CHAPTER 16
STRUCTURAL DESIGN

SECTION 1601 GENERAL

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

SECTION 1602 DEFINITIONS

1602.1 Definitions. The following words and terms shall, for the purposes of this chapter, have the meanings shown herein.

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

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

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

BOUNDARY MEMBERS. Portions along wall and diaphragm edges strengthened by longitudinal and transverse reinforcement and/or structural steel members.

CANTILEVERED COLUMN SYSTEM. A structural system relying on column elements that cantilever from a fixed base and have minimal rotational resistance capacity at the top with lateral forces applied essentially at the top and are used for lateral resistance.

COLLECTOR ELEMENTS. Members that serve to transfer forces between floor diaphragms and members of the lateral-force-resisting system.

CONFINED REGION. The portion of a reinforced concrete component in which the concrete is confined by closely spaced special transverse reinforcement restraining the concrete in directions perpendicular to the applied stress.

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

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

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

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

High deformability element. An element whose deformability is not less than 3.5 when subjected to four fully reversed cycles at the limit deformation.

Limited deformability element. An element that is neither a low deformability or a high deformability element.

Low deformability element. An element whose deformability is 1.5 or less.

DEFORMATION

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

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

DESIGN STRENGTH. The product of the nominal strength and a resistance factor (or strength reduction factor).

DIAPHRAGM, FLEXIBLE. A diaphragm is flexible for the purpose of distribution of story shear and torsional moment when the lateral deformation of the diaphragm is more than two times the average story drift of the associated story, determined by comparing the computed maximum in-plane deflection of the diaphragm itself under lateral load with the story drift of adjoining vertical-resisting elements under equivalent tributary lateral load.

DIAPHRAGM, RIGID. A diaphragm that does not conform to the definition of flexible diaphragm.

DURATION OF LOAD. The period of continuous application of a given load, or the aggregate of periods of intermittent applications of the same load.

ELEMENT

Ductile element. An element capable of sustaining large cyclic deformations beyond the attainment of its nominal strength without any significant loss of strength.

Limited ductile element. An element that is capable of sustaining moderate cyclic deformations beyond the attainment of nominal strength without significant loss of strength.

Nonductile element. An element having a mode of failure that results in an abrupt loss of resistance when the element is deformed beyond the deformation corresponding to the development of its nominal strength. Nonductile elements cannot reliably sustain significant deformation beyond that attained at their nominal strength.

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

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

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

FLEXIBLE EQUIPMENT CONNECTIONS. Those connections between equipment components that permit rotational and/or translational movement without degradation of performance.

FRAME

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

Concentrically braced frame (CBF). A braced frame in which the members are subjected primarily to axial forces.

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

Ordinary concentrically braced frame (OCBF). A steel concentrically braced frame in which members and connections are designed in accordance with the provisions of AISC Seismic without modification.

Special concentrically braced frame (SCBF). A steel or composite steel and concrete concentrically braced frame in which members and connections are designed for ductile behavior.

FRAME, MOMENT

Intermediate moment frame (IMF). A moment frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members.

Ordinary moment frame (OMF). A moment frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members.

Special moment frame (SMF). A moment frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members.

FRAME SYSTEM

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

Dual frame system. A structural system with an essentially complete space frame system providing support for vertical loads. Seismic force resistance is provided by a moment-resisting frame and shear walls or braced frames.

Space frame system. A structural system composed of interconnected members, other than bearing walls, that is capable of supporting vertical loads and that also may provide resistance to seismic forces.

GUARD. See Section 1002.1.

IMPACT LOAD. The load resulting from moving machinery, elevators, craneways, vehicles, and other similar forces and kinetic loads, pressure and possible surcharge from fixed or moving loads.

JOINT. A portion of a column bounded by the highest and lowest surfaces of the other members framing into it.

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

LIVE LOADS. Those loads produced by the use and occupancy of the building or other structure and do not include construction or environmental loads such as wind load, snow load, rain load, earthquake load, flood load or dead load.

LIVE LOADS (ROOF). Those loads produced (1) during maintenance by workers, equipment and materials; and (2) during the life of the structure by movable objects such as planters and by people.

LOAD AND RESISTANCE FACTOR DESIGN (LRFD). A method of proportioning structural members and their connections using load and resistance factors such that no applicable limit state is reached when the structure is subjected to appropriate load combinations. The term "LRFD" is used in the design of steel and wood structures.

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

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

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

NOMINAL LOADS. The magnitudes of the loads specified in this chapter (dead, live, soil, wind, snow, rain, flood and earthquake).

NOTATIONS

D = Dead load.

E = Combined effect of horizontal and vertical earthquake-induced forces as defined in Sections 1616.4.1 and 1617.1.1.

E_m = Maximum seismic load effect of horizontal and vertical seismic forces as set forth in Sections 1616.4.1 and 1617.1.1.

F = Load due to fluids.

F_a = Flood load.

H = Load due to lateral pressure of soil and water in soil.

L = Live load, except roof live load, including any permitted live load reduction.

L_r = Roof live load including any permitted live load reduction.

P = Ponding load.

R = Rain load.

S = Snow load.

T = Self-straining force arising from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep in component materials, movement due to differential settlement, or combinations thereof.

