ASCE 7 - MINIMUM DESIGN LOADS FOR BUILDINGS AND OTHER STRUCTURES

Chapter 4
LIVE LOADS

4.1 DEFINITIONS

BALCONY (EXTERIOR): An exterior floor projecting from and supported by a structure without additional independent supports.

DECK: An exterior floor supported on at least two opposing sides by an adjacent structure and/or posts, piers, or other independent supports.

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

GRAB BAR SYSTEM: A bar provided to support body weight in locations such as toilets, showers, and tub enclosures.

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

HANDRAIL: A rail grasped by hand for guidance and support. A handrail assembly includes the handrail, supporting attachments, and structures.

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

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

VEHICLE BARRIER SYSTEM: A system of building components near open sides of a garage floor or ramp, or building walls that act as restraints for vehicles.

4.2 UNIFORMLY DISTRIBUTED LOADS

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

4.2.2 Provision for Partitions. In office buildings or other buildings where partitions will be erected or rearranged, provision for partition weight shall be made, whether or not partitions are shown on the plans. Partition load shall not be less than 15 psf.

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

4.3 CONCENTRATED LOADS

Floors, roofs, and other similar surfaces shall be designed to support safely the uniformly distributed live loads prescribed in Section 4.2 or the concentrated load, in pounds or kilonewtons (kN), given in Table 4-1, whichever produces the greater load effects. Unless otherwise specified, the indicated concentration shall be assumed to be uniformly distributed over an area 2.5 ft (762 mm) square [6.25 ft^2 (0.58 m^2)] and shall be located so as to produce the maximum load effects in the structural members.

4.4 LOADS ON HANDRAILS, GUARDRAIL SYSTEMS, GRAB BAR SYSTEMS, VEHICLE BARRIER SYSTEMS, AND FIXED LADDERS

4.4.1 Loads on Handrails and Guardrail Systems. All handrail assemblies and guardrail systems shall be designed to resist a single concentrated load of 200 lb (0.89 kN) applied in any direction at any point along the top and to transfer this load through the supports to the structure.

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

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

Intermediate rails (all those except the handrail), balusters, and panel fillers shall be designed to withstand a horizontally applied normal load of 50 lb (0.22 kN) on an area not to exceed 1 ft square (305 mm square) including openings and space between rails. Reactions due to this loading are not required to be superimposed with those of either preceding paragraph.

4.4.2 Loads on Grab Bar Systems. Grab bar systems shall be designed to resist a single concentrated load of 250 lb (1.11 kN) applied in any direction at any point.

4.4.3 Loads on Vehicle Barrier Systems. Vehicle barrier systems for passenger cars shall be designed to resist a single load of 6,000 lb (26.70 kN) applied horizontally in any direction to the barrier system, and shall have anchorages or attachments capable of transferring this load to the structure. For design of the system, the load shall be assumed to act at a minimum height of 1 ft 6 in. (460 mm) above the floor or ramp surface on an area not to exceed 1 foot square (305 mm square), and is not required to be assumed to act concurrently with any handrail or guardrail loadings specified in Section 4.4.1. Garages accommodating trucks and buses shall be designed in accordance with an approved method, which contains provision for traffic railings.

4.4.4 Loads on Fixed Ladders. The minimum design live load on fixed ladders with rungs shall be a single concentrated load of 300 lb (1.33 kN), and shall be applied at any point to produce the maximum load effect on the element being considered. The number and position of additional concentrated live load units shall be a minimum of 1 unit of 300 lb (1.33 kN) for every 10 ft (3,048 mm) of ladder height.

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

4.5 LOADS NOT SPECIFIED

For occupancies or uses not designated in Sections 4.2 or 4.3, the live load shall be determined in accordance with a method approved by the authority having jurisdiction.

4.6 PARTIAL LOADING

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

4.7 IMPACT LOADS

The live loads specified in Sections 4.2.1 and 4.4.2 shall be assumed to include adequate allowance for ordinary impact conditions. Provision shall be made in the structural design for uses and loads that involve unusual vibration and impact forces.

4.7.1 Elevators. All elevator loads shall be increased by 100 percent for impact and the structural supports shall be designed within the limits of deflection prescribed by ANSI A17.2 and ANSI/ASME A17.1.

4.7.2 Machinery. For the purpose of design, the weight of machinery and moving loads shall be increased as follows to allow for impact: (1) elevator machinery, 100 percent; (2) light machinery, shaft- or motor-driven, 20 percent; (3) reciprocating machinery or power-driven units, 50 percent; and (4) hangers for floors or balconies, 33 percent. All percentages shall be increased where specified by the manufacturer.

4.8 REDUCTION IN LIVE LOADS

Except for roof uniform live loads, all other minimum uniformly distributed live loads, L_o in Table 4-1, may be reduced according to the following provisions.

4.8.1 General. Subject to the limitations of Sections 4.8.2 through 4.8.5, members for which a value of $K_{LL}A_T$ is 400 ft^2 (37.16 m^2) or more are permitted to be designed for a reduced live load in accordance with the following formula:

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \tag{4-1}$$

In SI:

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

where

L = reduced design live load per ft^2 (m^2) of area supported by the member

L_o = unreduced design live load per ft^2 (m^2) of area supported by the member (see Table 4-1)

K_{LL} = live load element factor (see Table 4-2)

A_T = tributary area in ft^2 (m^2)

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

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

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

4.8.3 Passenger Car Garages. The live loads shall not be reduced in passenger car garages.

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

4.8.4 Special Occupancies. Live loads of 100 lb/ft^2 (4.79 kN/m^2) or less shall not be reduced in public assembly occupancies.

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

4.9 REDUCTION IN ROOF LIVE LOADS

The minimum uniformly distributed roof live loads, L_o in Table 4-1, are permitted to be reduced according to the following provisions.

4.9.1 Flat, Pitched, and Curved Roofs. Ordinary flat, pitched, and curved roofs are permitted to be designed for a reduced roof live load, as specified in Eq. 4-2 or other controlling combinations of loads, as discussed in Chapter 2, whichever produces the greater load. In structures such as greenhouses, where special scaffolding is used as a work surface for workmen and materials during maintenance and repair operations, a lower roof load than specified in Eq. 4-2 shall not be used unless approved by the authority having jurisdiction. On such structures, the minimum roof live load shall be 12 psf (0.58 kN/m^2).

$$L_r = L_o R_1 R_2 \quad \text{where } 12 \le L_r \le 20 \tag{4-2}$$

In SI:

$$L_r = L_o R_1 R_2 \quad \text{where } 0.58 \le L_r \le 0.96$$

where

L_r = reduced roof live load per ft^2 (m^2) of horizontal projection in pounds per ft^2 (kN/m^2)

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

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

In SI:

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

where A_t = tributary area in ft^2 (m^2) supported by any structural member and

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

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

4.9.2 Special Purpose Roofs. Roofs that have an occupancy function, such as roof gardens, assembly purposes, or other special purposes are permitted to have their uniformly distributed live load reduced in accordance with the requirements of Section 4.8.

4.10 CRANE LOADS

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

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

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

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

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

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

4.11 CONSENSUS STANDARDS AND OTHER REFERENCED DOCUMENTS

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

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

ANSI A17.2
Section 4.7.1
American National Standard Practice for the Inspection of Elevators, Escalators, and Moving Walks (Inspectors' Manual), 1988.

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

ANSI/ASME A17.1
Section 4.7.1
American National Standard Safety Code for Elevators and Escalators, 1993.

TABLE 4-1 MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS

Occupancy or Use	Uniform psf (kN/m²)	Conc. lb (kN)
Apartments (see *Residential*)		
Access floor systems		
Office use	50 (2.4)	2,000 (8.9)
Computer use	100 (4.79)	2,000 (8.9)
Armories and drill rooms	150 (7.18)	
Assembly areas and theaters		
Fixed seats (fastened to floor)	60 (2.87)	
Lobbies	100 (4.79)	
Movable seats	100 (4.79)	
Platforms (assembly)	100 (4.79)	
Stage floors	150 (7.18)	
Balconies (exterior)	100 (4.79)	
On one- and two-family residences only, and not exceeding 100 ft² (9.3 m²)	60 (2.87)	
Bowling alleys, poolrooms, and similar recreational areas	75 (3.59)	
Catwalks for maintenance access	40 (1.92)	300 (1.33)
Corridors		
First floor	100 (4.79)	
Other floors, same as occupancy served except as indicated		
Dance halls and ballrooms	100 (4.79)	
Decks (patio and roof)		
Same as area served, or for the type of occupancy accommodated		
Dining rooms and restaurants	100 (4.79)	
Dwellings (see *Residential*)		
Elevator machine room grating (on area of 4 in.² [2,580 mm²])		300 (1.33)
Finish light floor plate construction (on area of 1 in.² [645 mm²])		200 (0.89)
Fire escapes	100 (4.79)	
On single-family dwellings only	40 (1.92)	
Fixed ladders	See Section 4.4	
Garages (passenger vehicles only)	40 (1.92)[a,b]	
Trucks and buses		
Grandstands (see *Stadiums and arenas, Bleachers*)		
Gymnasiums—main floors and balconies	100 (4.79)	
Handrails, guardrails, and grab bars	See Section 4.4	
Hospitals		
Operating rooms, laboratories	60 (2.87)	1,000 (4.45)
Patient rooms	40 (1.92)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Hotels (see *Residential*)		
Libraries		
Reading rooms	60 (2.87)	1,000 (4.45)
Stack rooms	150 (7.18)[c]	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Manufacturing		
Light	125 (6.00)	2,000 (8.90)
Heavy	250 (11.97)	3,000 (13.40)
Marquees	75 (3.59)	
Office Buildings		
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Lobbies and first-floor corridors	100 (4.79)	2,000 (8.90)
Offices	50 (2.40)	2,000 (8.90)
Corridors above first floor	80 (3.83)	2,000 (8.90)
Penal Institutions		
Cell blocks	40 (1.92)	
Corridors	100 (4.79)	
Residential		
Dwellings (one- and two-family)		
Uninhabitable attics without storage	10 (0.48)	
Uninhabitable attics with storage	20 (0.96)	
Habitable attics and sleeping areas	30 (1.44)	
All other areas except stairs and balconies	40 (1.92)	
Hotels and multifamily houses		
Private rooms and corridors serving them	40 (1.92)	
Public rooms and corridors serving them	100 (4.79)	
Reviewing stands, grandstands, and bleachers	100 (4.79)[d]	

TABLE 4-1 MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS (continued)

Occupancy or Use	Uniform psf (kN/m^2)	Conc. lb (kN)
Roofs		
Ordinary flat, pitched, and curved roofs	20 (0.96)^h	
Roofs used for promenade purposes	60 (2.87)	
Roofs used for roof gardens or assembly purposes	100 (4.79)	
Roofs used for other special purposes	^{i}	^{i}
Awnings and canopies		
Fabric construction supported by a lightweight rigid skeleton structure	5 (0.24) nonreduceable	
All other construction	20 (0.96)	
Primary roof members, exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages		2,000 (8.9)
All other occupancies		300 (1.33)
All roof surfaces subject to maintenance workers		300 (1.33)
Schools		
Classrooms	40 (1.92)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
First-floor corridors	100 (4.79)	1,000 (4.45)
Scuttles, skylight ribs, and accessible ceilings		200 (0.89)
Sidewalks, vehicular driveways, and yards subject to trucking	250 (11.97)^e	8,000 (35.60)^f
Stadiums and arenas		
Bleachers	100 (4.79)^d	
Fixed seats (fastened to floor)	60 (2.87)^d	
Stairs and exit ways	100 (4.79)	^{g}
One- and two-family residences only	40 (1.92)	
Storage areas above ceilings	20 (0.96)	
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Light	125 (6.00)	
Heavy	250 (11.97)	
Stores		
Retail		
First floor	100 (4.79)	1,000 (4.45)
Upper floors	75 (3.59)	1,000 (4.45)
Wholesale, all floors	125 (6.00)	1,000 (4.45)
Vehicle barriers	See Section 4.4	
Walkways and elevated platforms (other than exit ways)	60 (2.87)	
Yards and terraces, pedestrian	100 (4.79)	

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

^{b}Garages accommodating trucks and buses shall be designed in accordance with an approved method, which contains provisions for truck and bus loadings.

