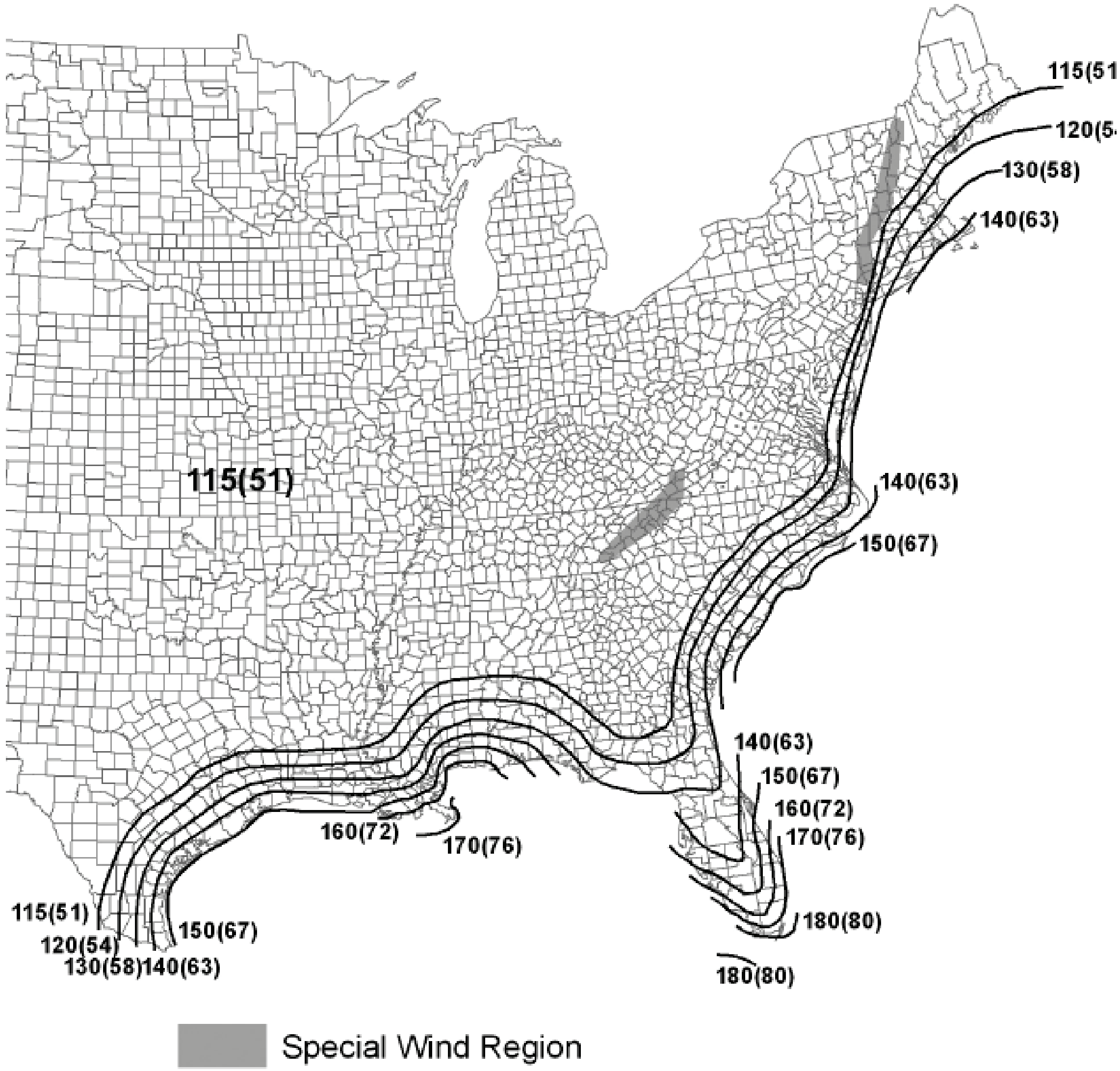
FIGURE 3.2b Basic Design Wind Speed Map from ASCE 7-10



Location	V mph	(m/s)
Guam	195	(87)
Virgin Islands	165	(74)
American Samoa	160	(72)
Hawaii – Special Wind Region Statewide	130	(58)

Notes:

1. Values are design 3-second gust wind speed 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).

TABLE 3.8 ***Basic Wind Velocity Pressures (psf) for Suburban Terrain[1] (MWFRS)***

Design Wind Speed, V (mph, peak gust)	One-Story Building (15') ($K_Z = 0.57$)[2]	Two-Story Building (30') ($K_Z = 0.7$)[2]	Three-Story Building (45') ($K_Z = 0.78$)
110	15	18	21
115	16	20	22
120	18	22	24
130	21	26	29
140	24	30	33
150	28	34	38
160	32	39	43
170	36	44	49
180	40	49	55

mph = miles per hour; MWFRS = main wind force-resisting system; psf = pounds per square foot.

Notes:
[1]Velocity pressure (psf) equals $0.00256\ K_D\ K_Z V^2$, where K_Z is the velocity pressure exposure coefficient associated with the vertical wind speed profile in suburban terrain (exposure B) at the mean roof height of the building. K_D is the wind directionality factor, with a default value of 0.85. All pressures have been rounded to nearest whole psf.
[2] To be compliant with ASCE 7-10, a minimum K_z of 0.7 should be applied to determine velocity pressure for one- and two-story buildings in exposure B (suburban terrain) for the design of components and cladding, in exposure B when the envelope procedure is used for the MWFRS, or when designing components and cladding.

Step 2: Adjustments to the basic velocity pressure

If appropriate, the basic velocity pressure from step 1 should be adjusted in accordance with the factors below. The adjustments are cumulative.

Open exposure. The wind pressure values in table 3.8 are based on typical residential exposures to wind (exposure B). If a site is located in generally open, flat terrain with few obstructions to the wind in most directions (exposure C), the designer should multiply the values in table 3.8 by a factor of 1.4. Exposure to a body of water (that is, an ocean or lake) increases wind pressures more because of reduced friction at the surface (exposure D). The values in table 3.8 should be multiplied by a factor of 1.7 to account for this increased pressure for exposure D conditions. The factor may be used to adjust wind loads according to the exposure related to the specific directions of wind approach to the building. The wind exposure conditions used in this guide are derived from ASCE 7-10, and more information about how to determine these exposures is provided in the ASCE 7-10 commentary.

Wind directionality. As noted, the direction of the wind in a given event does not create peak loads (which provide the basis for design pressure coefficients) simultaneously on all building surfaces. In some cases, the pressure zones with the highest design pressures are extremely sensitive to wind direction. In accordance with ASCE 7-10, the velocity pressures in table 3.8 are based on a directionality adjustment of 0.85.

Topographic effects. If topographic wind speed-up effects are likely because a structure is located near the crest of a protruding hill or cliff, the designer should consider using the topographic factor provided in ASCE 7-10. Wind loads can be increased for buildings sited in particularly vulnerable locations relative to topographic features that cause localized wind speed-up for specific wind directions.

The *International Residential Code* (IRC; ICC 2011) provides a “Simplified Topographic Wind Speed-up Method” for the K_{zt} factor where required. The simplified method in the IRC is based on the wind speed-up effect for cliff edges, the most vulnerable of the three types of features (hills, ridges or escarpments), and on certain terrain feature heights and dwelling locations. If a more accurate and potentially less conservative determination of an adjusted design wind speed is desired, the designer can apply the ASCE 7-10 provisions for adjusting the wind speed to account for the K_{zt} factor, where required.

Step 3: Determine lateral wind pressure coefficients

Lateral pressure coefficients in table 3.9 are composite pressure coefficients that combine the effect of positive pressures on the windward face of the building and negative (suction) pressures on the leeward faces of the building. The lateral pressure coefficients are the total effect of the shape factor (Cp) and the gust effect factor (G). When multiplied by the velocity pressure from steps 1 and 2, the selected pressure coefficient provides a single wind pressure that is applied to the vertical projected area of the roof and wall, as indicated in table 3.9. The resulting load is then used to design the home’s LFRS (see chapter 6). The lateral wind load must be determined for the two orthogonal directions on the building (that is, parallel to the ridge and perpendicular to the ridge), using the vertical projected area of the building for each direction. Lateral loads are then assigned to various systems (for example, shear walls, floor diaphragms, and roof diaphragms) by use of tributary areas or other methods described in chapter 6.

This method can be used for determining shear loads because the internal pressures in the building cancel out and do not affect the shear loads. Overturning moments and the design of wall studs and lateral out-of-plane wall loads at the roof-to-wall connection must consider the effects of internal pressure, however; thus, the projected area method is not useful for those calculations. See step 4 for additional information.