W = Load due to wind pressure.

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

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

PANEL (PART OF A STRUCTURE). The section of a floor, wall or roof comprised between the supporting frame of two adjacent rows of columns and girders or column bands of floor or roof construction.

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

SHALLOW ANCHORS. Shallow anchors are those with embedment length-to-diameter ratios of less than 8.

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

SHEAR WALL. A wall designed to resist lateral forces parallel to the plane of the wall.

SPECIAL TRANSVERSE REINFORCEMENT. Reinforcement composed of spirals, closed stirrups, or hoops and supplementary cross-ties provided to restrain the concrete and qualify the portion of the component, where used, as a confined region.

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

STRENGTH, REQUIRED. Strength of a member, cross section or connection required to resist factored loads or related internal moments and forces in such combinations as stipulated by these provisions.

STRENGTH DESIGN. A method of proportioning structural members such that the computed forces produced in the members by factored loads do not exceed the member design strength (also called load and resistance factor design.) The term "strength design" is used in the design of concrete and masonry structural elements.

WALL, LOAD BEARING. Any wall meeting either of the following classifications:

1. Any metal or wood stud wall that supports more than 100 pounds per linear foot (1459 N/m) of vertical load in addition to its own weight.
2. Any masonry or concrete wall that supports more than 200 pounds per linear foot (2919 N/m) of vertical load in addition to its own weight.

WALL, NONLOAD BEARING. Any wall that is not a load-bearing wall.

SECTION 1603 CONSTRUCTION DOCUMENTS

1603.1 General. Construction documents shall show the size, section and relative locations of structural members with floor levels, column centers and offsets fully dimensioned. The design loads and other information pertinent to the structural design required by Sections 1603.1.1 through 1603.1.8 shall be clearly indicated on the construction documents for parts of the building or structure.

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

1. Floor and roof live loads.
2. Ground snow load, P_g.
3. Basic wind speed (3-second gust), miles per hour (km/hr) and wind exposure.
4. Seismic Design Category and Site Class.

1603.1.1 Floor live load. The uniformly distributed, concentrated and impact floor live load used in the design shall be indicated for floor areas. Live load reduction of the uniformly distributed floor live loads, if used in the design, shall be indicated.

1603.1.2 Roof live load. The roof live load used in the design shall be indicated for roof areas (Section 1607.11).

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

1. Flat-roof snow load, P_f.
2. Snow exposure factor, C_e.
3. Snow load importance factor, I.
4. Thermal factor, C_t.

1603.1.4 Wind load. The following information related to wind loads shall be shown, regardless of whether wind loads govern the lateral design of the building:

1. Basic wind speed (3-second gust), miles per hour (km/hr).
2. Wind importance factor, I, and building category.
3. Wind exposure, if more than one wind exposure is utilized, the wind exposure and applicable wind direction shall be indicated.
4. The applicable internal pressure coefficient.

5. Components and cladding. The design wind pressures in terms of pounds per square foot (kN/m^2) to be used for the design of exterior component and cladding materials not specifically designed by the registered design professional.

1603.1.5 Earthquake design data. The following information related to seismic loads shall be shown, regardless of whether seismic loads govern the lateral design of the building:

1. Seismic use group.
2. Spectral response coefficients S_{DS} and S_{D1}.
3. Site class.
4. Basic seismic-force-resisting system.
5. Design base shear.
6. Analysis procedure.

1603.1.6 Flood load. For buildings located in flood-hazard areas as established in Section 1612.3, the following information, referenced to the datum on the community's flood insurance rate map (FIRM), shall be shown, regardless of whether flood loads govern the design of the building:

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

1603.1.7 Special loads. Special loads that are applicable to the design of the building, structure or portions thereof shall be indicated along with the specified section of this code that addresses the special loading condition.

1603.1.8 System and components requiring special inspections for seismic resistance. Construction documents or specifications shall be prepared for those systems and components requiring special inspection for seismic resistance as specified in Section 1707.1 by the registered design professional responsible for their design and shall be submitted for approval in accordance with Section 106.1. Reference to seismic standards in lieu of detailed drawings is acceptable.

1603.2 Restrictions on loading. It shall be unlawful to place, or cause or permit to be placed, on any floor or roof of a building, structure, or portion thereof, a load greater than is permitted by these requirements.

1603.3 [Comm 62.1603 (1)] Live loads posted. Where the live loads for which each floor or portion thereof of a commercial or industrial building is or has been designed to exceed 100 pounds per square foot (4.79 kN/m^2), such design live loads shall be conspicuously posted by the owner in that part of each story in which they apply, using durable signs. It shall be unlawful to remove or deface such notices.

1603.4 Deleted.

SECTION 1604 GENERAL DESIGN REQUIREMENTS

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

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

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

1604.3 Serviceability. Structural systems and members thereof shall be designed to have adequate stiffness to limit deflections and lateral drift. See Section 1617.3 for drift limits applicable to earthquake loading.

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

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

1604.3.3 Steel. The deflection of steel structural members shall not exceed that permitted by AISC LRFD, AISC HSS, AISC ASD, AISI, ASCE 3, ASCE 8-SSD-LRFD/ASD, and the standard specifications of SJI Standard Specifications, Load Tables and Weight Tables for Steel Joists and Joist Girders as applicable.