^{c}The loading applies to stack room floors that support nonmobile, double-faced library book stacks subject to the following limitations: (1) The nominal book stack unit height shall not exceed 90 in. (2290 mm); (2) the nominal shelf depth shall not exceed 12 in. (305 mm) for each face; and (3) parallel rows of double-faced book stacks shall be separated by aisles not less than 36 in. (914 mm) wide.

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

^{e}Other uniform loads in accordance with an approved method, which contains provisions for truck loadings, shall also be considered where appropriate.

^{f}The concentrated wheel load shall be applied on an area of 4.5 in. by 4.5 in. (114 mm by 114 mm) footprint of a jack.

^{g}Minimum concentrated load on stair treads (on area of 4 in.^{2} [2,580 mm^2]) is 300 lb (1.33 kN).

^{h}Where uniform roof live loads are reduced to less than 20 lb/ft^2 (0.96 kN/m^2) in accordance with Section 4.9.1 and are applied to the design of structural members arranged so as to create continuity, the reduced roof live load shall be applied to adjacent spans or to alternate spans, whichever produces the greatest unfavorable effect.

^{i}Roofs used for other special purposes shall be designed for appropriate loads as approved by the authority having jurisdiction.

TABLE 4-2 LIVE LOAD ELEMENT FACTOR, K_{LL}

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

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

Chapter 6
WIND LOADS

6.1 GENERAL

6.1.1 Scope. Buildings and other structures, including the Main Wind-Force Resisting System (MWFRS) and all components and cladding thereof, shall be designed and constructed to resist wind loads as specified herein.

6.1.2 Allowed Procedures. The design wind loads for buildings and other structures, including the MWFRS and component and cladding elements thereof, shall be determined using one of the following procedures: (1) Method 1—Simplified Procedure as specified in Section 6.4 for buildings meeting the requirements specified therein; (2) Method 2—Analytical Procedure as specified in Section 6.5 for buildings meeting the requirements specified therein; (3) Method 3—Wind Tunnel Procedure as specified in Section 6.6.

6.1.3 Wind Pressures Acting on Opposite Faces of Each Building Surface. In the calculation of design wind loads for the MWFRS and for components and cladding for buildings, the algebraic sum of the pressures acting on opposite faces of each building surface shall be taken into account.

6.1.4 Minimum Design Wind Loading. The design wind load, determined by any one of the procedures specified in Section 6.1.2, shall be not less than specified in this section.

6.1.4.1 Main Wind-Force Resisting System. The wind load to be used in the design of the MWFRS for an enclosed or partially enclosed building or other structure shall not be less than 10 lb/ft^2 (0.48 kN/m^2) multiplied by the area of the building or structure projected onto a vertical plane normal to the assumed wind direction. The design wind force for open buildings and other structures shall be not less than 10 lb/ft^2 (0.48 kN/m^2) multiplied by the area A_f.

6.1.4.2 Components and Cladding. The design wind pressure for components and cladding of buildings shall not be less than a net pressure of 10 lb/ft^2 (0.48 kN/m^2) acting in either direction normal to the surface.

6.2 DEFINITIONS

The following definitions apply only to the provisions of Chapter 6:

APPROVED: Acceptable to the authority having jurisdiction.

BASIC WIND SPEED, *V*: Three-second gust speed at 33 ft (10 m) above the ground in Exposure C (see Section 6.5.6.3) as determined in accordance with Section 6.5.4.

BUILDING, ENCLOSED: A building that does not comply with the requirements for open or partially enclosed buildings.

BUILDING ENVELOPE: Cladding, roofing, exterior walls, glazing, door assemblies, window assemblies, skylight assemblies, and other components enclosing the building.

BUILDING AND OTHER STRUCTURE, FLEXIBLE: Slender buildings and other structures that have a fundamental natural frequency less than 1 Hz.

BUILDING, LOW-RISE: Enclosed or partially enclosed buildings that comply with the following conditions:

1. Mean roof height h less than or equal to 60 ft (18 m).
2. Mean roof height h does not exceed least horizontal dimension.

BUILDING, OPEN: A building having each wall at least 80 percent open. This condition is expressed for each wall by the equation $A_o \geq 0.8A_g$ where

A_o = total area of openings in a wall that receives positive external pressure, in ft^2 (m^2)

A_g = the gross area of that wall in which A_o is identified, in ft^2 (m^2)

BUILDING, PARTIALLY ENCLOSED: A building that complies with both of the following conditions:

1. The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10 percent.
2. The total area of openings in a wall that receives positive external pressure exceeds 4 ft^2 (0.37 m^2) or 1 percent of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20 percent.

These conditions are expressed by the following equations:

1. $A_o > 1.10A_{oi}$
2. $A_o > 4$ sq ft (0.37 m^2) or $>0.01A_g$, whichever is smaller, and $A_{oi}/A_{gi} \leq 0.20$

where

A_o, A_g are as defined for Open Building

A_{oi} = the sum of the areas of openings in the building envelope (walls and roof) not including A_o, in ft^2 (m^2)

A_{gi} = the sum of the gross surface areas of the building envelope (walls and roof) not including A_g, in ft^2 (m^2)

BUILDING OR OTHER STRUCTURE, REGULAR-SHAPED: A building or other structure having no unusual geometrical irregularity in spatial form.

BUILDING OR OTHER STRUCTURES, RIGID: A building or other structure whose fundamental frequency is greater than or equal to 1 Hz.

BUILDING, SIMPLE DIAPHRAGM: A building in which both windward and leeward wind loads are transmitted through floor and roof diaphragms to the same vertical MWFRS (e.g., no structural separations).

COMPONENTS AND CLADDING: Elements of the building envelope that do not qualify as part of the MWFRS.

characteristics, shall be designed using recognized literature documenting such wind load effects or shall use the wind tunnel procedure specified in Section 6.6.

6.5.2.1 Shielding. There shall be no reductions in velocity pressure due to apparent shielding afforded by buildings and other structures or terrain features.

6.5.2.2 Air Permeable Cladding. Design wind loads determined from Section 6.5 shall be used for air permeable cladding unless approved test data or recognized literature demonstrate lower loads for the type of air permeable cladding being considered.

6.5.3 Design Procedure.

1. The *basic wind speed V* and *wind directionality factor* K_d shall be determined in accordance with Section 6.5.4.
2. An *importance factor I* shall be determined in accordance with Section 6.5.5.
3. An *exposure category* or *exposure categories* and *velocity pressure exposure coefficient* K_z or K_h, as applicable, shall be determined for each wind direction in accordance with Section 6.5.6.
4. A *topographic factor* K_{zt} shall be determined in accordance with Section 6.5.7.
5. A *gust effect factor* G or G_f, as applicable, shall be determined in accordance with Section 6.5.8.
6. An *enclosure classification* shall be determined in accordance with Section 6.5.9.
7. *Internal pressure coefficient* GC_{pi} shall be determined in accordance with Section 6.5.11.1.
8. *External pressure coefficients* C_p or GC_{pf}, or *force coefficients* C_f, as applicable, shall be determined in accordance with Section 6.5.11.2 or 6.5.11.3, respectively.
9. *Velocity pressure* q_z or q_h, as applicable, shall be determined in accordance with Section 6.5.10.
10. *Design wind load* p or F shall be determined in accordance with Sections 6.5.12, 6.5.13, 6.5.14, and 6.5.15, as applicable.

6.5.4 Basic Wind Speed. The basic wind speed, V, used in the determination of design wind loads on buildings and other structures shall be as given in Fig. 6-1 except as provided in Sections 6.5.4.1 and 6.5.4.2. The wind shall be assumed to come from any horizontal direction.

6.5.4.1 Special Wind Regions. The basic wind speed shall be increased where records or experience indicate that the wind speeds are higher than those reflected in Fig. 6-1. Mountainous terrain, gorges, and special regions shown in Fig. 6-1 shall be examined for unusual wind conditions. The authority having jurisdiction shall, if necessary, adjust the values given in Fig. 6-1 to account for higher local wind speeds. Such adjustment shall be based on meteorological information and an estimate of the basic wind speed obtained in accordance with the provisions of Section 6.5.4.2.

6.5.4.2 Estimation of Basic Wind Speeds from Regional Climatic Data. In areas outside hurricane-prone regions, regional climatic data shall only be used in lieu of the basic wind speeds given in Fig. 6-1 when (1) approved extreme-value statistical-analysis procedures have been employed in reducing the data; and (2) the length of record, sampling error, averaging time, anemometer height, data quality, and terrain exposure of the anemometer have been taken into account. Reduction in basic wind speed below that of Fig. 6-1 shall be permitted.

In hurricane-prone regions, wind speeds derived from simulation techniques shall only be used in lieu of the basic wind speeds given in Fig. 6-1 when (1) approved simulation and extreme value statistical analysis procedures are used (the use of regional wind speed data obtained from anemometers is not permitted to define the hurricane wind-speed risk along the Gulf and Atlantic coasts, the Caribbean, or Hawaii) and (2) the design wind speeds resulting from the study shall not be less than the resulting 500-year return period wind speed divided by $\sqrt{1.5}$.

In areas outside hurricane-prone regions, when the basic wind speed is estimated from regional climatic data, the basic wind speed shall be not less than the wind speed associated with an annual probability of 0.02 (50-year mean recurrence interval), and the estimate shall be adjusted for equivalence to a 3-s gust wind speed at 33 ft (10 m) above ground in exposure Category C. The data analysis shall be performed in accordance with this chapter.

6.5.4.3 Limitation. Tornadoes have not been considered in developing the basic wind-speed distributions.

6.5.4.4 Wind Directionality Factor. The wind directionality factor, K_d, shall be determined from Table 6-4. This factor shall only be applied when used in conjunction with load combinations specified in Sections 2.3 and 2.4.

6.5.5 Importance Factor. An importance factor, I, for the building or other structure shall be determined from Table 6-1 based on building and structure categories listed in Table 1-1.

6.5.6 Exposure. For each wind direction considered, the upwind exposure category shall be based on ground surface roughness that is determined from natural topography, vegetation, and constructed facilities.

6.5.6.1 Wind Directions and Sectors. For each selected wind direction at which the wind loads are to be evaluated, the exposure of the building or structure shall be determined for the two upwind sectors extending 45° either side of the selected wind direction. The exposures in these two sectors shall be determined in accordance with Sections 6.5.6.2 and 6.5.6.3 and the exposure resulting in the highest wind loads shall be used to represent the winds from that direction.

6.5.6.2 Surface Roughness Categories. A ground surface roughness within each 45° sector shall be determined for a distance upwind of the site as defined in Section 6.5.6.3 from the categories defined in the following text, for the purpose of assigning an exposure category as defined in Section 6.5.6.3.

Surface Roughness B: Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness C: Open terrain with scattered obstructions having heights generally less than 30 ft (9.1 m). This category includes flat open country, grasslands, and all water surfaces in hurricane prone regions.

Surface Roughness D: Flat, unobstructed areas and water surfaces outside hurricane prone regions. This category includes smooth mud flats, salt flats, and unbroken ice.

6.5.6.3 Exposure Categories
Exposure B: Exposure B shall apply where the ground surface roughness condition, as defined by Surface Roughness B, prevails in the upwind direction for a distance of at least 2,600 ft (792 m) or 20 times the height of the building, whichever is greater.

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} \tag{6-12}$$

$$R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) \quad \text{for } \eta > 0 \tag{6-13a}$$

$$R_\ell = 1 \quad \text{for } \eta = 0 \tag{6-13b}$$

where the subscript ℓ in Eq. 6-13 shall be taken as h, B, and L, respectively, where h, B, and L are defined in Section 6.3.

n_1 = building natural frequency

$R_\ell = R_h$ setting $\eta = 4.6 n_1 h / \bar{V}_{\bar{z}}$

$R_\ell = R_B$ setting $\eta = 4.6 n_1 EB / \bar{V}_{\bar{z}}$

$R_\ell = R_L$ setting $\eta = 15.4 n_1 L / \bar{V}_{\bar{z}}$

β = damping ratio, percent of critical

$\bar{V}_{\bar{z}}$ = mean hourly wind speed (ft/s) at height $\bar{z}$ determined from Eq. 6-14.

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V \left(\frac{88}{60}\right) \tag{6-14}$$

In SI: $\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{10}\right)^{\bar{\alpha}} V$

where $\bar{b}$ and $\bar{\alpha}$ are constants listed in Table 6-2 and V is the basic wind speed in mi/h.

6.5.8.3 Rational Analysis. In lieu of the procedure defined in Sections 6.5.8.1 and 6.5.8.2, determination of the gust-effect factor by any rational analysis defined in the recognized literature is permitted.