TABLE 3.9 ***Lateral Pressure Coefficients for Application to Vertical Projected Areas***

Application	Lateral Pressure Coefficients
Roof Vertical Projected Area (by slope)	
Flat	0.0
3:12	0.43
6:12	0.77
≥9:12	0.85
Wall Projected Area	1.1

Step 4: Determine wind pressure coefficients for components and assemblies

The pressure coefficients in table 3.9 are derived from ASCE 7-10, based on the assumption that the building is enclosed and not subject to higher internal pressures that may result from a windward opening in the building. Using the values in table 3.9 greatly simplifies the more detailed methodology described in ASCE 7-10; as a result, some numbers are "rounded." With the exception of the roof uplift coefficient, all pressures calculated with the coefficients are intended to be applied perpendicular to the building surface area that is tributary to the element of concern; thus, the wind load is applied perpendicular to the actual building surface, not to a projected area. The roof uplift pressure coefficient is used to determine a single wind pressure that may be applied to a horizontal projected area of the roof to determine roof tie-down connection forces.

For buildings in hurricane-prone regions subject to wind-borne debris, the GC_p values in table 3.10 are still valid, but the glazed openings in the building must be protected from the possibility of damage by wind-borne debris breaching a wall or roof opening, such as a window or skylight. Past versions of ASCE 7 had allowed design for a "partially enclosed" condition using higher internal pressure coefficients in wind-borne debris regions, but this technique allows a potentially significant amount of wind-driven rain into the building, which would still create a near total economic loss. ASCE 7 no longer allows this design method.

Step 5: Determine design wind pressures

Once the basic velocity pressure is determined in step 1 and adjusted in step 2 for exposure and other site-specific considerations, the designer can calculate the design wind pressures by multiplying the adjusted basic velocity pressure by the pressure coefficients selected in steps 3 and 4. The lateral pressures on the MWFRS are based on coefficients from step 3 and are applied to the tributary areas of the LFRS, such as shear walls and diaphragms. The pressures based on coefficients from step 4 are applied to tributary areas of members such as studs, rafters, trusses, and sheathing to determine stresses in members and forces in connections.

TABLE 3.10 ***Wind Pressure Coefficients for Systems and Components (enclosed building)***[1]

Application	Pressure Coefficients (GC_p)[2]
Roof	
Trusses, roof beams, ridge and hip/valley rafters	-0.9, +0.4
Rafters and truss panel members	-1.2, +0.7
Roof sheathing	-2.8, +0.7
Skylights and glazing	-1.2, +1.0
Roof uplift[3]	
- hip roof with slope between 3:12 and 6:12	-0.9
- hip roof with slope greater than 6:12	-0.8
- all other roof types and slopes	-1.2
Windward overhang[4]	+ 0.7
Wall	
All framing members	-1.5, +1.1
Wall sheathing and cladding/siding	-1.6, +1.2
Windows, doors, and glazing	-1.3, +1.2
Garage doors	-1.1, +1.0
Air-permeable claddings[5]	-0.9, 0.8

Notes:

[1]All coefficients include internal pressure in accordance with the assumption of an enclosed building. With the exception of the categories labeled trusses, roof beams, ridge and hip/valley rafters, and roof uplift, which are based on MWFRS loads, all coefficients are based on component and cladding wind loads.

[2]Positive and negative signs represent pressures acting inwardly and outwardly, respectively, from the building surface. A negative pressure is a suction or vacuum. Both pressure conditions should be considered to determine the controlling design criteria.

[3]The roof uplift pressure coefficient is used to determine uplift pressures that are applied to the horizontal projected area of the roof for the purpose of determining uplift tie-down forces. Additional uplift force on roof tie-downs resulting from roof overhangs should also be included. The uplift force must be transferred to the foundation or to a point where it is adequately resisted by the dead load of the building and the capacity of conventional framing connections.

[4]The windward overhang pressure coefficient is applied to the underside of a windward roof overhang and acts upwardly on the bottom surface of the roof overhang. If the bottom surface of the roof overhang is the roof sheathing, or if the soffit is not covered with a structural material on its underside, then the overhang pressure shall be considered additive to the roof sheathing pressure.

[5]Air-permeable claddings allow for pressure relief such that the cladding experiences about two-thirds of the pressure differential experienced across the wall assembly (FPL, 1999). Products that experience reduced pressure include lap-type sidings such as wood, vinyl, aluminum, and other similar sidings. Since these components are usually considered "nonessential," it may be practical to multiply the calculated wind load on any nonstructural cladding by 0.75 to adjust for a serviceability wind load (Galambos and Ellingwood, 1986). Such an adjustment would also be applicable to deflection checks, if required, for other components listed in the table. However, a serviceability load criterion is not included or clearly defined in existing design codes.

3.6.3 Special Considerations in Hurricane-Prone Environments

3.6.3.1 Wind-Borne Debris

The wind loads determined in the previous section assume an enclosed building. If glazing in windows and doors is not protected from wind-borne debris or otherwise designed to resist potential impacts during a major hurricane, a building is more susceptible to structural damage resulting from higher internal pressures that may develop with a windward opening. The potential for water damage to building contents also increases. Openings created in the building envelope during a major hurricane or tornado often are related to unprotected glazing, improperly fastened sheathing, or weak garage doors and their attachment to the building. Section 3.10 discusses tornado design conditions.

In recent years, much attention has been focused on wind-borne debris, based on the results of many damage investigations. Little research has been done to quantify the magnitude or type of debris. The current wind-borne debris protection trigger is wind speed, and the requirement for wind-borne debris protection is all or nothing—meaning that, in accordance with ASCE 7-10, protection must be provided where design wind speeds are 130 mph and the building is within one mile of the coastal mean high-water line, or anywhere the design wind speed is 140 mph or greater. Conventional practice for wind-borne debris protection in residential construction usually is either impact-resistant shutters installed over glazed openings or impact-resistant glazing. The IRC still permits the use of wood structural panels (plywood or oriented strand board [OSB]) as opening protection for glazing in one- and two-story buildings to resist impacts from wind-borne debris. To use wood structural panels for opening protection, however, attachment hardware is required, with anchors permanently installed on the building. Impact-resistant glazing or protective devices must be tested using an approved test method, such as ASTM E1886 (ASTM, 2005) and ASTM E1996 (ASTM, 2009b).

Just what defines impact resistance and the level of impact risk during a hurricane continues to be the subject of much debate. Surveys of damage following major hurricanes have identified several factors that affect the level of debris impact risk, including

- wind climate (design wind speed);
- exposure (for example, suburban, wooded, height of surrounding buildings);
- development density (that is, distance between buildings);
- construction characteristics (for example, type of roofing, degree of wind resistance); and
- debris sources (for example, roofing, fencing, and gravel).

Current standards for selecting impact criteria for wind-borne debris protection do not explicitly consider all of those factors. Further, the primary debris source in typical residential developments is asphalt roof shingles, clay roof tiles, landscaping materials and driveway gravel, vinyl siding, and vegetation from trees and shrubs, some of which are not represented in existing impact test methods. Recent research has provided insight into performance expectations (Fernandez, Masters, and Gurley, 2010; Masters et al., 2010). These factors have a dramatic effect on the level of wind-borne debris risk. Table 3.11 presents an example of missile types used for current impact tests. Additional factors to consider include emergency egress or access in the event of fire when impact-resistant glazing or fixed shutter systems are specified, potential injury or misapplication during installation of temporary methods of window protection, and durability of protective devices and connection details (including installation quality) such that they themselves do not become a debris hazard over time.

TABLE 3.11 Missile Types for Wind-Borne Debris Impact Tests[1,2]

Description	Velocity	Energy
2 gram steel balls	130 fps	10 ft-lb
4.5 lb 2x4	40 fps	100 ft-lb
9.0 lb 2x4	50 fps	350 ft-lb

fps = feet per second; ft-lb = foot-pounds; lb = pounds

[1]Consult ASTM E1886 (ASTM, 2005) for guidance on testing apparatus and methodology.
[2]These missile types are not necessarily representative of the predominant types or sources of debris at any particular site. The steel balls are intended to represent small gravel that would be commonly used for roof ballast. The 2x4 missiles are intended to represent a direct, end-on blow from construction debris.

Homes that experience wind-borne debris damage may exhibit more catastrophic failures, such as a roof blowoff, but usually this occurs only when large elements of the building envelope fail, such as large windows or garage doors. Wind pressure can also cause failures in these large elements; therefore, in hurricane-prone regions, large windows and garage doors should be specified that meet both wind pressure and wind-borne debris impact requirements, and the attachment of those elements to structural framing should be carefully designed.
One additional element that requires consideration, and for which research is being conducted, is wind-driven rain. Most window manufacturers have products tested to some limitation on water infiltration, and under normal weather conditions those limitations usually are sufficient (usually up to 15 percent of the design wind pressure). Hurricane-force winds will drive rain horizontally and that water can penetrate between window units, under doors, and into soffits and other small places such that, even with attention to this issue, the water can cause significant damage. Both the designer and the builder must pay attention to the construction details at every building joint and every hole in the building envelope to ensure that water penetration during high winds is minimized (Salzano, Masters, and Katsaros, 2010).