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

1604.3.5 Aluminum. The deflection of aluminum structural members shall not exceed that permitted by AA-94.

1604.3.6 Limits. Deflection of structural members over span, l, shall not exceed that permitted by Table 1604.3.

1604.4 Analysis. Load effects on structural members and their connections shall be determined by methods of structural analysis that take into account equilibrium, general stability, geometric compatibility, and both short- and long-term material properties.

Members that tend to accumulate residual deformations under repeated service loads shall have included in their analysis the added eccentricities expected to occur during their service life.

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

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

Every structure shall be designed to resist the overturning effects caused by the lateral forces specified in this chapter. See Section 1613 for earthquake, Section 1609.1.3 for wind, and Section 1610 for lateral soil loads.

TABLE 1604.3
DEFLECTION LIMITS[a, b, c]

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

For SI: 1 foot = 304.8 mm.

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

b. Interior partitions not exceeding 6 feet in height and flexible, folding and portable partitions are not governed by the provisions of this section. The deflection criterion for interior partitions is based on the horizontal load defined in Section 1607.13.

c. See Section 2403 for glass supports.

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

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

f. The wind load is permitted to be taken as 0.7 times the "component and cladding" loads for the purpose of determining deflection limits herein.

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

1604.5 Importance factors. The value for snow load, wind load and seismic load importance factors shall be determined in accordance with Table 1604.5.

1604.6 [Comm 62.1604] In-situ load tests. The building official is authorized to require an engineering analysis or a load test, or both, of any construction whenever there is reason to question the safety of the construction for the intended occupancy.

1604.7 Preconstruction load tests. Materials and methods of construction that are not capable of being designed by approved engineering analysis or that do not comply with the applicable material design standards listed in Chapter 35, or alternative test procedures in accordance with Section 1704, shall be load tested in accordance with Section 1709.

1604.8 Anchorage.

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

1604.8.2 Concrete and masonry walls. Concrete and masonry walls shall be anchored to floors, roofs and other structural elements that provide lateral support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces specified in this chapter but not less than a minimum horizontal force of 200 pounds per linear foot (2.92 kN/m) of wall, substituted for "*E*." Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet (1219 mm). Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall. See Sections 1609.6.5 and 1620 for wind and earthquake design requirements.

1604.8.3 Decks. Where supported by attachment to an exterior wall, decks shall be positively anchored to the primary structure and designed for both vertical and lateral loads as applicable. Such attachment shall not be accomplished by the use of toenails or nails subject to withdrawal. Where positive connection to the primary building structure cannot be verified during inspection, decks shall be self-supporting. For decks with cantilevered framing members, connections to exterior walls or other framing members shall be designed and constructed to resist uplift resulting from the full live load specified in Table 1607.1 acting on the cantilevered portion of the deck.

SECTION 1605
LOAD COMBINATIONS

1605.1 General. Buildings and other structures and portions thereof shall be designed to resist the load combinations specified in Section 1605.2 or 1605.3 and Chapters 18 through 23, and the special seismic load combinations of Section 1605.4 where required by Section 1620.1.6, 1620.1.9 or 1620.3.4. Applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations. Effects from one or more transient loads not acting shall be investigated.

TABLE 1604.5
CLASSIFICATION OF BUILDINGS AND OTHER STRUCTURES FOR IMPORTANCE FACTORS

CATEGORY[a]	NATURE OF OCCUPANCY	SEISMIC FACTOR I_E	SNOW FACTOR I_S	WIND FACTOR I_W
I	Buildings and other structures except those listed in Categories II, III and IV	1.00	1.0	1.00
II	Buildings and other structures that represent a substantial hazard to human life in the event of failure including, but not limited to: • Buildings and other structures where more than 300 people congregate in one area • Buildings and other structures with elementary school, secondary school or day-care facilities with capacity greater than 250 • Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities • Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities • Jails and detention facilities • Any other occupancy with an occupant load greater that 5,000 • Power-generating stations, water treatment for potable water, wastewater treatment facilities and other public utility facilities not included in Category III • Buildings and other structures not included in Category III containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released	1.25	1.1	1.15
III	Buildings and other structures designated as essential facilities including, but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities • Fire, rescue and police stations and emergency vehicle garages • Designated earthquake, hurricane or other emergency shelters • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response • Power-generating stations and other public utility facilities required as emergency back-up facilities for Category III structures • Structures containing highly toxic materials as defined by Section 307 where the quantity of the material exceeds the exempt amounts of Table 307.7(2) • Aviation control towers, air traffic control centers and emergency aircraft hangars • Buildings and other structures having critical national defense functions • Water treatment facilities required to maintain water pressure for fire suppression	1.50	1.2	1.15
IV	Buildings and other structures that represent a low hazard to human life in the event of failure including, but not limited to: • Agricultural facilities • Certain temporary facilities • Minor storage facilities	1.00	0.8	0.87[b]

a. "Category" is equivalent to "Seismic Use Group" for the purposes of Section 1616.2.
b. In hurricane-prone regions with $V > 100$ miles per hour, I_w shall be 0.77.