6.5.8.4 Limitations. Where combined gust-effect factors and pressure coefficients (GC_p, GC_{pi}, and GC_{pf}) are given in figures and tables, the gust-effect factor shall not be determined separately.

6.5.9 Enclosure Classifications.

6.5.9.1 General. For the purpose of determining internal pressure coefficients, all buildings shall be classified as enclosed, partially enclosed, or open as defined in Section 6.2.

6.5.9.2 Openings. A determination shall be made of the amount of openings in the building envelope to determine the enclosure classification as defined in Section 6.5.9.1.

6.5.9.3 Wind-Borne Debris. Glazing in buildings located in wind-borne debris regions shall be protected with an impact-resistant covering or be impact-resistant glazing according to the requirements specified in ASTM E1886 and ASTM E1996 or other approved test methods and performance criteria. The levels of impact resistance shall be a function of Missile Levels and Wind Zones specified in ASTM E1886 and ASTM E1996.

EXCEPTIONS:

1. Glazing in Category II, III, or IV buildings located over 60 ft (18.3 m) above the ground and over 30 ft (9.2 m) above aggregate surface roofs located within 1,500 ft (458 m) of the building shall be permitted to be unprotected.
2. Glazing in Category I buildings shall be permitted to be unprotected.

6.5.9.4 Multiple Classifications. If a building by definition complies with both the "open" and "partially enclosed" definitions, it shall be classified as an "open" building. A building that does not comply with either the "open" or "partially enclosed" definitions shall be classified as an "enclosed" building.

6.5.10 Velocity Pressure. Velocity pressure, q_z, evaluated at height z shall be calculated by the following equation:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \text{ (lb/ft}^2\text{)} \tag{6-15}$$

[In SI: $q_z = 0.613 K_z K_{zt} K_d V^2 I$ (N/m^2); V in m/s]

where K_d is the wind directionality factor defined in Section 6.5.4.4, K_z is the velocity pressure exposure coefficient defined in Section 6.5.6.6, K_{zt} is the topographic factor defined in Section 6.5.7.2, and q_h is the velocity pressure calculated using Eq. 6-15 at mean roof height h.

The numerical coefficient 0.00256 (0.613 in SI) shall be used except where sufficient climatic data are available to justify the selection of a different value of this factor for a design application.

6.5.11 Pressure and Force Coefficients.

6.5.11.1 Internal Pressure Coefficient. Internal pressure coefficients, GC_{pi}, shall be determined from Fig. 6-5 based on building enclosure classifications determined from Section 6.5.9.

6.5.11.1.1 Reduction Factor for Large Volume Buildings, R_i. For a partially enclosed building containing a single, unpartitioned large volume, the internal pressure coefficient, GC_{pi}, shall be multiplied by the following reduction factor, R_i:

$R_i = 1.0$ or

$$R_i = 0.5\left(1 + \frac{1}{\sqrt{1 + \frac{V_i}{22{,}800 A_{og}}}}\right) \le 1.0 \tag{6-16}$$

where

A_{og} = total area of openings in the building envelope (walls and roof, in ft^2)

V_i = unpartitioned internal volume, in ft^3

6.5.11.2 External Pressure Coefficients.

6.5.11.2.1 Main Wind-Force Resisting Systems. External pressure coefficients for MWFRSs C_p are given in Figs. 6-6, 6-7, and 6-8. Combined gust effect factor and external pressure coefficients, GC_{pf}, are given in Fig. 6-10 for low-rise buildings. The pressure coefficient values and gust effect factor in Fig. 6-10 shall not be separated.

6.5.11.2.2 Components and Cladding. Combined gust-effect factor and external pressure coefficients for components and cladding GC_p are given in Figs. 6-11 through 6-17. The pressure coefficient values and gust-effect factor shall not be separated.

6.5.11.3 Force Coefficients. Force coefficients C_f are given in Figs. 6-20 through 6-23.

6.5.11.4 Roof Overhangs.

6.5.11.4.1 Main Wind-Force Resisting System. Roof overhangs shall be designed for a positive pressure on the bottom surface of windward roof overhangs corresponding to $C_p = 0.8$ in combination with the pressures determined from using Figs. 6-6 and 6-10.

6.5.11.4.2 Components and Cladding. For all buildings, roof overhangs shall be designed for pressures determined from pressure coefficients given in Figs. 6-11B,C,D.

6.5.11.5 Parapets.

6.5.11.5.1 Main Wind-Force Resisting System. The pressure coefficients for the effect of parapets on the MWFRS loads are given in Section 6.5.12.2.4

Wind Directionality Factor, K_d

Table 6-4

Structure Type	Directionality Factor K_d*
Buildings	
Main Wind Force Resisting System	0.85
Components and Cladding	0.85
Arched Roofs	0.85
Chimneys, Tanks, and Similar Structures	
Square	0.90
Hexagonal	0.95
Round	0.95
Solid Signs	0.85
Open Signs and Lattice Framework	0.85
Trussed Towers	
Triangular, square, rectangular	0.85
All other cross sections	0.95

*Directionality Factor K_d has been calibrated with combinations of loads specified in Section 2. This factor shall only be applied when used in conjunction with load combinations specified in 2.3 and 2.4.

Velocity Pressure Exposure Coefficients, K_h and K_z

Table 6-3

Height above ground level, z		Exposure (Note 1)			
		B		C	D
ft	(m)	Case 1	Case 2	Cases 1 & 2	Cases 1 & 2
0-15	(0-4.6)	0.70	0.57	0.85	1.03
20	(6.1)	0.70	0.62	0.90	1.08
25	(7.6)	0.70	0.66	0.94	1.12
30	(9.1)	0.70	0.70	0.98	1.16
40	(12.2)	0.76	0.76	1.04	1.22
50	(15.2)	0.81	0.81	1.09	1.27
60	(18)	0.85	0.85	1.13	1.31
70	(21.3)	0.89	0.89	1.17	1.34
80	(24.4)	0.93	0.93	1.21	1.38
90	(27.4)	0.96	0.96	1.24	1.40
100	(30.5)	0.99	0.99	1.26	1.43
120	(36.6)	1.04	1.04	1.31	1.48
140	(42.7)	1.09	1.09	1.36	1.52
160	(48.8)	1.13	1.13	1.39	1.55
180	(54.9)	1.17	1.17	1.43	1.58
200	(61.0)	1.20	1.20	1.46	1.61
250	(76.2)	1.28	1.28	1.53	1.68
300	(91.4)	1.35	1.35	1.59	1.73
350	(106.7)	1.41	1.41	1.64	1.78
400	(121.9)	1.47	1.47	1.69	1.82
450	(137.2)	1.52	1.52	1.73	1.86
500	(152.4)	1.56	1.56	1.77	1.89

Notes:

1. **Case 1:** a. All components and cladding.
   b. Main wind force resisting system in low-rise buildings designed using Figure 6-10.

   Case 2: a. All main wind force resisting systems in buildings except those in low-rise buildings designed using Figure 6-10.
   b. All main wind force resisting systems in other structures.

2. The velocity pressure exposure coefficient K_z may be determined from the following formula:

   For 15 ft. $\leq z \leq z_g$ — For $z < 15$ ft.

   $K_z = 2.01\ (z/z_g)^{2/\alpha}$ — $K_z = 2.01\ (15/z_g)^{2/\alpha}$

   Note: z shall not be taken less than 30 feet for Case 1 in exposure B.

3. α and z_g are tabulated in Table 6-2.
4. Linear interpolation for intermediate values of height z is acceptable.
5. Exposure categories are defined in 6.5.6.

Importance Factor, I (Wind Loads)

Table 6-1

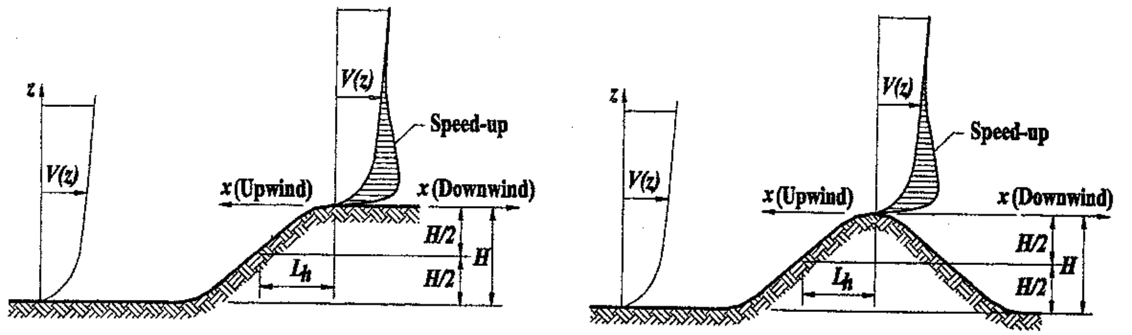
Category	Non-Hurricane Prone Regions and Hurricane Prone Regions with V = 85-100 mph and Alaska	Hurricane Prone Regions with V > 100 mph
I	0.87	0.77
II	1.00	1.00
III	1.15	1.15
IV	1.15	1.15

Note:

1. The building and structure classification categories are listed in Table 1-1.

Topographic Factor, K_{zt} – Method 2

Figure 6-4



ESCARPMENT **2-D RIDGE OR 3-D AXISYMMETRICAL HILL**

Topographic Multipliers for Exposure C

H/L_h	K_1 Multiplier			x/L_h	K_2 Multiplier		z/L_h	K_3 Multiplier		
	2-D Ridge	2-D Escarp.	3-D Axisym. Hill		2-D Escarp.	All Other Cases		2-D Ridge	2-D Escarp.	3-D Axisym. Hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Notes:

1. For values of H/L_h, x/L_h and z/L_h other than those shown, linear interpolation is permitted.
2. For $H/L_h > 0.5$, assume $H/L_h = 0.5$ for evaluating K_1 and substitute 2H for L_h for evaluating K_2 and K_3.
3. Multipliers are based on the assumption that wind approaches the hill or escarpment along the direction of maximum slope.
4. Notation:

H: Height of hill or escarpment relative to the upwind terrain, in feet (meters).

L_h: Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in feet (meters).

K_1: Factor to account for shape of topographic feature and maximum speed-up effect.

K_2: Factor to account for reduction in speed-up with distance upwind or downwind of crest.

K_3: Factor to account for reduction in speed-up with height above local terrain.

x: Distance (upwind or downwind) from the crest to the building site, in feet (meters).

z: Height above local ground level, in feet (meters).

μ: Horizontal attenuation factor.

γ: Height attenuation factor.