3.6.3.2 Tips to Improve Performance

The following design and construction tips are simple considerations for reducing a building's vulnerability to hurricane wind damage:

- One-story buildings are less vulnerable than two- or three-story buildings to wind damage.
- On average, hip roofs have demonstrated better performance than gable-end roofs.
- Moderate roof slopes (that is, 5:12 to 6:12) tend to optimize the tradeoff between lateral loads and roof uplift loads (that is, they are aerodynamically efficient).
- Roof sheathing installation should be inspected for the proper type and spacing of fasteners, particularly at connections to gable-end framing.
- The installation of metal strapping or other tie-down hardware should be inspected as required to ensure the transfer of uplift loads.
- If composition roof shingles are used, the shingles should be tested in accordance with ASTM D7158 (ASTM, 2011). All roof coverings should be designed or tested and installed to resist the applicable wind loads.

- Glazed-opening protection should be considered in the most severe hurricane-prone areas and in those areas defined as requiring wind-borne debris protection.
- The roof deck may be sealed or a secondary water barrier may be installed on the roof to prevent water infiltration in the event the primary roof covering is blown off. A sealed roof deck can be created by installing minimum 4-inch-wide strips of self-adhering underlayment complying with ASTM D1970 over the roof sheathing joints (ASTM, 2009c). The IRC also contains enhanced underlayment specifications for high-wind regions that require the use of ASTM D226 Type II (30 pound) or equivalent underlayment with a rigorous fastening schedule (ASTM, 2009a).

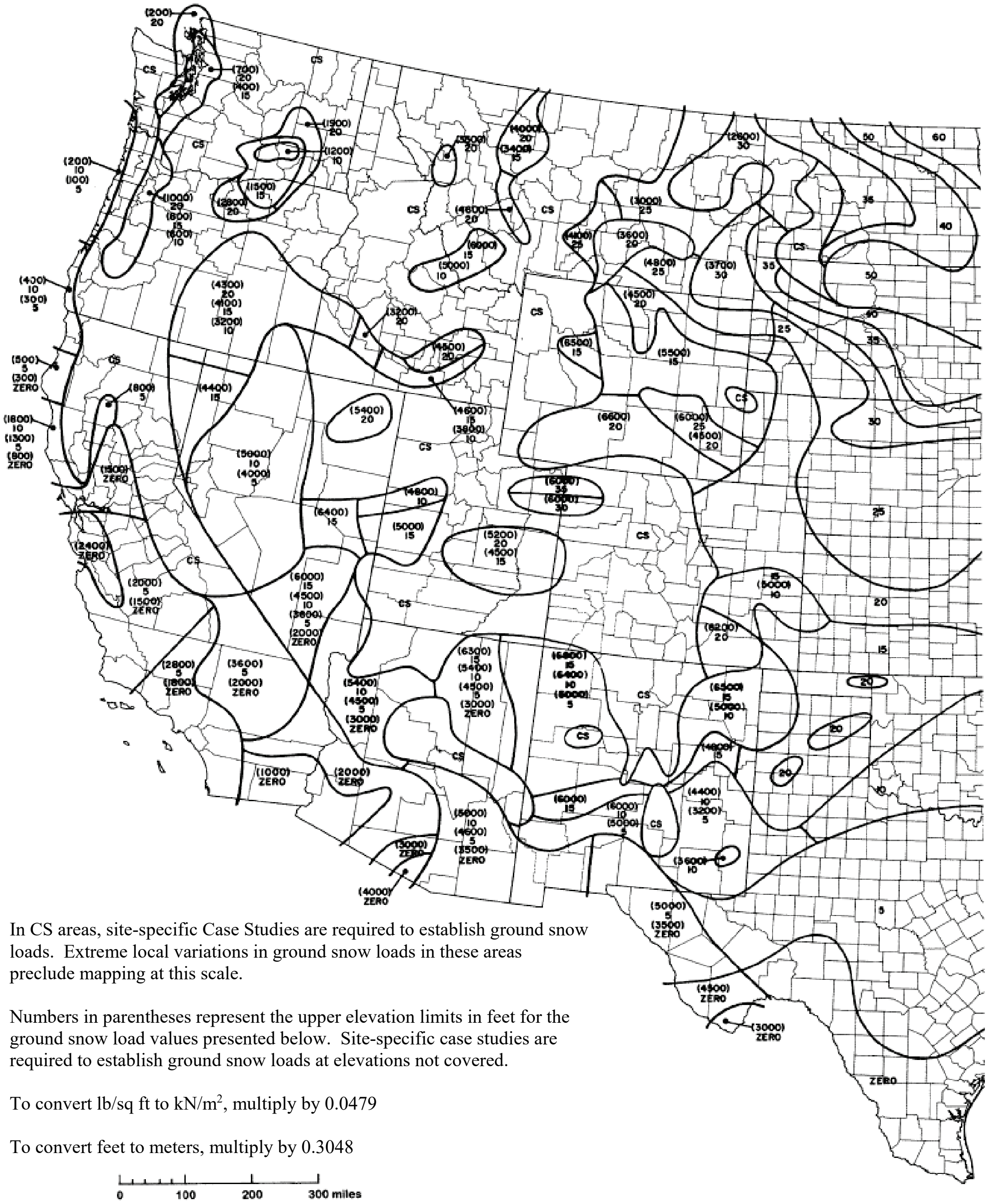
The HUD document *Safer, Stronger Homes* (HUD, 2011) includes further details regarding methods for improving the wind hazard resilience of new and existing residential structures.

3.7 Snow Loads

For design purposes, snow typically is treated as a simple uniform gravity load on the horizontal projected area of a roof. The uniformly distributed design snow load on residential roofs can be easily determined by using the unadjusted ground snow load. This simple approach also represents standard practice in some regions of the United States; however, it does not account for a reduction in roof snow load that may be associated with steep roof slopes with slippery surfaces (refer to ASCE 7-10). Drift loads on sloped gable or hip roofs must consider roof slope, warm and cold roof slope factors, and ridge-to-eave distances. ASCE 7-10 has design parameters for each of these snow and roof conditions. Drifting snow has caused numerous roof failures in the past 5 to 10 years. The design guidance in ASCE 7 addresses some of the issues important to consider for drifting snow; for building design, drifting snow must be considered at any building intersection where a roof adjoins a wall or other vertical surface where snow can accumulate. For buildings, snow drifting can occur where a building extension such as a garage or first floor addition adjoins an existing two-story wall. The problem of loading is complicated because snow loads vary with moisture content as well as depth, and depths vary with roof slope, wind speed, and vertical height where drifting can occur.

Design ground snow loads may be obtained from the map in figure 3.3 (for a larger ground snow load map with greater detail, refer to ASCE 7-10); however, snow loads usually are defined by the local building department. Typical ground snow loads range from 0 psf in the southern United States to 50 psf in the northern United States. In mountainous areas, the ground snow load can surpass 100 psf, so local snow data must be carefully considered. The ASCE 7-10 map includes varying ground snow loads with ground elevation above sea level. In areas where the ground snow load is less than 15 psf, the minimum roof live load (refer to section 3.4) usually is the controlling gravity load in roof design.