1605.2 Load combinations using strength design or load and resistance factor design.

1605.2.1 Basic load combinations. Where strength design or load and resistance factor design is used, structures and portions thereof shall resist the most critical effects from the following combinations of factored loads:

$1.4D$ **(Formula 16-1)**

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

$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (f_1 L \text{ or } 0.8W)$ **(Formula 16-3)**

$1.2D + 1.6W + f_1 L + 0.5(L_r \text{ or } S \text{ or } R)$ **(Formula 16-4)**

$1.2D + 1.0E + f_1 L + f_2 S$ **(Formula 16-5)**

$0.9D + (1.0E \text{ or } 1.6W)$ **(Formula 16-6)**

where:

f_1 = 1.0 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79 kN/m^2), and for parking garage live load.

f_1 = 0.5 for other live loads.

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

f_2 = 0.2 for other roof configurations.

Exceptions:

1. For concrete structures where load combinations do not include seismic forces, the factored load combinations of ACI 318 Section 9.2 shall be used. For concrete structures designed using the design wind forces of ASCE 7, W shall be divided by the directionality factor K_d. For concrete structures designed using Section 1609.6, W shall be divided by a directionality factor of 0.85.
2. Where other factored load combinations are specifically required by the provisions of this code, such combinations shall take precedence.

1605.2.2 Other loads. Where F, H, P or T is to be considered in design, each applicable load shall be added to the above combinations in accordance with Section 2.3.2 of ASCE 7. Where F_a is to be considered in design, the load combinations of Section 2.3.3 of ASCE 7 shall be used.

1605.3 Load combinations using allowable stress design.

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

D **(Formula 16-7)**

$D + L$ **(Formula 16-8)**

$D + L + (L_r \text{ or } S \text{ or } R)$ **(Formula 16-9)**

$D + (W \text{ or } 0.7E) + L + (L_r \text{ or } S \text{ or } R)$ **(Formula 16-10)**

$0.6D + W$ **(Formula 16-11)**

$0.6D + 0.7E$ **(Formula 16-12)**

Exceptions:

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

1605.3.1.1 Load reduction. It is permitted to multiply the combined effect of two or more transient loads by 0.75 and add to the effect of dead load. The combined load used in design shall not be less than the sum of the effects of dead load and any one of the transient loads. The 0.7 factor on E does not apply for this provision.

Increases in allowable stresses specified in the appropriate materials section of this code or referenced standard shall not be used with the load combinations of Section 1605.3.1 except that a duration of load increase shall be permitted in accordance with Chapter 23.

1605.3.1.2 Other loads. Where F, H, P or T are to be considered in design, the load combinations of Section 2.4.1 of ASCE 7 shall be used. Where F_a is to be considered in design, the load combinations of Section 2.4.2 of ASCE 7 shall be used.

1605.3.2 Alternative basic load combinations. In lieu of the basic load combinations specified in Section 1605.3.1, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. When using these alternate basic load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced, where permitted by the material section of this code or referenced standard. Where wind loads are calculated in accordance with Section 1609.6 or ASCE 7, the coefficient w in the following formulas shall be taken as 1.3. For other wind loads ω shall be taken as 1.0.

$D + L + (L_r \text{ or } S \text{ or } R)$ **(Formula 16-13)**

$D + L + (\omega W)$ **(Formula 16-14)**

$D + L + \omega W + S/2$ **(Formula 16-15)**

$D + L + S + \omega W/2$ **(Formula 16-16)**

$D + L + S + E/1.4$ **(Formula 16-17)**

$0.9D + E/1.4$ **(Formula 16-18)**

Exceptions:

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

1605.3.2.1 Other loads. Where *F, H, P* or *T* are to be considered in design, 1.0 times each applicable load shall be added to the combinations specified in Section 1605.3.2.

1605.4 Special seismic load combinations. For both allowable stress design and strength design methods, where specifically required by Sections 1613 through 1622 or by Chapters 18 through 23, elements and components shall be designed to resist the forces due to Formula 16-19 when the effects of the seismic ground motion are additive to gravity forces and Formula 16-20 when the effects of the seismic ground motion counteract gravity forces.

$$1.2D + f_1 L + E_m \qquad \textbf{(Formula 16-19)}$$

$$0.9D + E_m \qquad \textbf{(Formula 16-20)}$$

where:

E_m = The maximum effect of horizontal and vertical forces as set forth in Section 1617.1.2.

f_1 = 1.0 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79 kN/m^2), and for parking garage live load.

f_1 = 0.5 for other live loads.

1605.5 Heliports and helistops. Heliport and helistop landing or touchdown areas shall be designed for the following loads, combined in accordance with Section 1605:

1. Dead load, *D*, plus the gross weight of the helicopter, D_h, plus snow load, *S*.
2. Dead load, *D*, plus two single concentrated impact loads, *L*, approximately 8 feet (2438 mm) apart applied anywhere on the touchdown pad (representing each of the helicopter's two main landing gear, whether skid type or wheeled type), having a magnitude of 0.75 times the gross weight of the helicopter. Both loads acting together total 1.5 times the gross weight of the helicopter.
3. Dead load, *D*, plus a uniform live load, *L*, of 100 pounds per square foot (4.79 kN/m^2).