Main Wind Force Res. Sys. / Comp and Clad. – Method 2		All Heights
Figure 6-5	Internal Pressure Coefficient, GC_{pi}	Walls & Roofs
Enclosed, Partially Enclosed, and Open Buildings		

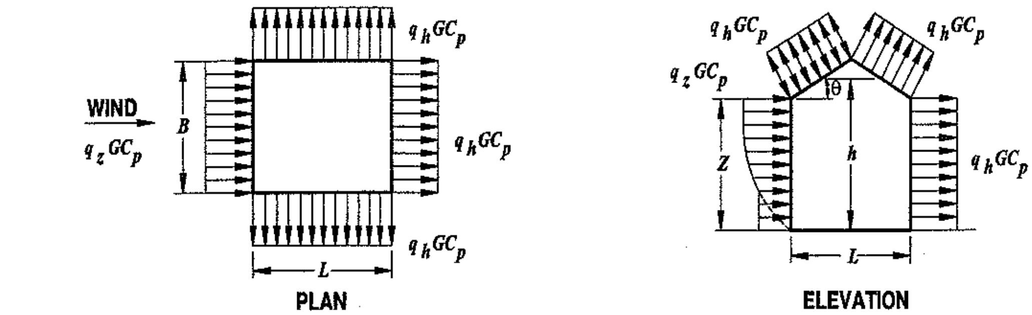
Enclosure Classification	GC_{pi}
Open Buildings	0.00
Partially Enclosed Buildings	+0.55 -0.55
Enclosed Buildings	+0.18 -0.18

Notes:

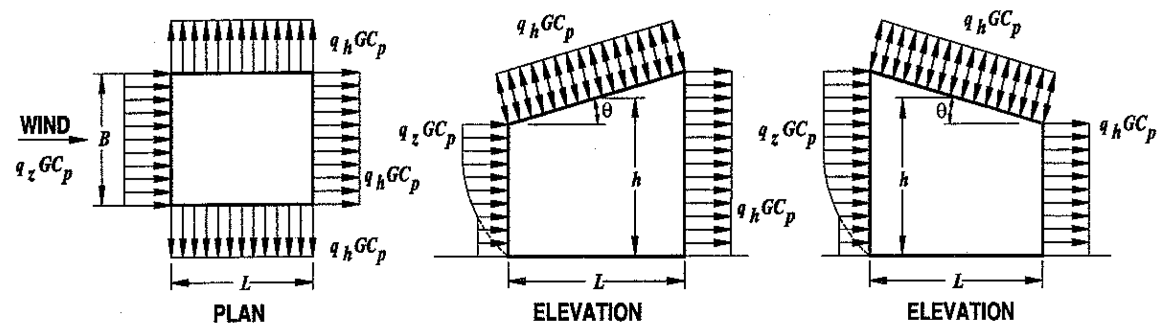
1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
2. Values of GC_{pi} shall be used with q_z or q_h as specified in 6.5.12.
3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:

   (i) a positive value of GC_{pi} applied to all internal surfaces
   (ii) a negative value of GC_{pi} applied to all internal surfaces

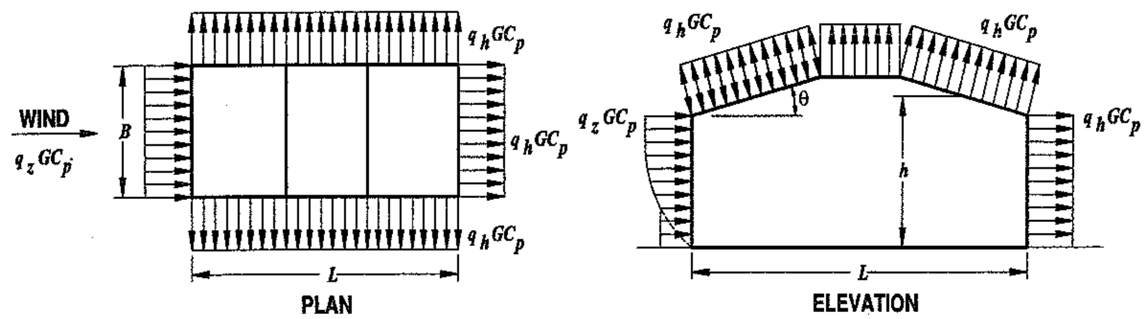
Main Wind Force Resisting System – Method 2		All Heights
Figure 6-6	External Pressure Coefficients, C_p	Walls & Roofs
Enclosed, Partially Enclosed Buildings		



GABLE, HIP ROOF



MONOSLOPE ROOF (NOTE 4)



MANSARD ROOF (NOTE 8)

Main Wind Force Resisting System – Method 2		All Heights
Figure 6-6 (con't)	External Pressure Coefficients, C_p	**Walls & Roofs**
Enclosed, Partially Enclosed Buildings		

Wall Pressure Coefficients, C_p

Surface	L/B	C_p	Use With
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥4	-0.2	
Side Wall	All values	-0.7	q_h

Roof Pressure Coefficients, C_p, for use with q_h

Wind Direction	Windward									Leeward		
	Angle, θ (degrees)									Angle, θ (degrees)		
	h/L	10	15	20	25	30	35	45	≥60#	10	15	≥20
Normal to ridge for θ ≥ 10°	≤0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01 θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01 θ	-0.5	-0.5	-0.6
	≥1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01 θ	-0.7	-0.6	-0.6

Wind Direction	h/L	Horiz distance from windward edge	C_p
Normal to ridge for θ < 10 and Parallel to ridge for all θ	≤ 0.5	0 to h/2	-0.9, -0.18
		h/2 to h	-0.9, -0.18
		h to 2 h	-0.5, -0.18
		> 2h	-0.3, -0.18
	≥ 1.0	0 to h/2	-1.3**, -0.18
		> h/2	-0.7, -0.18

*Value is provided for interpolation purposes.

**Value can be reduced linearly with area over which it is applicable as follows

Area (sq ft)	Reduction Factor
≤ 100 (9.3 sq m)	1.0
200 (23.2 sq m)	0.9
≥ 1000 (92.9 sq m)	0.8

Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. Linear interpolation is permitted for values of *L/B, h/L* and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
3. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
4. For monoslope roofs, entire roof surface is either a windward or leeward surface.
5. For flexible buildings use appropriate G_f as determined by Section 6.5.8.
6. Refer to Figure 6-7 for domes and Figure 6-8 for arched roofs.
7. Notation:
   B: Horizontal dimension of building, in feet (meter), measured normal to wind direction.
   L: Horizontal dimension of building, in feet (meter), measured parallel to wind direction.
   h: Mean roof height in feet (meters), except that eave height shall be used for θ ≤ 10 degrees.
   z: Height above ground, in feet (meters).
   G: Gust effect factor.
   q_z, q_h: Velocity pressure, in pounds per square foot (N/m^2), evaluated at respective height.
   θ: Angle of plane of roof from horizontal, in degrees.
8. For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table.
9. Except for MWFRS's at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.

#For roof slopes greater than 80°, use C_p = 0.8

Chapter 7
SNOW LOADS

7.1 SYMBOLS AND NOTATION

C_e = exposure factor as determined from Table 7-2
C_s = slope factor as determined from Fig. 7-2
C_t = thermal factor as determined from Table 7-3
h_b = height of balanced snow load determined by dividing p_s by γ, in ft (m)
h_c = clear height from top of balanced snow load to (1) closest point on adjacent upper roof, (2) top of parapet, or (3) top of a projection on the roof, in ft (m)
h_d = height of snow drift, in ft (m)
h_o = height of obstruction above the surface of the roof, in ft (m)
I = importance factor as determined from Table 7-4
l_u = length of the roof upwind of the drift, in ft (m)
L = roof length parallel to the ridge line, in ft (m)
p_d = maximum intensity of drift surcharge load, in lb/ft^2 (kN/m^2)
p_f = snow load on flat roofs ("flat" = roof slope $\leq 5°$), in lb/ft^2 (kN/m^2)
p_g = ground snow load as determined from Fig. 7-1 and Table 7-1; or a site-specific analysis, in lb/ft^2 (kN/m^2)
p_s = sloped roof snow load, in lb/ft^2 (kN/m^2)
s = separation distance between buildings, in ft (m)
S = roof slope run for a rise of one
θ = roof slope on the leeward side, in degrees
w = width of snow drift, in ft (m)
W = horizontal distance from eave to ridge, in ft (m)
γ = snow density, in lb/ft^3 (kN/m^3) as determined from Eq. 7-3

7.2 GROUND SNOW LOADS, p_g

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

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

7.3 FLAT ROOF SNOW LOADS, p_f

The snow load, p_f, on a roof with a slope equal to or less than 5° (1 in./ft = 4.76°) shall be calculated in lb/ft^2 (kN/m^2) using the following formula:

$$p_f = 0.7\, C_e C_t I p_g \tag{7-1}$$

but not less than the following minimum values for low slope roofs as defined in Section 7.3.4:

where p_g is 20 lb/ft^2 (0.96 kN/m^2) or less,

$$p_f = (I)\, p_g \text{ (Importance factor times } p_g\text{)}$$

where p_g exceeds 20 lb/ft^2 (0.96 kN/m^2),

$$p_f = 20(I) \text{ (20 lb/ft}^2\text{ times Importance factor)}$$

7.3.1 Exposure Factor, C_e. The value for C_e shall be determined from Table 7-2.

7.3.2 Thermal Factor, C_t. The value for C_t shall be determined from Table 7-3.

7.3.3 Importance Factor, I. The value for I shall be determined from Table 7-4.

7.3.4 Minimum Values of p_f for Low-Slope Roofs. Minimum values of p_f shall apply to monoslope roofs with slopes less than 15°, hip and gable roofs with slopes less than the larger of 2.38° (1/2 on 12) and $(70/W) + 0.5$ with W in ft (in SI: $21.3/W + 0.5$, with W in m), and curved roofs where the vertical angle from the eaves to the crown is less than 10°.

7.4 SLOPED ROOF SNOW LOADS, p_s

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

$$p_s = C_s p_f \tag{7-2}$$

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

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

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

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

7.4.4 Roof Slope Factor for Multiple Folded Plate, Sawtooth, and Barrel Vault Roofs. Multiple folded plate, sawtooth, or barrel vault roofs shall have a $C_s = 1.0$, with no reduction in snow load because of slope (i.e., $p_s = p_f$).

7.4.5 Ice Dams and Icicles Along Eaves. Two types of warm roofs that drain water over their eaves shall be capable of sustaining a uniformly distributed load of $2p_f$ on all overhanging portions: those that are unventilated and have an R-value less than 30 ft^2 h °F/Btu (5.3 °C m^2/W) and those that are ventilated and have an R-value less than 20 ft^2 h °F/Btu (3.5 °C m^2/W). No other loads except dead loads shall be present on the roof when this uniformly distributed load is applied.

7.5 PARTIAL LOADING

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

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

Case 1: Full balanced snow load on either exterior span and half the balanced snow load on all other spans.

Case 2: Half the balanced snow load on either exterior span and full balanced snow load on all other spans.

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

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

Partial load provisions need not be applied to structural members that span perpendicular to the ridgeline in gable roofs with slopes greater than the larger of 2.38° (1/2 on 12) and $70/W + 0.5$ with W in ft (in SI: $21.3/W + 0.5$, with W in m).

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

7.6 UNBALANCED ROOF SNOW LOADS

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

7.6.1 Unbalanced Snow Loads for Hip and Gable Roofs. For hip and gable roofs with a slope exceeding 70° or with a slope less than the larger of $70/W + 0.5$ with W in ft (in SI: $21.3/W + 0.5$, with W in m) and 2.38° (1/2 on 12) unbalanced snow loads are not required to be applied. Roofs with an eave to ridge distance, W, of 20 ft (6.1 m) or less, having simply supported prismatic members spanning from ridge to eave shall be designed to resist an unbalanced uniform snow load on the leeward side equal to Ip_g. For these roofs the windward side shall be unloaded. For all other gable roofs, the unbalanced load shall consist of $0.3p_s$ on the windward side, p_s on the leeward side plus a rectangular surcharge with magnitude $h_d\gamma/\sqrt{S}$ and horizontal extent from the ridge $8\sqrt{S}h_d/3$ where h_d is the drift height from Fig. 7-9 with ℓ_u equal to the eave to ridge distance for the windward portion of the roof, W. Balanced and unbalanced loading diagrams are presented in Fig. 7-5.

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

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

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

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

7.7 DRIFTS ON LOWER ROOFS (AERODYNAMIC SHADE)

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

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

For leeward drifts, the drift height h_d shall be determined directly from Fig. 7-9 using the length of the upper roof. For windward drifts, the drift height shall be determined by substituting the length of the lower roof for l_u in Fig. 7-9 and using three-quarters of h_d as determined from Fig. 7-9 as the drift height. The larger of these two heights shall be used in design. If this height is equal to or less than h_c, the drift width, w, shall equal $4h_d$ and the drift height shall equal h_d. If this height exceeds h_c, the drift width, w, shall equal $4h_d^2/h_c$ and the drift height shall equal h_c. However, the drift width, w, shall not be greater than $8h_c$. If the drift width, w, exceeds the width of the lower roof, the drift shall be truncated at the far edge of the roof, not reduced to zero there. The maximum intensity of the drift surcharge load, p_d, equals $h_d\gamma$ where snow density, γ, is defined in Eq. 7-3:

$$\gamma = 0.13p_g + 14 \text{ but not more than 30 pcf} \tag{7-3}$$

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

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

7.7.2 Adjacent Structures and Terrain Features. The requirements in Section 7.7.1 shall also be used to determine drift loads caused by a higher structure or terrain feature within 20 ft (6.1 m) of a roof. The separation distance, s, between the roof and adjacent structure or terrain feature shall reduce applied drift loads on the lower roof by the factor (20-s)/20 where s is in ft ([6.1-s]/6.1 where s is in m).

7.8 ROOF PROJECTIONS

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

7.9 SLIDING SNOW

The load caused by snow sliding off a sloped roof onto a lower roof shall be determined for slippery upper roofs with slopes greater than 1/4 on 12, and for other (i.e., nonslippery) upper roofs with slopes greater than 2 on 12. The total sliding load per unit length of eave shall be $0.4p_f W$, where W is the horizontal distance from the eave to ridge for the sloped upper roof. The sliding load shall be distributed uniformly on the lower roof over a distance of 15 ft from the upper roof eave. If the width of the lower roof is less than 15 ft, the sliding load shall be reduced proportionally.