FIGURE 3.3a *Ground Snow Loads (ASCE 7-10)*



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 kN/m^2, multiply by 0.0479

To convert feet to meters, multiply by 0.3048

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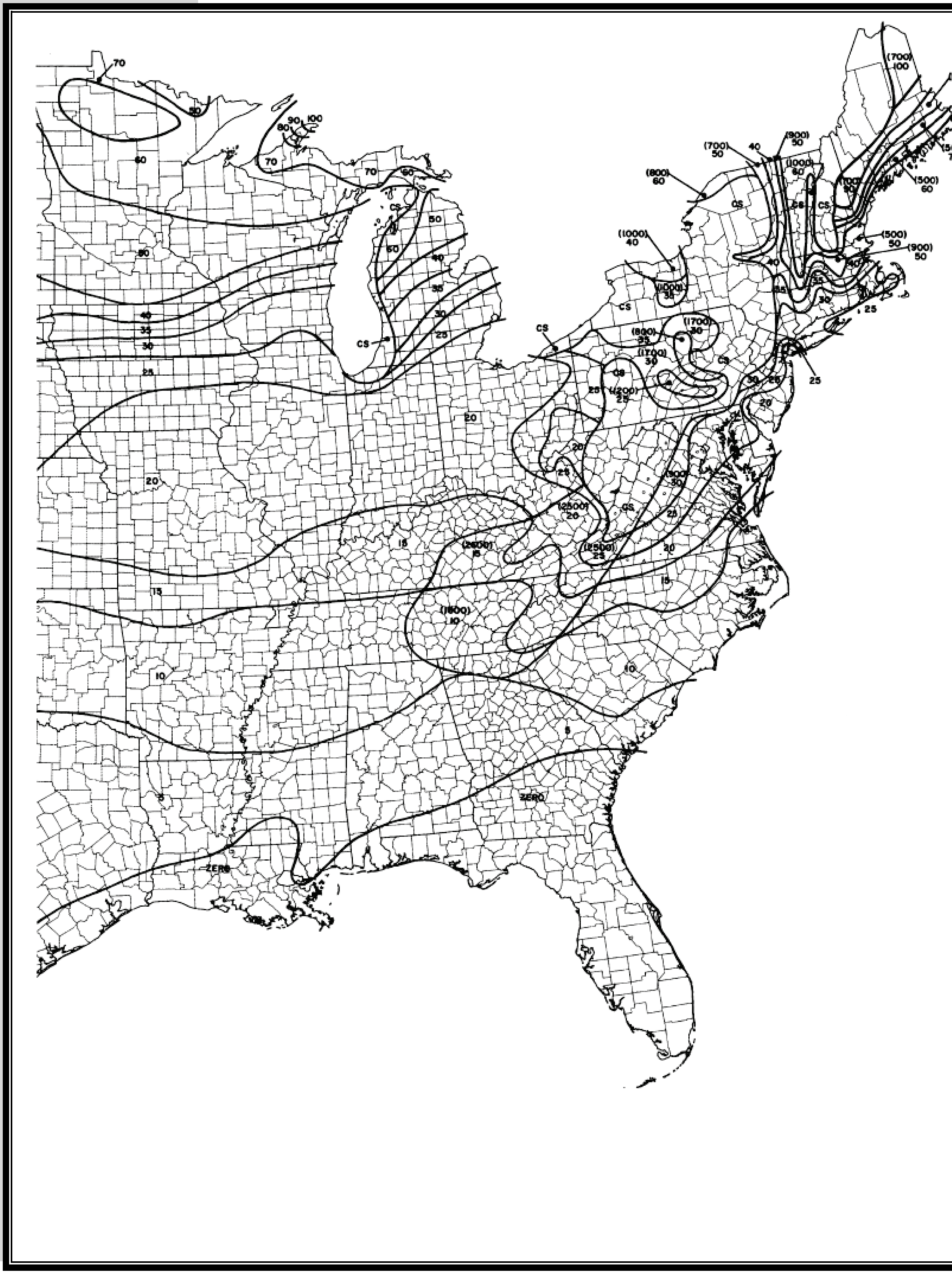
FIGURE 3.3b Ground Snow Loads (ASCE 7-10)



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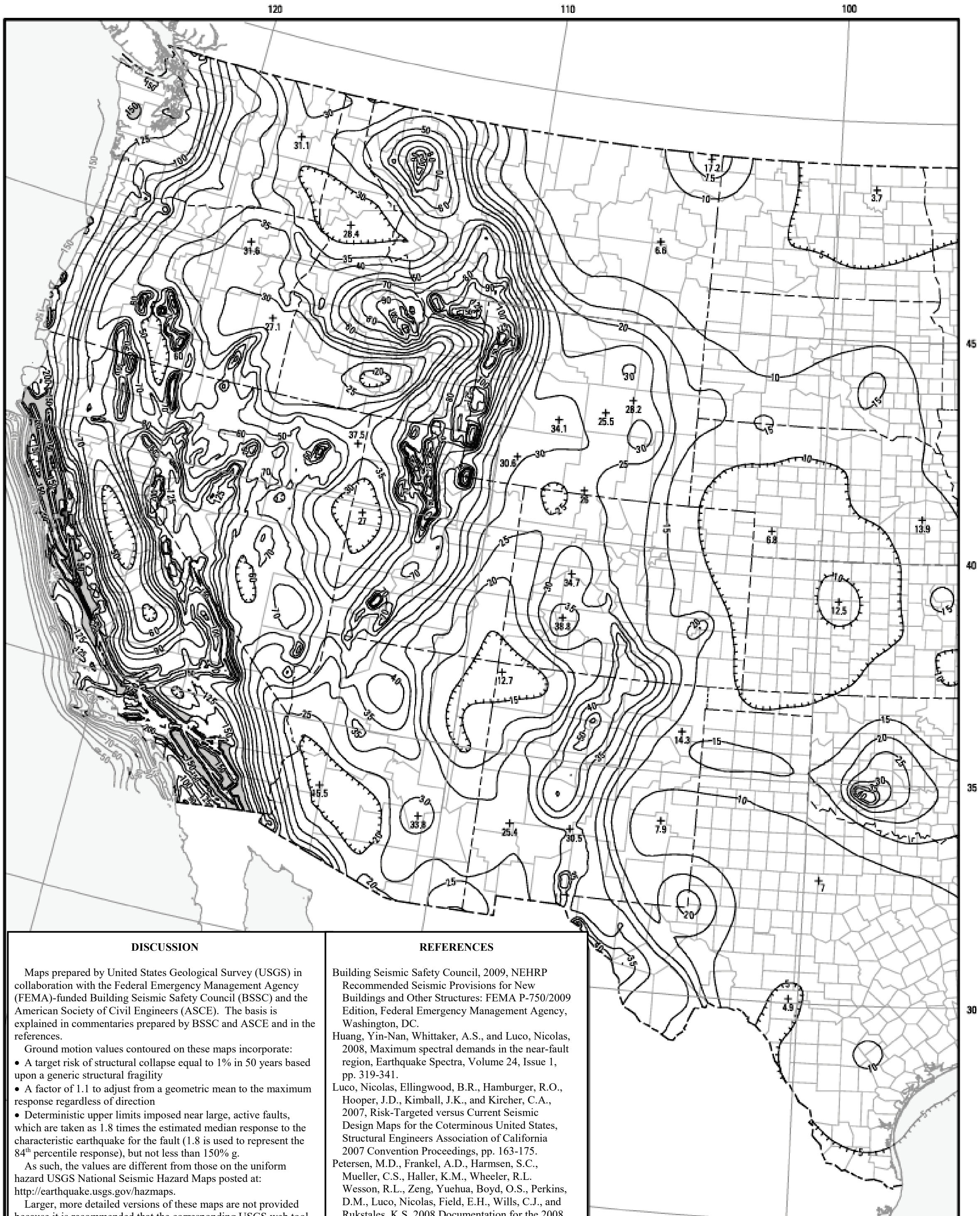
3.8 Earthquake Loads

3.8.1 General

This section provides a simplified earthquake load analysis procedure appropriate for use in residential light-frame construction of not more than three stories above grade. As described in chapter 2, the lateral forces associated with seismic ground motion are based on fundamental Newtonian mechanics (F = ma), expressed in terms of an equivalent static load. The method provided in this section is a simplification of the most current seismic design provisions *NEHRP (National Earthquake Hazards Reduction Program) Recommended Seismic Provisions for New Buildings and Other Structures* (FEMA, 2009a). The method herein also is similar to a simplified approach found in more recent building codes (ICC, 2012).

In general, wood-framed homes have performed well from a life safety standpoint in major seismic events, probably because of, among other factors, their lightweight and resilient construction, the strength provided by nonstructural systems such as interior walls, and their load distribution capabilities. Wood-framed homes have not performed as well from a damage-reduction standpoint, in part because of brittle finishes such as gypsum board and masonry exteriors, as well as insufficient anchorage of wall framing to foundations, lack of sheathing on cripple walls, and slope failures on hillside sites (HUD, 1994). Garages with wide doors or houses with many large windows on the ground floor can fail as a result of so-called “soft story” or “weak story” behavior because the garage or ground floor walls are much less stiff than the roof or stories above.

FIGURE 3.4a Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration



DISCUSSION

Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC) and the American Society of Civil Engineers (ASCE). The basis is explained in commentaries prepared by BSSC and ASCE and in the references.

Ground motion values contoured on these maps incorporate:

- A target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
- A factor of 1.1 to adjust from a geometric mean to the maximum response regardless of direction
- Deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the fault (1.8 is used to represent the 84th percentile response), but not less than 150% g.

As such, the values are different from those on the uniform hazard USGS National Seismic Hazard Maps posted at: http://earthquake.usgs.gov/hazmaps.

Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps or http://content.seinstitute.org) be used to determine the mapped value for a specified location.