SECTION 1606 DEAD LOADS

1606.1 Weights of materials and construction. In determining dead loads for purposes of design, the actual weights of materials and construction shall be used. In the absence of definite information, values used shall be subject to the approval of the building official.

1606.2 Weights of fixed service equipment. In determining dead loads for purposes of design, the weight of fixed service equipment, such as plumbing stacks and risers, electrical feeders, heating, ventilating and air conditioning systems and fire sprinkler systems, shall be included.

SECTION 1607 LIVE LOADS

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

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

1607.3 Uniform live loads. The live loads used in the design of buildings and other structures shall be the maximum loads expected by the intended use or occupancy but shall in no case be less than the minimum uniformly distributed unit loads required by Table 1607.1.

1607.4 Concentrated loads. Floors and other similar surfaces shall be designed to support the uniformly distributed live loads prescribed in Section 1607.2 or the concentrated load, in pounds (kilonewtons), given in Table 1607.1, whichever produces the greater load effects. Unless otherwise specified, the indicated concentration shall be assumed to be uniformly distributed over an area 2.5 feet 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.

1607.5 Partition loads. In office buildings and in other buildings where partition locations are subject to change, provision for partition weight shall be made, whether or not partitions are shown on the construction documents, unless the specified live load exceeds 80 pounds per square foot (3.83 kN/m^2). Such partition load shall not be less than a uniformly distributed live load of 20 pounds per square foot (0.96kN/m^2).

1607.6 [Comm 62.1607] Truck and bus garages. Minimum live loads for garages having trucks or buses shall be as specified in Table 1607.6, but shall not be less than 50 pounds per square foot (2.40 kN/m^2). Actual loads shall be used where they are greater than the loads specified in the table.

1607.6.1 Truck and bus garage live load application. The concentrated load and uniform load shall be uniformly distributed over a 10-foot (3048 mm) width on a line normal to the centerline of the lane placed within a 12-foot-wide (3658 mm) lane. The loads shall be placed within their individual lanes so as to produce the maximum stress in each structural member. Single spans shall be designed for the uniform load in Table 1607.6 and one simultaneous concentrated load positioned to produce the maximum effect. Multiple spans shall be designed for the uniform load in Table 1607.6 on the spans and two simultaneous concentrated loads in two spans positioned to produce the maximum negative moment effect. Multiple span design loads, for other effects, shall be the same as for single spans.

1607.7 Loads on handrails, guards, grab bars and vehicle barriers. Handrails, guards, grab bars as designed in ICC A117.1, and vehicle barriers shall be designed and constructed to the structural loading conditions set forth in this section.

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
1. Apartments (see residential)	—	—
2. Access floor systems		
Office use	50	2,000
Computer use	100	2,000
3. Armories and drill rooms	150	—
4. Assembly areas and theaters		—
Fixed seats (fastened to floor)	60	
Lobbies	100	
Movable seats	100	
Stages and platforms	125	
Follow spot, projections and control rooms	50	
Catwalks	40	
5. Balconies (exterior)	100	—
On one- and two-family residences only, and not exceeding 100 ft.^{2}	60	
6. Decks	Same as occupancy served[h]	—
7. Bowling alleys	75	—
8. Cornices	60	—
9. Corridors, except as otherwise indicated	100	—
10. Dance halls and ballrooms	100	—
11. Dining rooms and restaurants	100	—
12. Dwellings (see residential)	—	—
13. Elevator machine room grating (on area of 4 in.^{2})	—	300
14. Finish light floor plate construction (on area of 1 in.^{2})	—	200
15. Fire escapes	100	—
On single-family dwellings only	40	
16. Garages (passenger cars only)	50	Note a
Trucks and buses	See Section 1607.6	
17. Grandstands (see stadium and arena bleachers)	—	—
18. Gymnasiums, main floors and balconies	100	—
19. Handrails, guards and grab bars	See Section 1607.7	
20. Hospitals		
Operating rooms, laboratories	60	1,000
Private rooms	40	1,000
Wards	40	1,000
Corridors above first floor	80	1,000
21. Hotels (see residential)	—	—
22. Libraries		
Reading rooms	60	1,000
Stack rooms	150[b]	1,000
Corridors above first floor	80	1,000
23. Manufacturing		
Light	125	2,000
Heavy	250	3,000
24. Marquees and canopies	75	—
25. Office buildings		
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Lobbies and first floor corridors	100	2,000
Offices	50	2,000
Corridors above first floor	80	2,000
26. Penal Institutions		—
Cell blocks	40	
Corridors	100	
27. Residential		—
Group R-3 as applicable in Section 101.2		
Uninhabitable attics without storage	10	
Uninhabitable attics with storage	20	
Habitable attics and sleeping areas	30	
All other areas except balconies and decks	40	
Hotels and multifamily dwellings		
Private rooms	40	
Public rooms and corridors serving them	100	
28. Reviewing stands, grandstands and bleachers	100[c]	—
29. Roofs	See Section 1607.11	
30. Schools		
Classrooms	40	1,000
Corridors above first floor	80	1,000
First floor corridors	100	1,000
31. Scuttles, skylight ribs, and accessible ceilings	—	200
32. Sidewalks, vehicular driveways and yards, subject to trucking	250[d]	8,000[e]
33. Skating rinks	100	—
34. Stadiums and arenas		—
Bleachers	100[c]	
Fixed seats (fastened to floor)	60[c]	
35. Stairs and exits	100	Note f
One- and two-family dwellings	40	
All other	100	
36. Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		—
Light	125	
Heavy	250	
37. Stores		
Retail		
First floor	100	1,000
Upper floors	75	1,000
Wholesale, all floors	125	1,000
38. Vehicle barriers	See Section 1607.7	
39. Walkways and elevated platforms (other than exitways)	60	—
40. Yards and terraces, pedestrians	100	—