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

Sliding loads shall be superimposed on the balanced snow load.

7.10 RAIN-ON-SNOW SURCHARGE LOAD

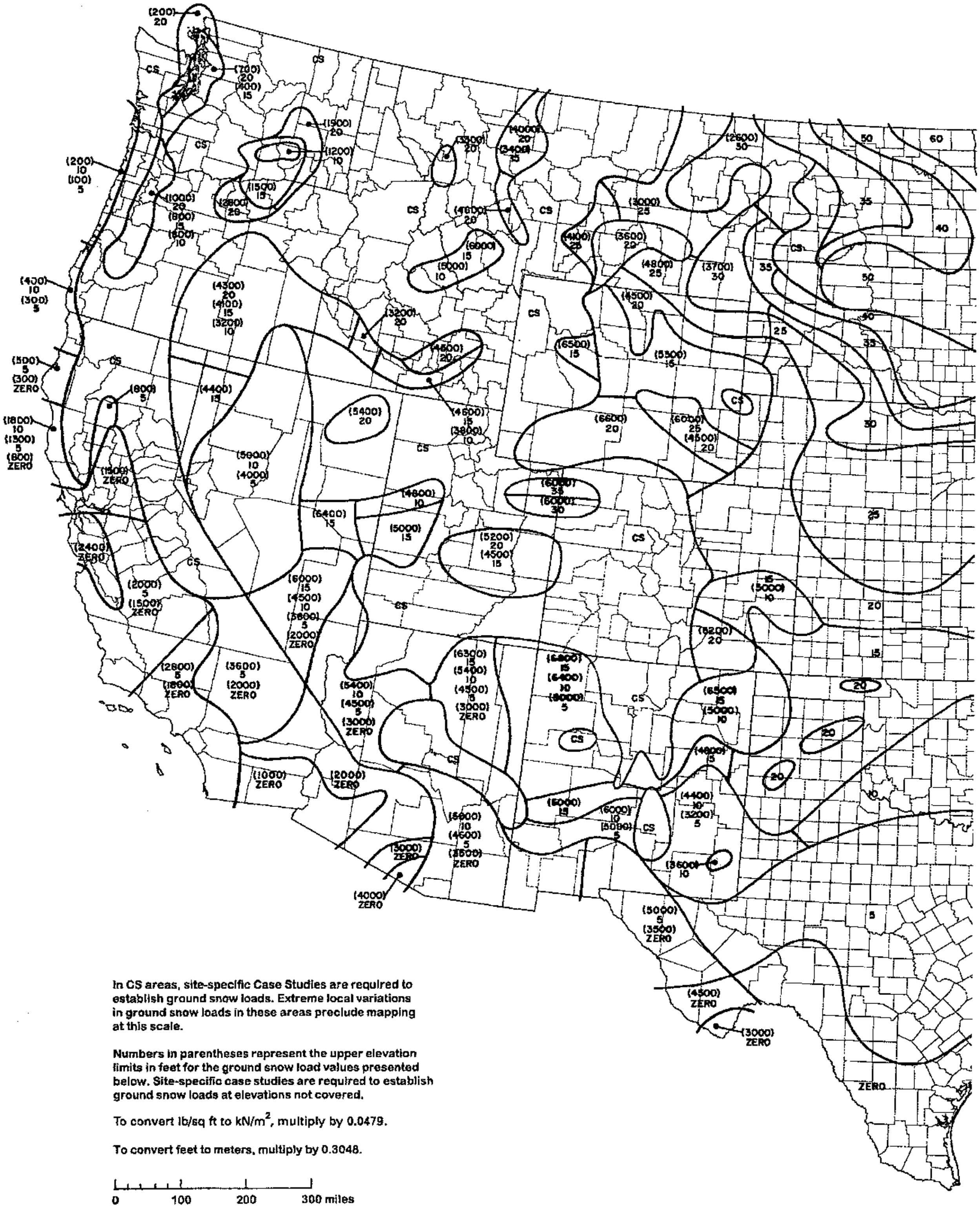
For locations where p_g is 20 lb/ft^2 (0.96 kN/m^2) or less, but not zero, all roofs with slopes (in degrees) less than W/50 with W in ft (in SI: W/15.2 with W in m) shall have a 5 lb/ft^2 (0.24 kN/m^2) rain-on-snow surcharge. This rain-on-snow augmented design load applies only to the balanced load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

7.11 PONDING INSTABILITY

Roofs shall be designed to preclude ponding instability. For roofs with a slope less than 1/4 in./ft (1.19°), roof deflections caused by full snow loads shall be investigated when determining the likelihood of ponding instability from rain-on-snow or from snow meltwater (see Section 8.4).

7.12 EXISTING ROOFS

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



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.

FIGURE 7-1 GROUND SNOW LOADS, p_g, FOR THE UNITED STATES (LB/FT^2)

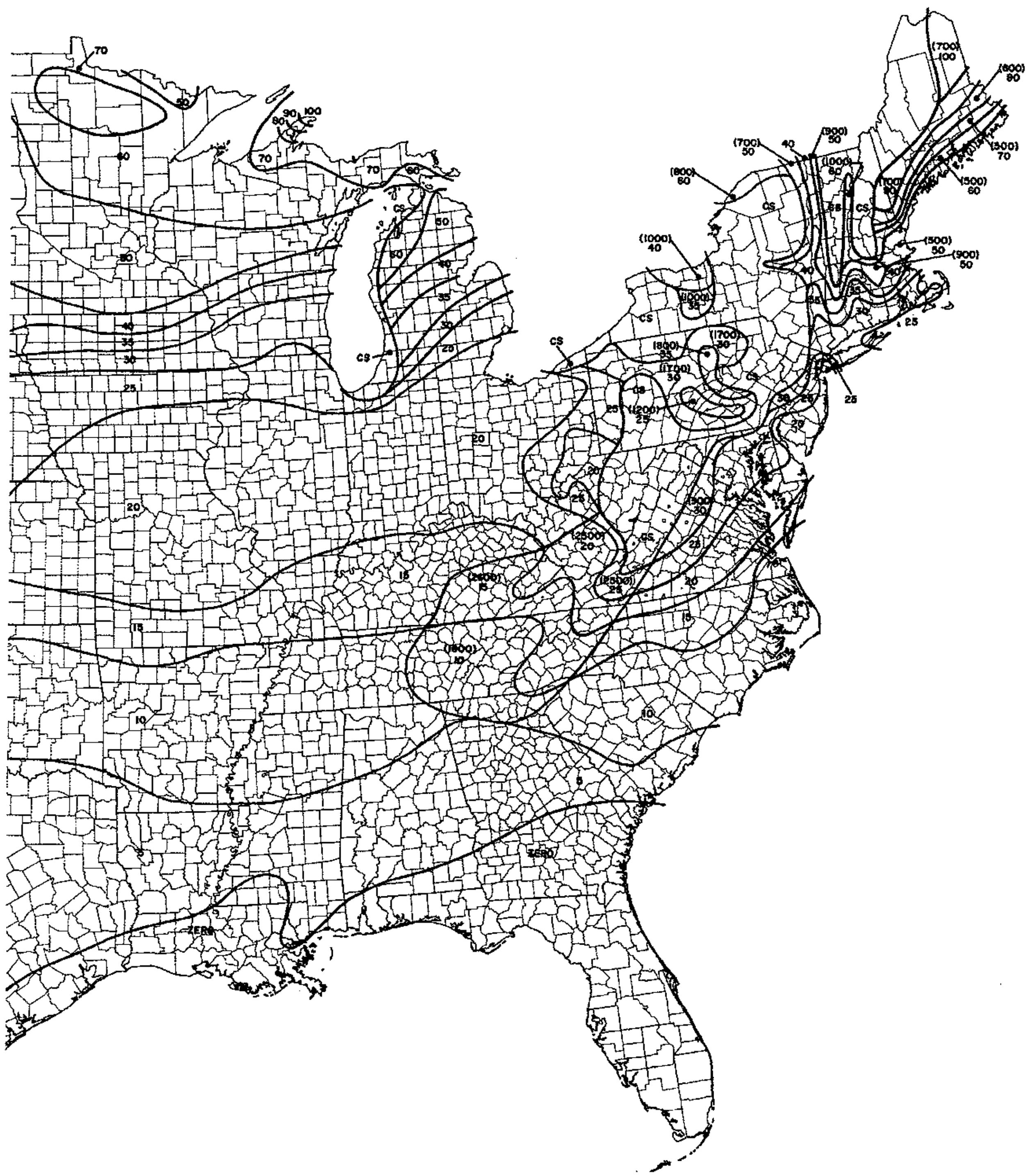
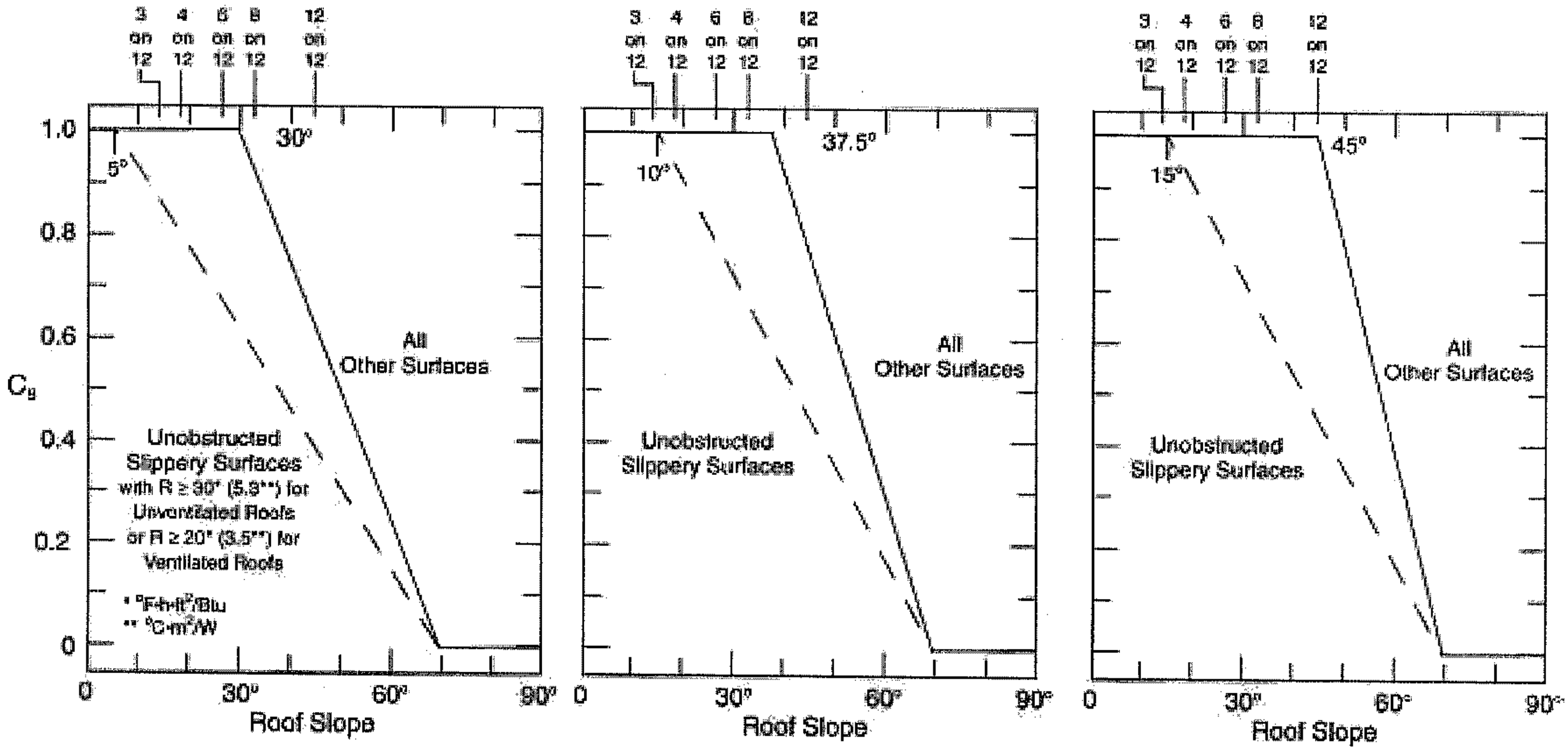