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Petersen, M.D., Frankel, A.D., Harmsen, S.C., Mueller, C.S., Haller, K.M., Wheeler, R.L. Wesson, R.L., Zeng, Yuehua, Boyd, O.S., Perkins, D.M., Luco, Nicolas, Field, E.H., Wills, C.J., and Rukstales, K.S, 2008 Documentation for the 2008 Update of the United States Seismic Hazard Maps: U.S. Geological Survey Open-File Report 2008-1128, 61 p.

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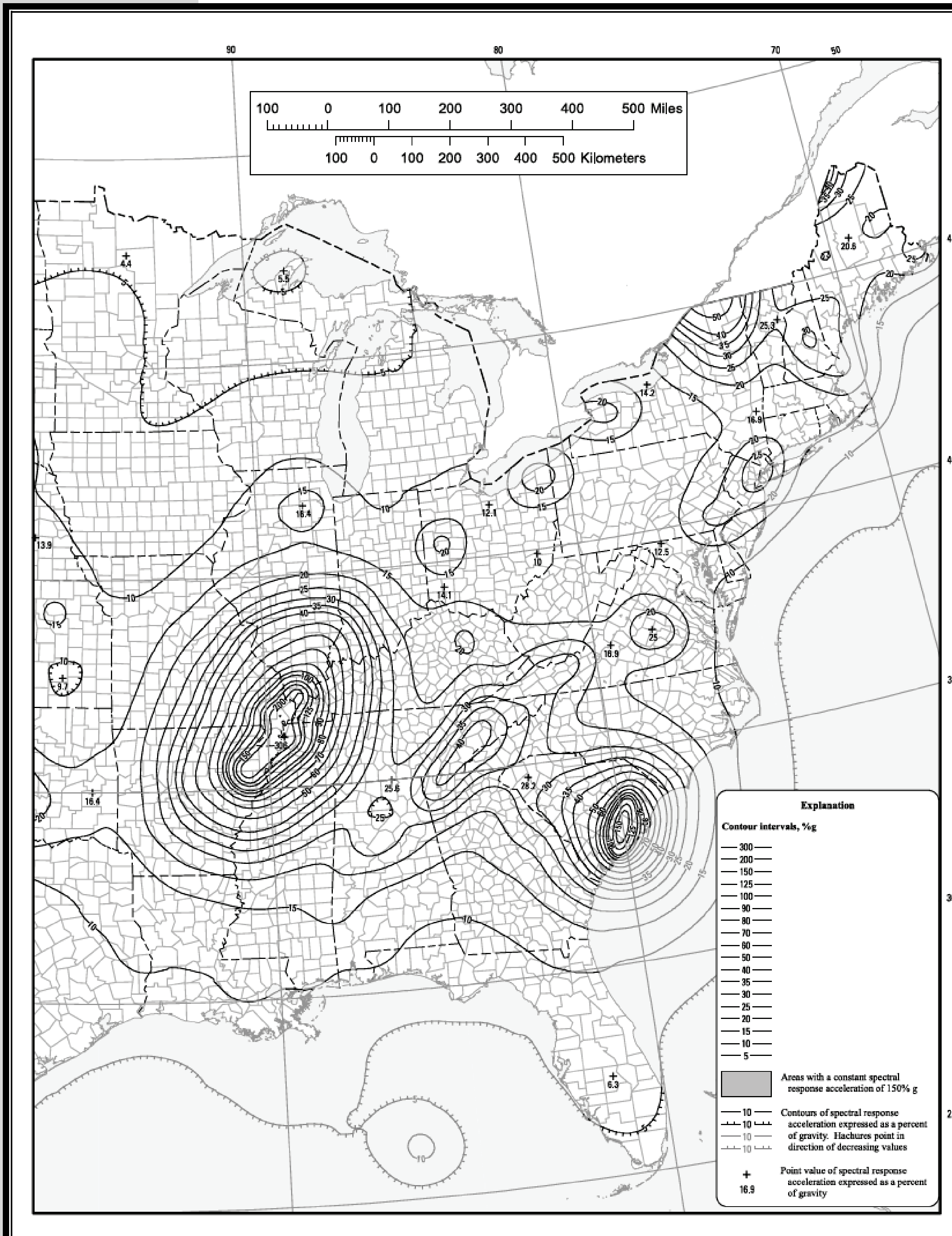
FIGURE 3.4b ***Mapped Risk-Targeted Maximum Considered Earthquake (MCE$_R$) Spectral Response Acceleration***



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3.8.2 Determination of Earthquake Loads on Houses

The total lateral force at the base of a building is called seismic base shear. The lateral force experienced at a particular story level is called the story shear. The story shear is greatest in the ground story and least in the top story. Seismic base shear and story shear (V) are determined in accordance with the following equation:

Equation 3.8-1

$$V = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} W \text{ ,}$$

where

S_{DS}	=	the design spectral response acceleration in the short-period range determined by equation 3.8-2 (g)
R	=	THE RESPONSE MODIFICATION FACTOR (DIMENSIONLESS)
W	=	the effective seismic weight of the building or supported by the story under consideration (lb); 20 percent of the roof snow load is also included where the ground snow load exceeds 30 psf
I_e	=	importance factor, which is 1.0 for residential buildings.

When determining story shear for a given story, the designer attributes to that story one-half of the dead load of the walls on the story under consideration and the dead load supported by the story (dead loads used in determining seismic story shear or base shear are found in section 3.3). For housing, the interior partition wall dead load is reasonably accounted for by the use of a 6 psf load distributed uniformly over the floor area. When applicable, the snow load may be determined in accordance with section 3.7. The inclusion of any snow load, however, is based on the assumption that the snow is always frozen solid and adhered to the building such that it is part of the building mass during the entire seismic event.

The design spectral response acceleration for short-period ground motion S_{DS} typically is used because light-frame buildings such as houses are believed to have a short period of vibration in response to seismic ground motion (that is, high natural frequency). For example, the building tested as part of the NEESWood project in 2006 (Filiatrault et al., 2010) had an elastic period of 0.21 seconds, consistent with the 0.2-second period used to establish the short-period ground motions.

Values of S_{MS} are from figure 3.4. For a larger map with greater detail, refer to ASCE 7-10 or find the response accelerations using the U.S. Geological Survey (USGS) seismic design maps, based on either latitude and longitude or zip codes: **http://earthquake.usgs.gov/hazards/designmaps/usdesign.php**. The value of S_{DS} should be determined in consideration of the mapped short-period spectral response acceleration S_{MS} and the required soil site amplification factor F_a as follows:

Equation 3.8-2

$$S_{DS} = 2/3(S_{MS})(F_a)$$

The value of S_{MS} ranges from practically zero in low-risk areas to 3g in the highest risk regions of the United States. A typical value in high seismic areas is 1.5g. In general, wind loads control the design of the LFRS of light-frame houses when S_{MS} is low.

Table 3.12 provides the values of F_a associated with a standard "firm" soil condition used for the design of residential buildings. F_a decreases with increasing ground motion because the soil begins to dampen the ground motion as shaking intensifies. The soil can therefore have a moderating effect on the seismic shear loads experienced by buildings in high-seismic-risk regions. Dampening also occurs between a building foundation and the soil and thus has a moderating effect. The soil-structure interaction effects on residential buildings have been the topic of little study; therefore, precise design procedures have yet to be developed. If a site is located on fill soils or "soft" ground, a different value of F_a should be considered (see ASCE, 2010, for the full table). Nonetheless, as noted in the Anchorage Earthquake of 1964 and again 30 years later in the Northridge Earthquake, soft soils do not necessarily affect the performance of the aboveground house structure as much as they affect the site and foundations (for example, by settlement, fissuring, or liquefaction).

TABLE 3.12 Site Soil Amplification Factor Relative to Acceleration (short period, Site Class D)

Short-Period Spectral Response Acceleration, S_{MS}	≤ 0.25g	0.5g	0.75g	1.0g	≥ 1.25g
Site Soil Amplification Factor, F_a,	1.6	1.4	1.2	1.1	1.0

The seismic response factor R has a long history in seismic design but with little in the way of scientific underpinnings until recently (FEMA, 2009b). In fact, the R factor can be traced back to expert opinion in the development of seismic design codes during the 1950s (ATC, 1995). In recognition that buildings can effectively dissipate energy from seismic ground motions through ductile damage, the R factor was conceived to adjust the shear forces from that which would be experienced if a building could exhibit perfectly elastic behavior without some form of ductile energy dissipation (Chopra, 2012).

Those structural building systems that are able to withstand greater ductile damage and deformation without substantial loss of strength are assigned a higher value for R. The R factor also incorporates differences in dampening that are believed to occur for various structural systems. Table 3.13 provides some values for R that are relevant to residential construction. The *Quantification of Building Seismic Performance Factors FEMA P-695* (FEMA, 2009b) methodology allows one to develop an R factor for a new LFRS based on the margin against collapse.