(continued)

NOTES TO TABLE 1607.1

For SI: 1 square inch = 645.16 mm^2, 1 pound per square foot = 0.0479 kN/m^2, 1 pound = 0.004448 kN.

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 1607.1 or the following concentrated load: (1) for passenger cars accommodating not more than nine passengers, 2,000 pounds acting on an area of 20 square inches; (2) mechanical parking structures without slab or deck, passenger car only, 1,500 pounds per wheel.

b. The weight of books and shelving shall be computed using an assumed density of 65 pounds per cubic foot and converted to a uniformly distributed load; this load shall be used if it exceeds 150 pounds per square foot.

c. In addition to the vertical live loads, horizontal swaying forces parallel and normal to the length of seats shall be included in the design according to the requirements of NFPA 102.

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

e. The concentrated wheel load shall be applied on an area of 20 square inches.

f. Minimum concentrated load on stair treads (on area of 4 square inches) is 300 pounds.

g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official. See Section 1608. For special-purpose roofs, see Section 1607.11.2.2.

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

TABLE 1607.6
UNIFORM AND CONCENTRATED LOADS

LOADING CLASS[a]	UNIFORM LOAD (pounds/linear foot of lane)	CONCENTRATED LOAD (pounds)[b]	
		For moment design	For shear design
H20-44 and HS20-44	640	18,000	26,000
H15-44 and HS15-44	480	13,500	19,500

For SI: 1 pound per linear foot = 0.01459 kN/m, 1 pound = 0.004448 kN, 1 ton = 8.90 kN.

a. An H loading class designates a two-axle truck with a semi-trailer. An HS loading class designates a tractor truck with a semi-trailer. The numbers following the letter classification indicate the gross weight in tons of the standard truck and the year the loadings were instituted.

b. See Section 1607.6.1 for the loading of multiple spans.

1607.7.1 Handrails and guards. Handrail assemblies and guards shall be designed to resist a load of 50 pounds per linear foot (pound per foot) (0.73 kN/m) applied in any direction at the top and to transfer this load through the supports to the structure.

Exceptions:

1. For one- and two-family dwellings, only the single, concentrated load required by Section 1607.7.1.1 shall be applied.
2. In Group I-3, F, H and S occupancies, for areas that are not accessible to the general public and that have an occupant load no greater than 50, the minimum load shall be 20 pounds per foot (0.29 kN/m).

1607.7.1.1 Concentrated load. Handrail assemblies and guards shall be able to resist a single concentrated load of 200 pounds (0.89 kN), applied in any direction at any point along the top, and have attachment devices and supporting structure to transfer this loading to appropriate structural elements of the building. This load need not be assumed to act concurrently with the loads specified in the preceding paragraph.

1607.7.1.2 Components. Intermediate rails (all those except the handrail), balusters and panel fillers shall be designed to withstand a horizontally applied normal load of 50 pounds (0.22 kN) on an area not to exceed 1 square foot (305 mm^2) including openings and space between rails. Reactions due to this loading are not required to be superimposed with those of either preceding paragraph.

1607.7.1.3 Stress increase. Where handrails and guards are designed in accordance with the provisions for allowable stress design (working stress design) exclusively for the loads specified in Section 1607.7.1, the allowable stress for the members and their attachments are permitted to be increased by one-third.

1607.7.2 Grab bars, shower seats and dressing room bench seats. Grab bars, shower seats and dressing room bench seat systems shall be designed to resist a single concentrated load of 250 pounds (1.11 kN) applied in any direction at any point.

1607.7.3 Vehicle barriers. Vehicle barrier systems for passenger cars shall be designed to resist a single load of 6,000 pounds (26.70 kN) applied horizontally in any direction to the barrier system and shall have anchorage or attachment capable of transmitting this load to the structure. For design of the system, the load shall be assumed to act at a minimum height of 1 foot, 6 inches (457 mm) above the floor or ramp surface on an area not to exceed 1 square foot (305 mm^2), and is not required to be assumed to act concurrently with any handrail or guard loadings specified in the preceding paragraphs of Section 1607.7.1. Garages accommodating trucks and buses shall be designed in accordance with an approved method that contains provision for traffic railings.

1607.8 Impact loads. The live loads specified in Section 1607.2 include allowance for impact conditions. Provision shall be made in the structural design for uses and loads that involve unusual vibration and impact forces.

1607.8.1 Elevators. Elevator loads shall be increased by 100 percent for impact and the structural supports shall be designed within the limits of deflection prescribed by ASME A17.1.

1607.8.2 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; (4) hangers for floors or balconies, 33 percent. Percentages shall be increased where specified by the manufacturer.