FIGURE 7-1 (continued) GROUND SNOW LOADS, p_g, FOR THE UNITED STATES (LB/FT^2)



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

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

7-2c: Cold roofs with $C_t = 1.2$

FIGURE 7-2 GRAPHS FOR DETERMINING ROOF SLOPE FACTOR C_s, FOR WARM AND COLD ROOFS (SEE TABLE 7-3 FOR C_t DEFINITIONS)

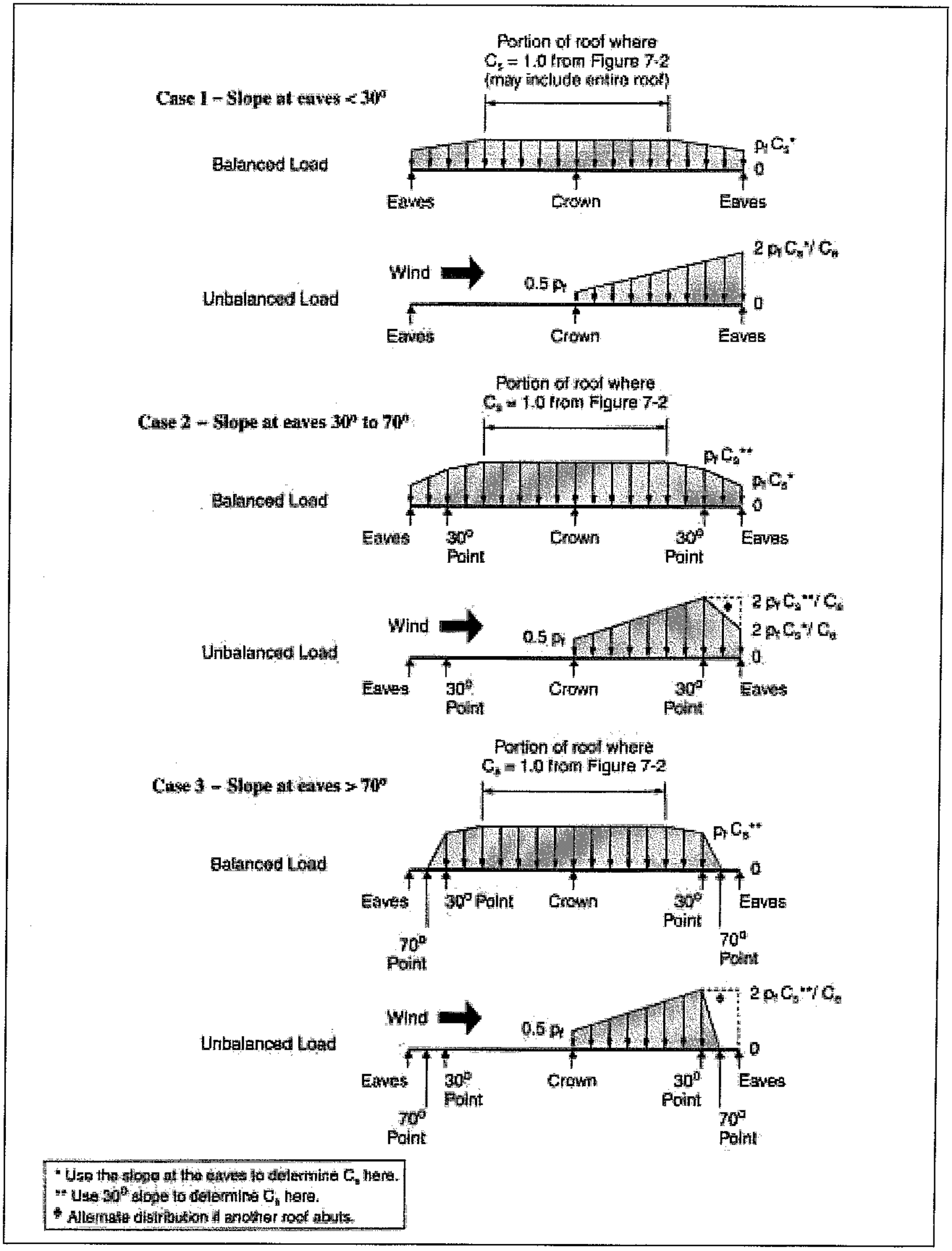
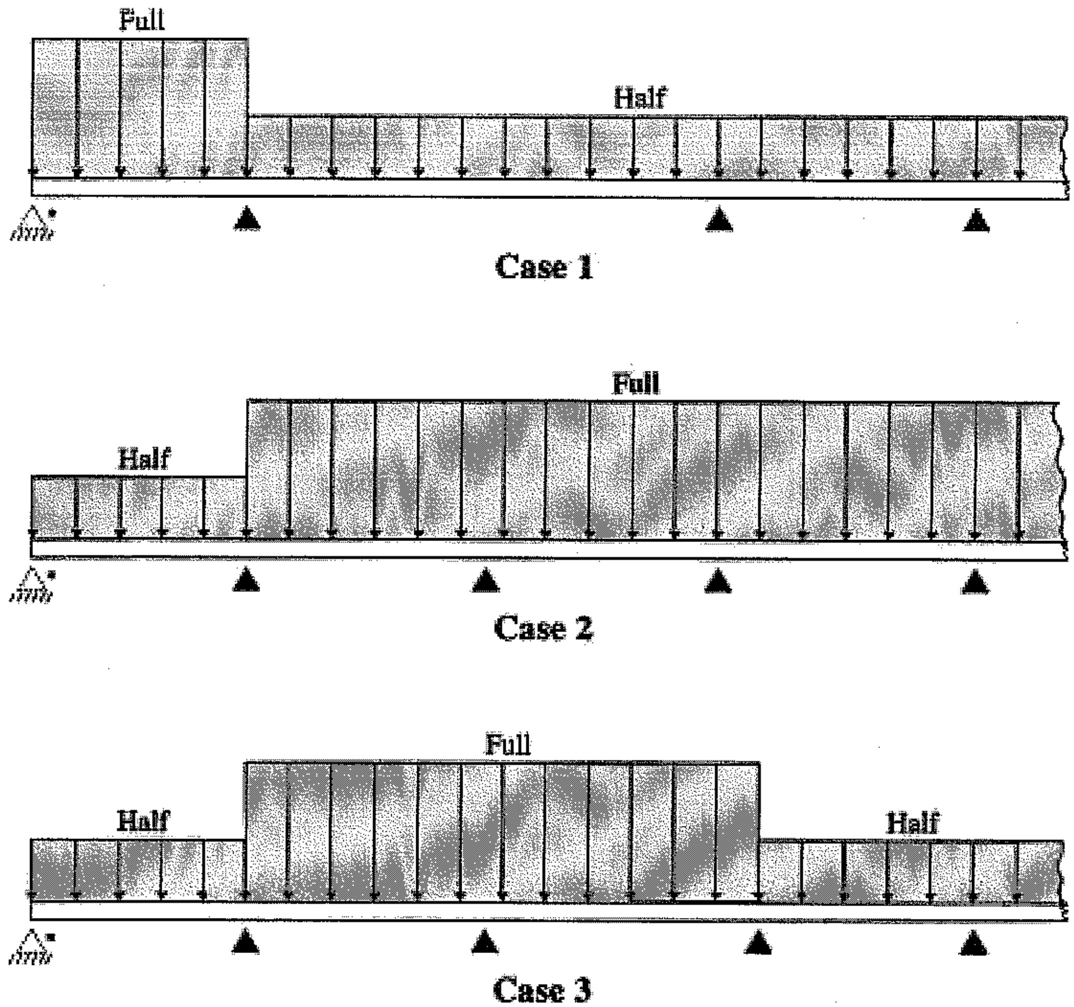