The overstrength factor Ω_0 addresses the idea that a shear resisting system's ultimate capacity usually is significantly higher than required by the design load as a result of intended safety margins. Designers incorporate overstrength factors in an attempt to address the principle of balanced design, striving to ensure that components such as connections have sufficient capacity to allow the LFRS to act in its intended ductile manner. These factors are applied at the load combination stage of force development.

The deflection amplification factor C_d is applied to adjust the deflection of story drift, which is determined by use of the seismic shear load as adjusted downward by the R factor. The use of this amplification factor will likely produce a conservative result of expected drift; drift calculations rarely are required in lowrise light-frame buildings because code-required drift limits have not been established for these structure types.

TABLE 3.13 Seismic Design Factors for Residential Construction

Structural System	Response Modification Coefficient, R[1]	Overstrength factor, Ω_o	Deflection Amplification Factor, C_d
Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance or steel sheets	6.5	3	4
Light-frame shear walls with shear panels of all other materials	2.0	2.5	2
Special reinforced concrete shear walls[2]	5.0	2.5	5
Special reinforced masonry shear walls[2]	5.0	2.5	3.5
Ordinary plain concrete shear walls	1.5	2.5	1.5
Ordinary plain masonry shear walls	1.5	2.5	1.25

Notes:

[1]The R factors may vary for a given structural system type depending on wall configuration, material selection, and connection detailing.

[2]The wall is reinforced in accordance with concrete design requirements in ACI-318 or ACI-530. Nominally reinforced concrete or masonry that has conventional amounts of vertical reinforcement, such as one #5 rebar at openings and at 4 feet on center, may use the value for reinforced walls, provided the construction is no more than two stories above grade.

Design example 3.3 in section 3.12 demonstrates the calculation of design seismic shear load based on the simplified procedures (the reader is referred to chapter 6 for additional information on seismic loads and analysis).

3.8.3 Seismic Shear Force Distribution

As described in the previous section, the *vertical distribution* of seismic forces to separate stories on a light-frame building is assumed to be in accordance with the mass supported by each story. The lateral seismic force, F_x, induced at any level, is determined as

Equation 3.8-3

$$F_x = C_{vx}V, \text{ and}$$

$$C_{vx} = \frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

Where:

C_{vx}	=	vertical distribution factor
V	=	total base shear
w_i	=	portion of the total effective seismic weight of the structure at level *i*
w_x	=	portion of the total effective seismic weight of the structure at level *x*
h_i	=	height from the base to level *i*
h_x	=	height from the base to level *x*

The *horizontal distribution* of seismic forces to various shear walls on a given story also varies in current practice for light-frame buildings. In chapter 6, several existing approaches to the design of the LFRS of light-frame houses address the issue of horizontal force distribution, with varying degrees of sophistication. Until methods of vertical and horizontal seismic force distribution are better understood for application to light-frame buildings, the importance of designer judgment cannot be overemphasized.

3.8.4 Other Seismic Design Considerations

Perhaps the single most important principle in seismic design is ensuring that the structural components and systems are adequately tied together to perform as a structural unit. Underlying this principle are a host of analytic challenges and uncertainties in actually defining what “adequately tied together” means in a repeatable, accurate, and theoretically sound manner.

Irregularities in the building shape are a key design consideration. Irregularites can occur in plan and in elevation. Plan irregularities can create torsional imbalances, thus requiring designs for moment distribution. Vertical irregularities are often stiffness irregularities, such as “soft stories,” “weak stories,” or “heavy stories.” Sometimes the vertical discontinuities can create unusual distribution of shear between LFRS.

When diaphragms are not flexible, the design should include the inherent torsional moment, M_t, resulting from the location of the structural masses plus the accidental torsional moment, M_{ta}, caused by assumed displacement of the center of mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied forces. Overturning must be anticipated in the design, and the structure must be designed to resist such forces. Story drift is computed as the largest difference of the deflections aligned vertically at the top and bottom of the story under consideration. The design story drift at level x is computed as

$$\delta_x = \left(\frac{C_d \delta_{xe}}{I_e} \right)$$

where C_d is the deflection amplification factor from table 3.12, δ_{xe} is the deflection at the location of interest determined by elastic analysis, and I_e is the importance factor, which is 1.0 for residential buildings.

For one- and two-story dwellings, the diaphragms are assumed to be flexible. Rigid diaphragms usually are those constructed of concrete or concrete-filled metal deck. ASCE 7-10 has a set of conditions that are used to determine whether a diaphragm is flexible or rigid.

A key issue related to building damage involves *deformation compatibility* of materials and detailing in a constructed system. This issue may be handled through specification of materials that have similar deformation capabilities or by system detailing that improves compatibility. For example, a relatively flexible hold-down device installed near a rigid sill anchor causes greater stress concentration on the more rigid element, as evidenced by the splitting of wood sill plates in the Northridge Earthquake. The solution can involve increasing the rigidity of the hold-down device (which can lessen the ductility of the system, increase stiffness, and effectively increase seismic load) or redesigning the sill plate connection to accommodate the hold-down deformation and improve load distribution. Researchers in a FEMA-funded CUREE-CalTech project developed a solution for the sill plate connection: a 3-inch-square washer for use on the sill plate anchor bolt. As a nonstructural example of deformation compatibility, gypsum board interior finishes crack in a major seismic event well before the structural capability of the wall's structural sheathing is exhausted. Conversely, wood exterior siding and similar resilient finishes tend to deform compatibly with the wall and limit unacceptable visual damage (HUD, 1994).

3.9 Flood Loads

A significant level of construction occurs in the nation's floodplains, so the design professional should be acquainted with the regulations and the design constraints required for building in these areas. The basic design premise for a flood condition is either to elevate the structure above the expected flood level or to build outside the regulatory floodplain. The full explanation of floodplain regulations is beyond the scope of this document; however, the following issues are important for all designers of buildings in floodplains:

- Floodplain regulations are local, so local zoning and/or building codes govern any construction in a floodplain.
- Local ordinances define the regulatory flood elevation, but it usually is the Base Flood Elevation (BFE), defined as the 1-percent annual exceedance probability flood. Frequently the community requires a certain number of feet of freeboard above the BFE to provide a margin of safety.

- The BFE usually is shown on a Flood Insurance Rate Map (FIRM), and this map is available either locally or digitally at FEMA's Map Service Center.
- Minimum construction standards exist for buildings located in floodplains, and these standards govern elevation of the lowest living floor, the type of foundation that can be constructed, and the materials that can be used below the BFE.

Equations for flood loads are provided in ASCE 7, chapter 5, and for coastal flood conditions, flood formulas are available in FEMA's *Coastal Construction Manual* (FEMA, 2011a) or in the United States Army Corps of Engineers *Coastal Engineering Manual* (USACE, 2009). Details specific to residential construction can be found in FEMA's *Home Builder's Guide to Coastal Construction* (FEMA, 2011b).

ASCE 24, the *Flood Resistant Design and Construction* standard, also contains significant flood design information. This standard does not include any information about flood loads but does suggest flood elevations for various building occupancies and provides design guidance for building issues from foundations to utilities.

3.10 Tornadoes

A tornado is a narrow, violently rotating column of air that extends from the base of a thunderstorm to the ground. They are the most violent of all atmospheric storms. Tornadoes occur in many parts of the world, but most of them by far occur in the United States, which experiences on average about 1,200 tornadoes per year (NCDC, 2013). Still, a direct tornado strike on a building is a relatively rare event, and the annual probability is lower than for other natural hazard events (that is 1.87E-4 for a tornado strike of any intensity [Ramsdell et al., 2007] vs. 1.43E-3 for hurricane design wind speeds [ASCE, 2010]). Despite their small size, tornadoes can travel great distances and thus cause destruction to several communities within their path.

Building codes do not provide design guides for tornado loads for two reasons: (1) the rarity of the event and (2) the extreme magnitude of the tornado loads. The media report numerous opinions about whether building codes should cover that type of low probability, high consequence event. Many people believe that it would be economically unfeasible to design houses to resist the expected 200 mph and higher wind speeds produced in tornadoes. Substantial evidence also exists, however, that much damage could be reduced even when communities are struck by extremely violent (EF-4 and EF-5) tornadoes. The Enhanced Fujita (EF) scale is used to classify tornadoes by wind speed, using damage as the indicator of that speed. Examination of damage suggests that such extensive devastation is the result of inadequate structural systems in homes that were not designed for—and are incapable of resisting—any significant wind load. The damage report for a 1970 Lubbock tornado concluded that although the maximum estimated wind speed was 200 mph, the majority of building damage was caused by winds that were only in the 75-to-125 mph range (Mehta et al., 1971).