1607.9 Reduction in live loads. The minimum uniformly distributed live loads, L_o, in Table 1607.1 are permitted to be reduced according to the following provisions.

1607.9.1 General. Subject to the limitations of Section 1607.9.1.1 through 1607.9.1.4, members for which a value of $K_{LL}A_T$ is 400 square feet (37.16 m²) or more are permitted to be designed for a reduced live load in accordance with the following equation:

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \qquad \textbf{(Equation 16-1)}$$

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

where:

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

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

K_{LL} = Live load element factor (see Table 1607.9.1).

A_T = Tributary area, in square feet (square meters).

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.

TABLE 1607.9.1
LIVE LOAD ELEMENT FACTOR, K_{LL}

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

1607.9.1.1 Heavy live loads. Live loads that exceed 100 pounds per foot squared (4.79 kN/m²) shall not be reduced except the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than L as calculated in Section 1607.9.1.

1607.9.1.2 Passenger car garages. The live loads shall not be reduced in passenger car garages except the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than L as calculated in Section 1607.9.1.

1607.9.1.3 Special occupancies. Live loads of 100 pounds per foot squared (4.79 kN/m²) or less shall not be reduced in public assembly occupancies.

1607.9.1.4 Special structural elements. Live loads shall not be reduced for one-way slabs except as permitted in Section 1607.9.1.1. Live loads of 100 pound per foot squared (4.79 kN/m²) or less shall not be reduced for roof members except as specified in Section 1607.11.2.

1607.9.2 Alternate floor live load reduction. As an alternative to Section 1607.9.1, floor live loads are permitted to be reduced in accordance with the following provisions. Such reductions shall apply to slab systems, beams, girders columns, piers, walls and foundations.

1. A reduction shall not be permitted in Group A occupancies.
2. A reduction shall not be permitted when the live load exceeds 100 pounds per square foot (4.79 kN/m²) except that the design live load for columns may be reduced by 20 percent.
3. For live loads not exceeding 100 pounds per square foot (4.79 kN/m²), the design live load for any structural member supporting 150 square feet (13.94 m²) or more is permitted to be reduced in accordance with the following equation:

$$R = r(A - 150) \qquad \textbf{(Equation 16-2)}$$

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

Such reduction shall not exceed 40 percent for horizontal members, 60 percent for vertical members, nor R as determined by the following equation:

$$R = 23.1(1 + D/L_o) \qquad \textbf{(Equation 16-3)}$$

where:

A = Area of floor or roof supported by the member, square feet (m²).

D = Dead load per square foot (m²) of area supported.

L_o = Unreduced live load per square foot (m²) of area supported.

R = Reduction in percent.

r = Rate of reduction equal to 0.08 percent for floors.

1607.10 Distribution of floor loads. Where uniform floor live loads are involved in the design of structural members arranged so as to create continuity, the minimum applied loads shall be the full dead loads on all spans in combination with the reduced floor live load or the full live loads on adjacent spans and on alternate spans.

1607.11 Roof loads. The structural supports of roofs and marquees shall be designed to resist wind and, where applicable, snow and earthquake loads, in addition to the dead load of construction and the appropriate live loads as prescribed in this section, or as set forth in Table 1607.1. The live loads acting on a sloping surface shall be assumed to act vertically on the horizontal projection of that surface.

1607.11.1 Distribution of roof loads. Where uniform roof live loads are involved in the design of structural members arranged so as to create continuity, the minimum applied loads shall be the full dead loads on all spans in combination with full roof live loads on adjacent spans and on alternate spans. See Section 1608.5 for partial snow loading.

1607.11.2 Minimum roof live loads. Minimum roof loads shall be determined for the specific conditions in accordance with Sections 1607.11.2.1 through 1607.11.2.5.

1607.11.2.1 Flat, pitched and curved roofs. Ordinary flat, pitched and curved roofs shall be designed for the live loads specified in the following formula or other controlling combinations of loads in Section 1605, whichever produces the greater load. In structures, where special scaffolding is used as a work surface for workers and materials during maintenance and repair operations, a lower roof load than specified in the following formula shall not be used unless approved by the building official. Greenhouses shall be designed for a minimum roof live load of 10 pounds per square foot (0.479 kN/m²).

$L_r = 20R_1R_2$ **(Equation 16-4)**

where: $12 \leq L_r \leq 20$

For SI: $L_r = 0.96\ R_1R_2$

where: $0.58 \leq L_r \leq 0.96$

L_r = Roof live load per square foot (m²) of horizontal projection in pounds per square foot (kN/m²).