FIGURE 7-3 BALANCED AND UNBALANCED LOADS FOR CURVED ROOFS



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

FIGURE 7-4 PARTIAL LOADING DIAGRAMS FOR CONTINUOUS BEAMS

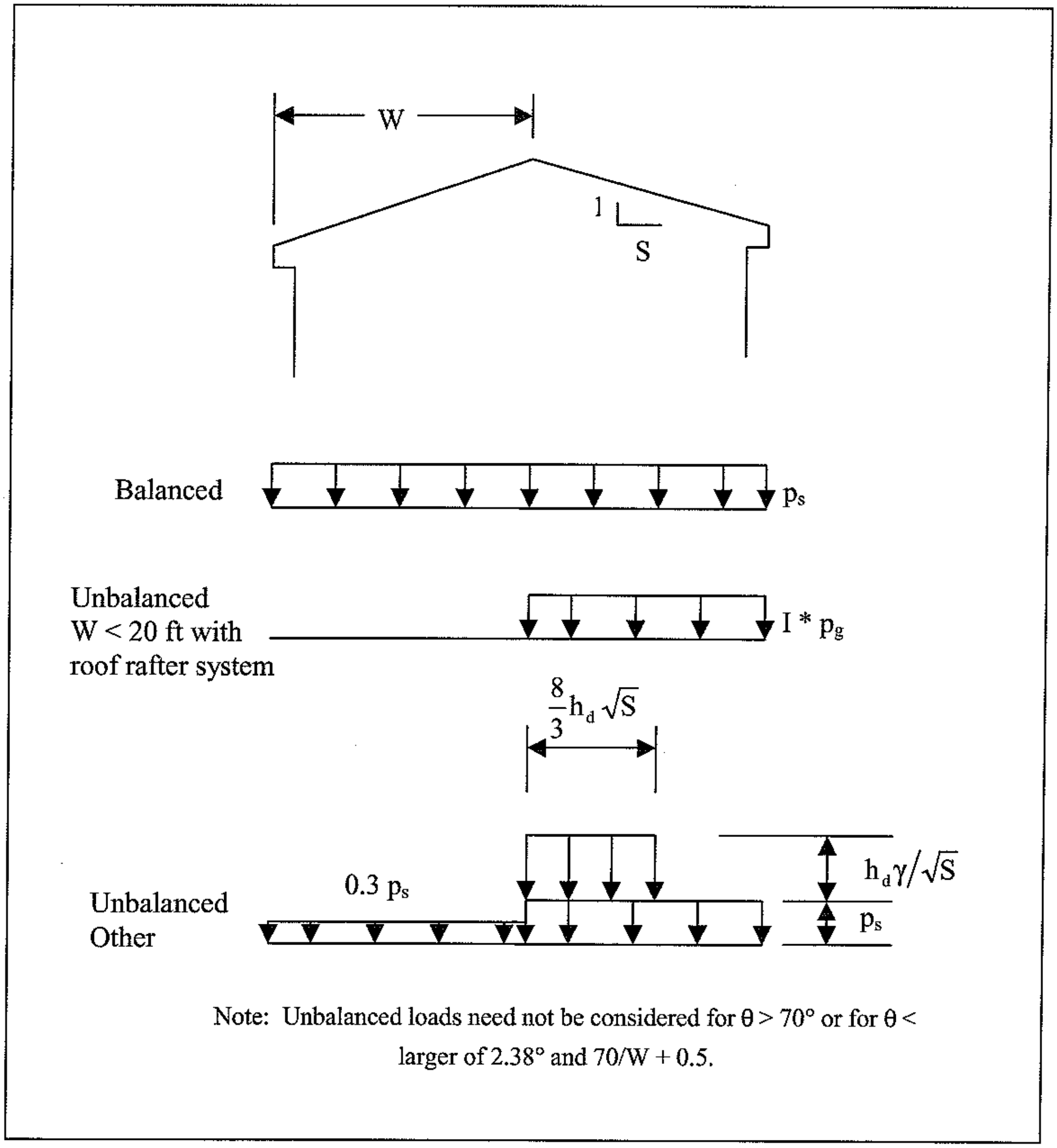


FIGURE 7-5 BALANCED AND UNBALANCED SNOW LOADS FOR HIP AND GABLE ROOFS

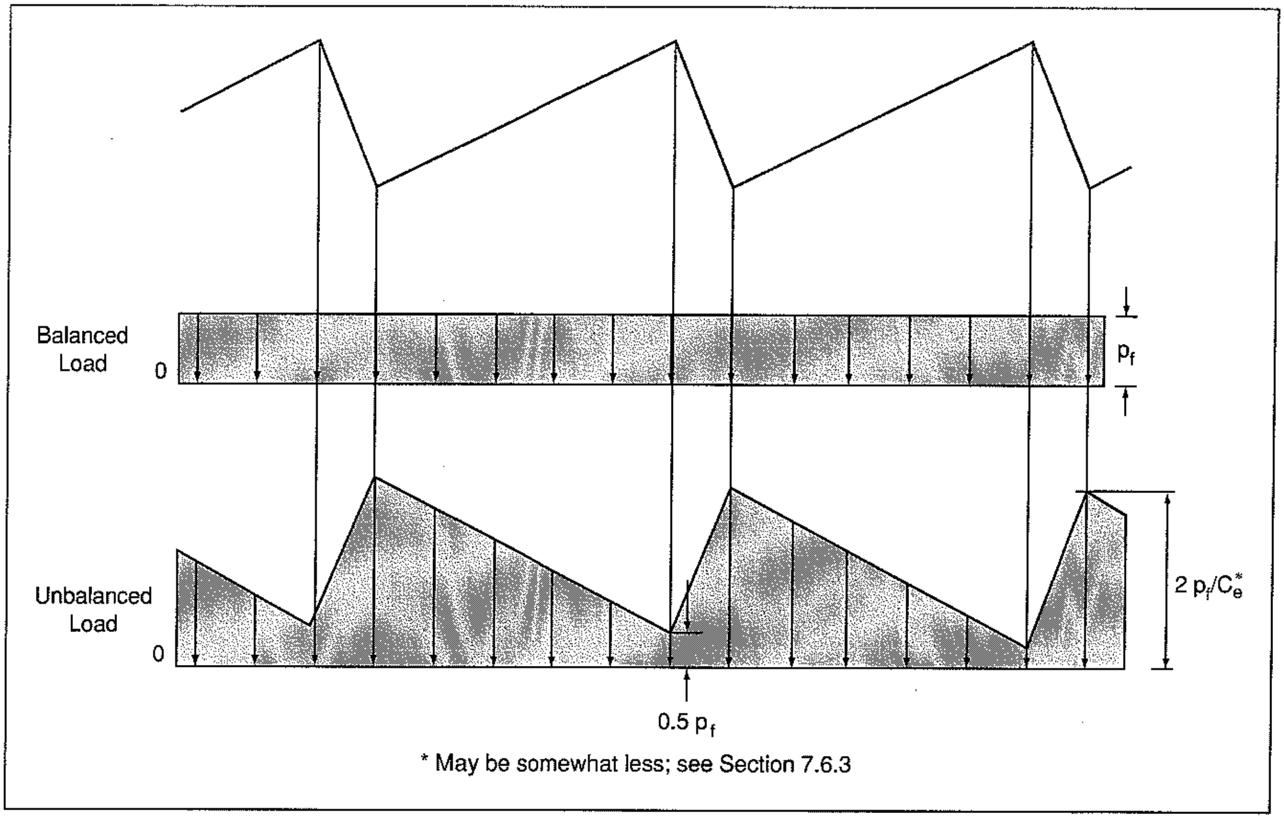


FIGURE 7-6 BALANCED AND UNBALANCED SNOW LOADS FOR A SAWTOOTH ROOF

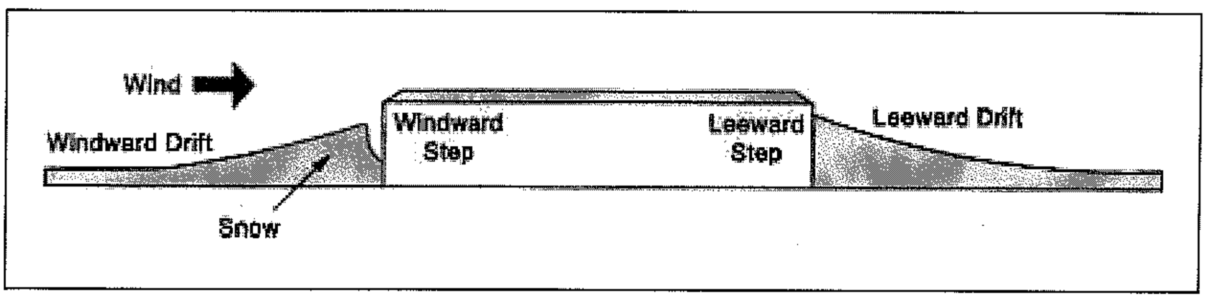


FIGURE 7-7 DRIFTS FORMED AT WINDWARD AND LEEWARD STEPS

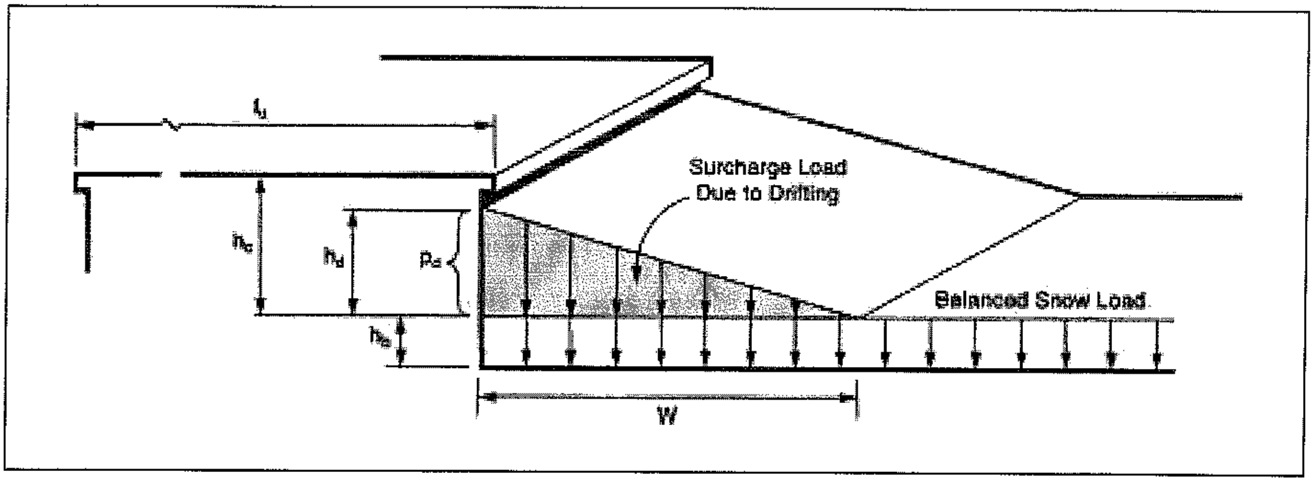
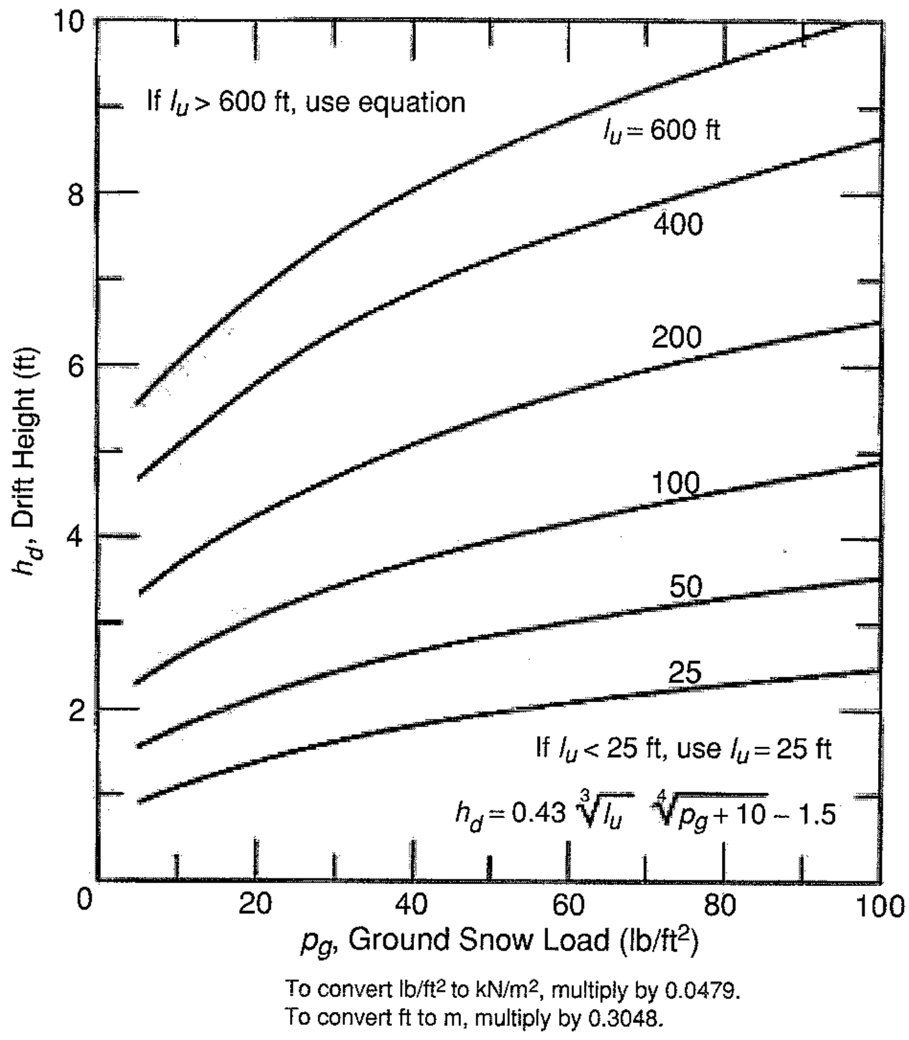


FIGURE 7-8 CONFIGURATION OF SNOW DRIFTS ON LOWER ROOFS



To convert lb/ft^2 to kN/m^2, multiply by 0.0479.
To convert ft to m, multiply by 0.3048.

FIGURE 7-9 GRAPH AND EQUATION FOR DETERMINING DRIFT HEIGHT, h_d

TABLE 7-1 GROUND SNOW LOADS, p_g, FOR ALASKAN LOCATIONS

Location	p_g lb/ft²	p_g (kN/m²)	Location	p_g lb/ft²	p_g (kN/m²)	Location	p_g lb/ft²	p_g (kN/m²)
Adak	30	(1.4)	Galena	60	(2.9)	Petersburg	150	(7.2)
Anchorage	50	(2.4)	Gulkana	70	(3.4)	St Paul	40	(1.9)
Angoon	70	(3.4)	Homer	40	(1.9)	Seward	50	(2.4)
Barrow	25	(1.2)	Juneau	60	(2.9)	Shemya	25	(1.2)
Barter	35	(1.7)	Kenai	70	(3.4)	Sitka	50	(2.4)
Bethel	40	(1.9)	Kodiak	30	(1.4)	Talkeetna	120	(5.8)
Big Delta	50	(2.4)	Kotzebue	60	(2.9)	Unalakleet	50	(2.4)
Cold Bay	25	(1.2)	McGrath	70	(3.4)	Valdez	160	(7.7)
Cordova	100	(4.8)	Nenana	80	(3.8)	Whittier	300	(14.4)
Fairbanks	60	(2.9)	Nome	70	(3.4)	Wrangell	60	(2.9)
Fort Yukon	60	(2.9)	Palmer	50	(2.4)	Yakutat	150	(7.2)

TABLE 7-2 EXPOSURE FACTOR, C_e

Terrain Category	Fully Exposed	Exposure of Roof[a] Partially Exposed	Sheltered
B (see Section 6.5.6)	0.9	1.0	1.2
C (see Section 6.5.6)	0.9	1.0	1.1
D (see Section 6.5.6)	0.8	0.9	1.0
Above the treeline in windswept mountainous areas.	0.7	0.8	N/A
In Alaska, in areas where trees do not exist within a 2-mile (3 km) radius of the site.	0.7	0.8	N/A

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

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

[b]Obstructions within a distance of $10h_o$ provide "shelter," where h_o is the height of the obstruction above the roof level. If the only obstructions are a few deciduous trees that are leafless in winter, the "fully exposed" category shall be used. Note that these are heights above the roof. Heights used to establish the terrain category in Section 6.5.3 are heights above the ground.