The interest in developing tornado-resilient design of housing and other structures has gained interest recently following several years (for example, 2011 and

2013) in which violent tornadoes have hit large, densely populated areas. The economic losses attributed to tornadoes since 2000 amount to 15 percent of the economic losses from hurricanes over that same period (NWS, 2013). Despite the low probability of tornado occurrences, the consequences are fairly high when a community is impacted by one of these natural hazard events.

The unique wind loads produced by an extreme tornado (that is, an EF-5 on the Enhanced Fujita scale) well exceed typical design wind loads, particularly in interior portions of the country, where tornadoes are most common but where design wind speeds typically are 115 mph. Most tornadoes, though, are not the most devastating kind; more than 90 percent of all tornadoes are classified as an EF-2 or lower on the EF Scale. Further, detailed analysis of the damage paths of recent violent tornadoes have shown that nearly 90 percent of the damage paths experience wind speeds at or less than the intensity of an EF-2 tornado (Prevatt et al., 2012). Applying the concepts used for hurricane design to buildings located in tornado-prone areas can reduce damage from the lowest wind speed tornadoes.

Tornado loads differ from typical straightline wind events such as hurricanes in that the loads are a superposition of the aerodynamic effects of the wind passing over and around the building and the significant pressure drop within the vortex of the tornado. In combination, these two effects can produce loads on the building in a tornado that are nearly three times higher than those for a straightline wind event with equivalent wind speed (Haan et al, 2010). Many factors affect the magnitude of tornado loads, however, including the tornado size, translation speed, approach angle, and air leakage through the impacted structure. The contributions of each effect have only recently been quantified. As a result, little current information exists for designing structures to survive tornado events. The next version of the commentary on the ASCE 7 standard will likely have some information that will help designers incorporate some level of tornado resistance in their designs. For the most severe events, such as those created by EF-4 or EF-5 tornadoes, a safe room or shelter built to FEMA guidance or ICC standards affords the best life safety protection. ICC 500 (ICC, 2008) is a *Standard for the Design and Construction of Storm Shelters*; tornado safe room guidance is available in FEMA P-320 (FEMA, 2008b) and community shelter guidance is available in FEMA P-361 (FEMA, 2008a). These safe room and shelter guidance documents and standards have substantial design information about what wind speeds should be used for design and how to modify the wind pressure equation in ASCE 7 to accommodate the differences in the tornado wind structure compared to the hurricane or thunderstorm wind structure.

3.11 Other Load Conditions

In addition to the loads covered in sections 3.3 through 3.10 that are typically considered in the design of a home, other “forces of nature” may create loads on buildings. Some examples include

- frost heave;
- expansive soils; and
- temperature effects.

In certain cases, forces from these phenomena can drastically exceed reasonable design loads for homes. For example, frost heave forces can easily exceed 10,000 psf (Linell and Lobacz, 1980). Similarly, the force of expanding clay soil can be impressive. The self-straining stresses induced by temperature-related expansion or contraction of a member or system that is restrained against movement can be very large, although those stresses are not typically a concern in wood-framed housing.

Sound design detailing is common practice to reduce or eliminate the load increases mentioned. For example, frost heave can be avoided by placing footings below a "safe" frost depth, building on nonfrost-susceptible materials, or using other frost protection methods (see chapter 4). Expansive soil loads can be avoided by isolating building foundations from expansive soil, supporting foundations on a system of deep pilings, and designing foundations that provide for differential ground movements. Temperature effects can be eliminated by providing construction joints that allow for expansion and contraction. Although such temperature effects on wood materials are practically negligible, some finishes, such as ceramic tile, can experience cracking when inadvertently restrained against small movements resulting from variations in temperature.

As noted at the beginning of this chapter, this guide does not address loads from ice, rain, and other exceptional sources. The reader is referred to ASCE 7 and other resources for information regarding special load conditions (ASCE, 2010).

3.12 Design Examples

EXAMPLE 3.1	*Design Gravity Load Calculations and Use of ASD Load Combinations*

Given

Three-story conventional wood-framed home
28' x 44' plan, clear-span roof, floors supported at mid-span
Roof dead load = 15 psf (table 3.2)
Wall dead load = 8 psf (table 3.2)
Floor dead load = 10 psf (table 3.2)
Roof snow load = 16 psf (section 3.7)
Attic live load = 10 psf (table 3.4)
Second- and third-floor live load = 30 psf (table 3.4)
First-floor live load = 40 psf (table 3.4)

Find

Gravity load on first-story exterior bearing wall
Gravity load on a column supporting loads from two floors

Solution

1. Gravity load on first-story exterior bearing wall

- Determine loads on wall

Dead load = roof DL + 2 wall DL + 2 floor DL
= 1/2 (28 ft)(15 psf) + 2(8 ft)(8 psf) + 2(7 ft)(10 psf)
= 478 plf

Floor span assumes a center support wall, thus load on exterior wall is 28'/4

Roof snow = 1/2(28 ft)(16 psf) = 224 plf
Live load (two floors) = (30 psf + 30 psf)(7 ft) = 420 plf
Attic live load = (10 psf)(14 ft - 5 ft*) = 90 plf
*edges of roof span not accessible to roof storage because of low clearance

- Apply applicable ASD load combinations (table 3.1)

D + 0.75L + 0.75S

Wall axial gravity load = 478 plf + 0.75*420 plf + 0.75*224 plf
= 961 plf*
*equals 1,029 plf if full attic live load allowance is included with L

The same load applies to the design of headers as well as to the wall studs. Of course, combined lateral (bending) and axial loads on the wall studs also must be checked (that is, D+W); refer to table 3.1 and example 3.2. For non-load-bearing exterior walls (that is, gable-end curtain walls), contributions from floor and roof live loads may be negligible (or significantly reduced), and the D+W load combination likely governs the design.

2. Gravity load on a column supporting a center floor girder carrying loads from two floors (first and second stories)

- Assume a column spacing of 16 ft
- Determine loads on column

(a) Dead load = Second floor + first floor + bearing wall supporting second floor

= (14 ft)(16 ft)(10 psf) + (14 ft)(16 ft)(10 psf) + (8 ft)(16 ft)(7 psf)

= 5,376 lbs

(b) Live load area reduction (equation 3.4-1)

- supported floor area = 2(14 ft)(16 ft) = 448 ft^2 per floor
- reduction = $\left[0.25 + \dfrac{15}{\sqrt{4*448}}\right] = 0.6 > 0.5$

OK

- first-floor live load = 0.6 (40 psf) = 24 psf
- second-floor live load = 0.6 (30 psf) = 18 psf

(c) Live load = (14 ft)(16 ft)[24 psf + 18 psf]

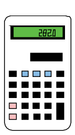
= 9,408 lbs

- Apply ASD load combinations (table 3.1)

The controlling load combination is D+L because the column supports no attic or roof loads. The total axial gravity design load on the column is 14,748 lbs (5,376 lbs + 9,408 lbs).

Note: If LRFD material design specifications are used, the various loads would be factored in accordance with table 3.1; all other considerations and calculations remain unchanged.

EXAMPLE 3.2 *Design Wind Load Calculations and Use of ASD Load Combinations*



Given

Site wind speed: 120 mph, gust
Site wind exposure: suburban
Two-story home, 7:12 roof pitch, 28' x 44' plan (rectangular), gable roof, 12-inch overhang

Find

Lateral (shear) load on lower-story end wall
Net roof uplift at connections to the side wall
Roof sheathing pull-off (suction) pressure
Wind load on a roof truss
Wind load on a rafter

Lateral (out-of-plane) wind load on a wall stud

Solution

1. Lateral (shear) load on lower story end wall

Step 1: LRFD velocity pressure = 22 psf (table 3. 8)
Step 2: Adjusted velocity pressure (none required) = 22 psf
Step 3: Lateral roof coefficient = 0.8 (interpolated from table 3.9)
Lateral wall coefficient = 1.1 (table 3.9)
Step 4: Skip
Step 5: Determine design wind pressures
Roof projected area pressure = (22 psf)(0.8) = 17.6 psf (LRFD)
Wall projected area pressure = (22 psf)(1.1) = 24.2 psf (LRFD)

Now determine vertical projected areas (VPA) for lower-story end-wall tributary loading (assuming no contribution from interior walls in resisting lateral loads).

Roof VPA = [1/2 (building width)(roof pitch)] x [1/2 (building length)]
= [1/2 (28 ft)(7/12)] x [1/2 (44 ft)]
= [8.2 ft] x [22 ft]
= 180 ft^2

Wall VPA = [(second-story wall height) + (thickness of floor) + 1/2 (first-story wall height)] x [1/2 (building length)]
= [8 ft + 1 ft + 4 ft] x [1/2 (44 ft)]
= [13 ft] x [22 ft]
= 286 ft^2

Now determine shear load on the first-story end wall.