TABLE 7-3 THERMAL FACTOR, C_t

Thermal Condition[a]	C_t
All structures except as indicated below:	1.0
Structures kept just above freezing and others with cold, ventilated roofs in which the thermal resistance (R-value) between the ventilated space and the heated space exceeds 25 °F× *h*× ft²/Btu (4.4 K × m²/W).	1.1
Unheated structures and structures intentionally kept below freezing.	1.2
Continuously heated greenhouses[b] with a roof having a thermal resistance (R-value) less than 2.0 °F× *h*× ft²/Btu (0.4 K × m²/W)	0.85

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

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

TABLE 7-4 IMPORTANCE FACTOR, I (SNOW LOADS)

Category[a]	I
I	0.8
II	1.0
III	1.1
IV	1.2

[a]See Section 1.5 and Table 1-1.

Chapter 12
SEISMIC DESIGN REQUIREMENTS FOR BUILDING STRUCTURES

12.1 STRUCTURAL DESIGN BASIS

12.1.1 Basic Requirements. The seismic analysis and design procedures to be used in the design of building structures and their components shall be as prescribed in this section. The building structure shall include complete lateral and vertical force-resisting systems capable of providing adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand. The design ground motions shall be assumed to occur along any horizontal direction of a building structure. The adequacy of the structural systems shall be demonstrated through the construction of a mathematical model and evaluation of this model for the effects of design ground motions. The design seismic forces, and their distribution over the height of the building structure, shall be established in accordance with one of the applicable procedures indicated in Section 12.6 and the corresponding internal forces and deformations in the members of the structure shall be determined. An approved alternative procedure shall not be used to establish the seismic forces and their distribution unless the corresponding internal forces and deformations in the members are determined using a model consistent with the procedure adopted.

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

12.1.2 Member Design, Connection Design, and Deformation Limit. Individual members, including those not part of the seismic force–resisting system, shall be provided with adequate strength to resist the shears, axial forces, and moments determined in accordance with this standard, and connections shall develop the strength of the connected members or the forces indicated in Section 12.1.1. The deformation of the structure shall not exceed the prescribed limits where the structure is subjected to the design seismic forces.

12.1.3 Continuous Load Path and Interconnection. A continuous load path, or paths, with adequate strength and stiffness shall be provided to transfer all forces from the point of application to the final point of resistance. All parts of the structure between separation joints shall be interconnected to form a continuous path to the seismic force–resisting system, and the connections shall be capable of transmitting the seismic force (F_p) induced by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure with elements having a design strength capable of transmitting a seismic force of 0.133 times the short period design spectral response acceleration parameter, S_{DS}, times the weight of the smaller portion or 5 percent of the portion's weight, whichever is greater. This connection force does not apply to the overall design of the seismic force–resisting system. Connection design forces need not exceed the maximum forces that the structural system can deliver to the connection.

12.1.4 Connection to Supports. A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder, or truss either directly to its supporting elements, or to slabs designed to act as diaphragms. Where the connection is through a diaphragm, then the member's supporting element must also be connected to the diaphragm. The connection shall have a minimum design strength of 5 percent of the dead plus live load reaction.

12.1.5 Foundation Design. The foundation shall be designed to resist the forces developed and accommodate the movements imparted to the structure by the design ground motions. The dynamic nature of the forces, the expected ground motion, the design basis for strength and energy dissipation capacity of the structure, and the dynamic properties of the soil shall be included in the determination of the foundation design criteria. The design and construction of foundations shall comply with Section 12.13.

12.1.6 Material Design and Detailing Requirements. Structural elements including foundation elements shall conform to the material design and detailing requirements set forth in Chapter 14.

12.2 STRUCTURAL SYSTEM SELECTION

12.2.1 Selection and Limitations. The basic lateral and vertical seismic force–resisting system shall conform to one of the types indicated in Table 12.2-1 or a combination of systems as permitted in Sections 12.2.2, 12.2.3, and 12.2.4. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. The structural system used shall be in accordance with the seismic design category and height limitations indicated in Table 12.2-1. The appropriate response modification coefficient, R, system overstrength factor, Ω_0, and the deflection amplification factor, C_d, indicated in Table 12.2-1 shall be used in determining the base shear, element design forces, and design story drift.

The selected seismic force-resisting system shall be designed and detailed in accordance with the specific requirements for the system per the applicable reference document and the additional requirements set forth in Chapter 14.

Seismic force–resisting systems that are not contained in Table 12.2-1 are permitted if analytical and test data are submitted that establish the dynamic characteristics and demonstrate the lateral force resistance and energy dissipation capacity to be equivalent to the structural systems listed in Table 12.2-1 for equivalent response modification coefficient, R, system overstrength coefficient, Ω_0, and deflection amplification factor, C_d, values.

12.2.2 Combinations of Framing Systems in Different Directions. Different seismic force–resisting systems are permitted to be used to resist seismic forces along each of the two orthogonal axes of the structure. Where different systems are used, the respective R, C_d, and Ω_0 coefficients shall apply to each system, including the limitations on system use contained in Table 12.2-1.

12.2.3 Combinations of Framing Systems in the Same Direction. Where different seismic force–resisting systems are used in combination to resist seismic forces in the same direction of structural response, other than those combinations considered as

Alternatively, it is permitted to determine the approximate fundamental period (T_a), in s, from the following equation for structures not exceeding 12 stories in height in which the seismic force–resisting system consists entirely of concrete or steel moment resisting frames and the story height is at least 10 ft (3 m):

$$T_a = 0.1\,N \tag{12.8-8}$$

where N = number of stories.

The approximate fundamental period, T_a, in s for masonry or concrete shear wall structures is permitted to be determined from Eq. 12.8-9 as follows:

$$T_a = \frac{0.0019}{\sqrt{C_w}} h_n \tag{12.8-9}$$

where h_n is as defined in the preceding text and C_w is calculated from Eq. 12.8-10 as follows:

$$C_W = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]} \tag{12.8-10}$$

where

A_B = area of base of structure, ft^2
A_i = web area of shear wall "i" in ft^2
D_i = length of shear wall "i" in ft
h_i = height of shear wall "i" in ft
x = number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

12.8.3 Vertical Distribution of Seismic Forces. The lateral seismic force (F_x) (kip or kN) induced at any level shall be determined from the following equations:

$$F_x = C_{vx} V \tag{12.8-11}$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{12.8-12}$$

where

C_{vx} = vertical distribution factor,
V = total design lateral force or shear at the base of the structure (kip or kN)
w_i and w_x = the portion of the total effective seismic weight of the structure (W) located or assigned to Level i or x
h_i and h_x = the height (ft or m) from the base to Level i or x
k = an exponent related to the structure period as follows:
for structures having a period of 0.5 s or less, $k = 1$
for structures having a period of 2.5 s or more, $k = 2$
for structures having a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2

12.8.4 Horizontal Distribution of Forces. The seismic design story shear in any story (V_x) (kip or kN) shall be determined from the following equation:

$$V_x = \sum_{i=x}^{n} F_i \tag{12.8-13}$$

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

The seismic design story shear (V_x) (kip or kN) shall be distributed to the various vertical elements of the seismic force–resisting system in the story under consideration based on the relative lateral stiffness of the vertical resisting elements and the diaphragm.

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

12.8.4.2 Accidental Torsion. Where diaphragms are not flexible, the design shall include the inherent torsional moment (M_t) (kip or kN) resulting from the location of the structure masses plus the accidental torsional moments (M_{ta}) (kip or kN) caused by assumed displacement of the center of mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied forces.

Where earthquake forces are applied concurrently in two orthogonal directions, the required 5 percent displacement of the center of mass need not be applied in both of the orthogonal directions at the same time, but shall be applied in the direction that produces the greater effect.

12.8.4.3 Amplification of Accidental Torsional Moment. Structures assigned to Seismic Design Category C, D, E, or F, where Type 1a or 1b torsional irregularity exists as defined in Table 12.3-1 shall have the effects accounted for by multiplying M_{ta} at each level by a torsional amplification factor (A_x) as illustrated in Fig. 12.8-1 and determined from the following equation:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \tag{12.8-14}$$

where

δ_{max} = the maximum displacement at Level x (in. or mm) computed assuming $A_x = 1$
δ_{avg} = the average of the displacements at the extreme points of the structure at Level x computed assuming $A_x = 1$ (in. or mm)

EXCEPTION: The accidental torsional moment need not be amplified for structures of light-frame construction.

The torsional amplification factor (A_x) is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

12.8.5 Overturning. The structure shall be designed to resist overturning effects caused by the seismic forces determined in Section 12.8.3.

12.8.6 Story Drift Determination. The design story drift (Δ) shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration. See Fig. 12.8-2. Where allowable stress design is used, Δ shall be computed using the strength level seismic forces specified in Section 12.8 without reduction for allowable stress design.

12.7.4 Interaction Effects. Moment-resisting frames that are enclosed or adjoined by elements that are more rigid and not considered to be part of the seismic force–resisting system shall be designed so that the action or failure of those elements will not impair the vertical load and seismic force–resisting capability of the frame. The design shall provide for the effect of these rigid elements on the structural system at structural deformations corresponding to the design story drift (Δ) as determined in Section 12.8.6. In addition, the effects of these elements shall be considered where determining whether a structure has one or more of the irregularities defined in Section 12.3.2.

12.8 EQUIVALENT LATERAL FORCE PROCEDURE

12.8.1 Seismic Base Shear. The seismic base shear, V, in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \tag{12.8-1}$$

where

C_s = the seismic response coefficient determined in accordance with Section 12.8.1.1
W = the effective seismic weight per Section 12.7.2.

12.8.1.1 Calculation of Seismic Response Coefficient. The seismic response coefficient, C_s, shall be determined in accordance with Eq. 12.8-2.

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \tag{12.8-2}$$

where

S_{DS} = the design spectral response acceleration parameter in the short period range as determined from Section 11.4.4
R = the response modification factor in Table 12.2-1
I = the occupancy importance factor determined in accordance with Section 11.5.1

The value of C_s computed in accordance with Eq. 12.8-2 need not exceed the following:

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \quad \text{for } T \leq T_L \tag{12.8-3}$$

$$C_s = \frac{S_{D1} T_L}{T^2\left(\frac{R}{I}\right)} \quad \text{for } T > T_L \tag{12.8-4}$$

C_s shall not be less than

$$C_s = 0.01 \tag{12.8-5}$$

In addition, for structures located where S_1 is equal to or greater than $0.6g$, C_s shall not be less than

$$C_s = \frac{0.5 S_1}{\left(\frac{R}{I}\right)} \tag{12.8-6}$$

TABLE 12.8-1 COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD

Design Spectral Response Acceleration Parameter at 1 s, S_{D1}	Coefficient C_u
≥ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤ 0.1	1.7

where I and R are as defined in Section 12.8.1.1 and

S_{D1} = the design spectral response acceleration parameter at a period of 1.0 s, as determined from Section 11.4.4
T = the fundamental period of the structure (s) determined in Section 12.8.2
T_L = long-period transition period (s) determined in Section 11.4.5
S_1 = the mapped maximum considered earthquake spectral response acceleration parameter determined in accordance with Section 11.4.1

12.8.1.2 Soil Structure Interaction Reduction. A soil structure interaction reduction is permitted where determined using Chapter 19 or other generally accepted procedures approved by the authority having jurisdiction.

12.8.1.3 Maximum S_s Value in Determination of C_s. For regular structures five stories or less in height and having a period, T, of 0.5 s or less, C_s is permitted to be calculated using a value of 1.5 for S_S.

12.8.2 Period Determination. The fundamental period of the structure, T, in the direction under consideration shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The fundamental period, T, shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 12.8-1 and the approximate fundamental period, T_a, determined from Eq. 12.8-7. As an alternative to performing an analysis to determine the fundamental period, T, it is permitted to use the approximate building period, T_a, calculated in accordance with Section 12.8.2.1, directly.

12.8.2.1 Approximate Fundamental Period. The approximate fundamental period (T_a), in s, shall be determined from the following equation:

$$T_a = C_t h_n^x \tag{12.8-7}$$

where h_n is the height in ft above the base to the highest level of the structure and the coefficients C_t and x are determined from Table 12.8-2.

TABLE 12.8-2 VALUES OF APPROXIMATE PERIOD PARAMETERS C_t AND x

Structure Type	C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel moment-resisting frames	0.028 (0.0724)[a]	0.8
Concrete moment-resisting frames	0.016 (0.0466)[a]	0.9
Eccentrically braced steel frames	0.03 (0.0731)[a]	0.75
All other structural systems	0.02 (0.0488)[a]	0.75

[a] Metric equivalents are shown in parentheses.