Shear = (roof VPA)(roof projected area pressure) + (wall VPA)(wall projected area pressure)
= (180 ft^2)(17.6 psf) + (286 ft^2)(24.2 psf)
= 10,089 lbs (LRFD) or 10,089*0.6 = 6,053 lbs (ASD)

The first-story end wall must be designed to transfer a shear load of 6,053 lbs. If side-wall loads were determined instead, the vertical projected area would include only the gable-end wall area and the triangular wall area formed by the roof. Use of a hip roof would reduce the shear load for the side and end walls.

2. Roof uplift at connection to the side wall (parallel-to-ridge)

Step 1: Velocity pressure = 22 psf (as before) (LRFD)
Step 2: Adjusted velocity pressure = 22 psf (as before)
Step 3: Skip
Step 4: Roof uplift pressure coefficient = -1.2 (table 3.10)
Roof overhang pressure coefficient = 0.7 (table 3.10)
Step 5: Determine design wind pressure
Roof horizontal projected area (HPA) pressure = -1.2 (22 psf)
= -24.2 psf
Roof overhang pressure = 0.7 (22 psf) = 15.4 psf (upward)

Now determine gross uplift at roof-wall reaction.

Gross uplift = 1/2 (roof span)(roof HPA pressure) + (overhang)(overhang pressure coefficient)
= 1/2 (30 ft)(-24.2 psf) + (1 ft)(-15.4 psf)
= -385 plf (upward)

Roof dead load reaction = 1/2 (roof span)(uniform dead load)
= 1/2 (30 ft)(15 psf*)
*table 3.2
= 225 plf (downward)

Now determine net design uplift load at roof-wall connection.

Net design uplift load = $0.6D + 0.6W_u$ (table 3.1)
= 0.6 (225 plf) + 0.6(-385 plf)
= -96 plf (net uplift)

The roof-wall connection must be capable of resisting a design uplift load of 96 plf. Generally, a toenail connection will meet the design requirement, depending on the nail type, nail size, number of nails, and density of wall framing lumber (see chapter 7). At high design wind speeds or in more open wind exposure conditions, roof tie-down straps, brackets, or other connectors should be considered.

3. Roof sheathing pull-off (suction) pressure

Step 1:	Velocity pressure	= 22 psf (as before)
Step 2:	Adjusted velocity pressure	= 22 psf (as before)
Step 3:	Skip	
Step 4:	Roof sheathing pressure coefficient (suction)	= -2.8 (table 3.10)
Step 5:	Roof sheathing pressure (suction)	= (22 psf)(-2.8)
		= -61.6 psf

The fastener load depends on the spacing of roof framing and spacing of the fastener. Fasteners in the interior of the roof sheathing panel usually have the largest tributary area and therefore are critical. Assuming 24-inch-on-center roof framing, the fastener withdrawal load for a 12-inch-on-center fastener spacing is as follows:

Fastener withdrawal load = (fastener spacing)(framing spacing) (roof sheathing pressure)
= (1 ft)(2 ft)(-61.6 psf)
= -123.2 lbs (LRFD) or 0.6*123.2
= 73.9 lbs (ASD)

At high wind conditions, a closer fastener spacing or higher capacity fastener (that is, deformed shank nail) may be required; refer to chapter 7.

4. Load on a roof truss

Step 1:	Velocity pressure	= 22 psf (as before)
Step 2:	Adjusted velocity pressure	= 22 psf (as before)
Step 3:	Skip	
Step 4:	Roof truss pressure coefficient	= -0.9, +0.4 (table 3.10)
Step 5:	Determine design wind pressures	

(a) Uplift = -0.9 (22 psf) = -19.8 psf

(b) Inward = 0.4 (22 psf) = 8.8 psf

Because the inward wind pressure is less than the minimum roof live load (that is, 15 psf, table 3.4), the following load combinations would govern the roof truss design, and and the D+W load combination could be dismissed (refer to table 3.1):

D + (L_r or S)
0.6D + 0.6W_u*
*The net uplift load for truss design is relatively small in this case (approximately 4.9 psf).

5. Load on a rafter

Step 1:	Velocity pressure	= 22 psf (as before)
Step 2:	Adjusted velocity pressure	= 22 psf (as before)
Step 3:	Skip	
Step 4:	Rafter pressure coefficient	= -1.2, +0.7 (table 3.10)
Step 5:	Determine design wind pressures	

(a) Uplift = (-1.2)(22 psf) = -26.4 psf
(b) Inward = (0.7)(22 psf) = 15.4 psf

Rafters in cathedral ceilings are sloped, simply supported beams, whereas rafters that are framed with cross-ties (that is, ceiling joists) constitute a component (that is, top chord) of a site-built truss system. Assuming the former in this case, the rafter should be designed as a sloped beam by using the span measured along the slope. By inspection, the minimum roof live load (D+L_r) governs the design of the rafter in comparison to the wind load combinations (see table 3.1). The load combination 0.6 D+0.6W_u can be dismissed in this case for rafter sizing but must be considered when investigating wind uplift for the rafter-to-wall and rafter-to-ridge beam connections.

6. Lateral (out-of-plane) wind load on a wall stud

Step 1: Velocity pressure	= 22 psf (as before)
Step 2: Adjusted velocity pressure	= 22 psf (as before)
Step 3: Skip	
Step 4: Wall stud pressure coefficient	= -1.2, +1.1 (table 3.10)
Step 5: Determine design wind pressures	

(a) Outward = (-1.5)(22 psf) = -33.0 psf
(b) Inward = (1.1)(22 psf) = 24.2 psf

Obviously, the outward pressure of 33.0 psf governs the out-of-plane bending load design of the wall stud. Because the load is a lateral pressure (not uplift), the applicable load combination is D+W (refer to table 3.1), resulting in a combined axial and bending load. The axial load would include the tributary building dead load from supported assemblies (that is, walls, floors, and roof). The bending load would then be determined by using the wind pressure of 33.0 psf applied to the stud as a uniform line load on a simply supported beam, calculated as follows:

Uniform line load, w = (wind pressure)(stud spacing)
= (33.0 psf)(1.33 ft*)
*assumes stud spacing of 16 inches on center
= 43.9 plf (LRFD)

Of course, the following gravity load combination would also need to be considered in the stud design (refer to table 3.1):

$$D + 0.75\ L + 0.75\ (L_r \text{ or } S)$$

The stud is actually part of a wall system (that is, sheathing and interior finish) and can add substantially to the calculated bending capacity; refer to chapter 5.

EXAMPLE 3.3 *Design Earthquake Load Calculation*

Given Site ground motion, S_s = 1g
Site soil condition = firm (default)
Roof snow load < 30 psf
Two-story home, 28' x 44' plan, typical construction

Find Design seismic shear on first-story end wall, assuming no interior shear walls or contribution from partition walls

Solution

1. Determine tributary mass (weight) of building to first-story seismic shear.

Roof dead load = (28 ft)(44 ft)(15 psf) = 18,480 lb
Second-story exterior wall dead load = (144 lf)(8 ft)(8 psf) = 9,216 lb
Second-story partition wall dead load = (28 ft)(44 ft)(6 psf) = 7,392 lb
Second-story floor dead load = (28 ft)(44 ft)(10 psf) = 12,320 lb
First-story exterior walls (1/2 height) = (144 lf)(4 ft)(8 psf) = 4,608 lb
Assume first-story interior partition walls are capable of supporting at least the seismic shear produced by their own weight

Total tributary weight = 52,016 lb

2. Determine total seismic story shear on first story.

S_{DS} = 2/3 $(S_s)(F_a)$ (equation 3.8-2)
= 2/3 (1.0g)(1.1) (F_a = 1.1 from table 3.12)
= 0.74 g

V = W (equation 3.8-1)

$$= \frac{1.2\,(0.74g)}{5.5}(52{,}016 \text{ lb}) \qquad (R = 5.5 \text{ from table 3.13})$$

= 8,399 lb

3. Determine design shear load on the 28-foot end walls.

Assume that the building mass is evenly distributed and that stiffness is also reasonably balanced between the two end walls; refer to chapter 6 for additional guidance.

With the above assumption, the load is simply distributed to the end walls according to tributary weight (or plan area) of the building; therefore,

End wall shear = 1/2 (8,399 lb) = 4,200 lb

Note that the design shear load from wind (100 mph gust, exposure B) in example 3.2 is somewhat greater (5,912 lbs).

3.13 References

American Concrete Institute (ACI). 2011. *Building Code Requirements for Structural Concrete and Commentary*, ACI Standard 318. Farmington Hills, MI: ACI.

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