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

$R_1 = 1$ for $A_t \leq 200$ square feet (18.58 m²) **(Equation 16-5)**

$R_1 = 1.2 - 0.001A_t$ for 200 square feet $< A_t <$ 600 square feet **(Equation 16-6)**

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

$R_1 = 0.6$ for $A_t \geq 600$ square feet (55.74 m²) **(Equation 16-7)**

where:

A_t = Tributary area (span length multiplied by effective width) in square feet (m²) supported by any structural member, and

F = for a sloped roof, the number of inches of rise per foot (for SI: $F = 0.12 \times$ slope, with slope expressed in percentage points), and

F = for an arch or dome, rise-to-span ratio multiplied by 32, and

$R_2 = 1$ for $F \leq 4$ **(Equation 16-8)**

$R_2 = 1.2 - 0.05\ F$ for $4 < F < 12$ **(Equation 16-9)**

$R_2 = 0.6$ for $F \geq 12$ **(Equation 16-10)**

1607.11.2.2 Special-purpose roofs. Roofs used for promenade purposes shall be designed for a minimum live load of 60 pounds per square foot (2.87 kN/m²). Roofs used for roof gardens or assembly purposes shall be designed for a minimum live load of 100 pounds per square foot (4.79 kN/m²). Roofs used for other special purposes shall be designed for appropriate loads, as directed or approved by the building official.

1607.11.2.3 Landscaped roofs. Where roofs are to be landscaped, the uniform design live load in the landscaped area shall be 20 pounds per square foot (0.958 kN/m²). The weight of the landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil.

1607.11.2.4 Awnings and canopies. Awnings and canopies shall be designed for a uniform live load of 5 pounds per square foot (0.240 kN/m²) as well as for snow loads and wind loads as specified in Sections 1608 and 1609.

1607.11.2.5 Overhanging eaves. In other than occupancies in Group R-3 as applicable in Section 101.2, and except where the overhang framing is a continuation of the roof framing, overhanging eaves, cornices and other roof projections shall be designed for a minimum uniformly distributed live load of 60 pounds per square foot (2.87 kN/m²).

1607.12 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.

1607.12.1 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.

1607.12.2 Vertical impact force. The maximum wheel loads of the crane shall be increased by the percentages shown below to determine the induced vertical impact or vibration force:

Monorail cranes (powered) . . . . 25 percent

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

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

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

1607.12.3 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 according to the lateral stiffness of the runway beam and supporting structure.

1607.12.4 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.

1607.13 Interior walls and partitions. Interior walls and partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength to resist the loads to which they are subjected but not less than a horizontal load of 5 pounds per square foot (0.240 kN/m²).

SECTION 1608 SNOW LOADS

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

1608.2 Ground snow loads. The ground snow loads to be used in determining the design snow loads for roofs are given in Figure 1608.2 for the contiguous United States and Table 1608.2 for Alaska. Site-specific case studies shall be made in areas designated CS in Figure 1608.2. Ground snow loads for sites at elevations above the limits indicated in Figure 1608.2 and for all sites within the CS areas shall be approved. Ground snow load determination for such sites shall be based on an extreme value statistical analysis of data available in the vicinity of the site using a value with a 2-percent annual probability of being exceeded (50-year mean recurrence interval). Snow loads are zero for Hawaii, except in mountainous regions as approved by the building official.

Comm 62.1608 (1) Alternative ground snow loads.

(a) A ground snow load of 35 pounds per square foot may be assumed for the south zone in Figure 62.16–1.

(b) A ground snow load of 40 pounds per square foot may be assumed for the middle zone in Figure 62.16–1.

(c) A ground snow load of 60 pounds per square foot may be assumed for the north zone in Figure 62.16–1.

1608.3 Flat roof snow loads. The flat roof snow load, p_f, on a roof with a slope equal to or less than 5 degrees (0.09 rad) (1 inch per foot = 4.76 degrees) shall be calculated in accordance with Section 7.3 of ASCE 7.

1608.3.1 Exposure factor. The value for the snow exposure factor, C_e, used in the calculation of p_f shall be determined from Table 1608.3.1.

Comm 62.1608 (2) Alternative exposure factor. A snow exposure factor of 1.0 may be used for any flat roof.

1608.3.2 Thermal factor. The value for the thermal factor, C_t, used in the calculation of p_f shall be determined from Table 1608.3.2.

1608.3.3 Snow load importance factor. The value for the snow load importance factor, I_S, used in the calculation of p_f shall be determined in accordance with Table 1604.5. Greenhouses that are occupied for growing plants on production or research basis, without public access, shall be included in Importance Category IV.

1608.3.4 Rain-on-snow surcharge load. Roofs with a slope less than $^1/_2$ inch per foot (2.38 degrees) shall be designed for a rain-on-snow surcharge load determined in accordance with Section 7.10 of ASCE 7.

1608.3.5 Ponding instability. For roofs with a slope less than $^1/_4$ inch per foot (1.19 degrees), the design calculations shall include verification of the prevention of ponding instability in accordance with Section 7.11 of ASCE 7.

1608.4 Sloped roof snow loads. The snow load, p_s, on a roof with a slope greater than 5 degrees (0.09 rad) (1 inch per foot = 4.76 degrees) shall be calculated in accordance with Section 7.4 of ASCE 7.

1608.5 Partial loading. The effect of not having the balanced snow load over the entire loaded roof area shall be analyzed in accordance with Section 7.5 of ASCE 7.

1608.6 Unbalanced snow loads. Unbalanced roof snow loads shall be determined in accordance with Section 7.6 of ASCE 7. Winds from all directions shall be accounted for when establishing unbalanced snow loads.

TABLE 1608.2
GROUND SNOW LOADS, p_g, FOR ALASKAN LOCATIONS

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

For SI: 1 pound per square foot = 0.0479 kN/m².

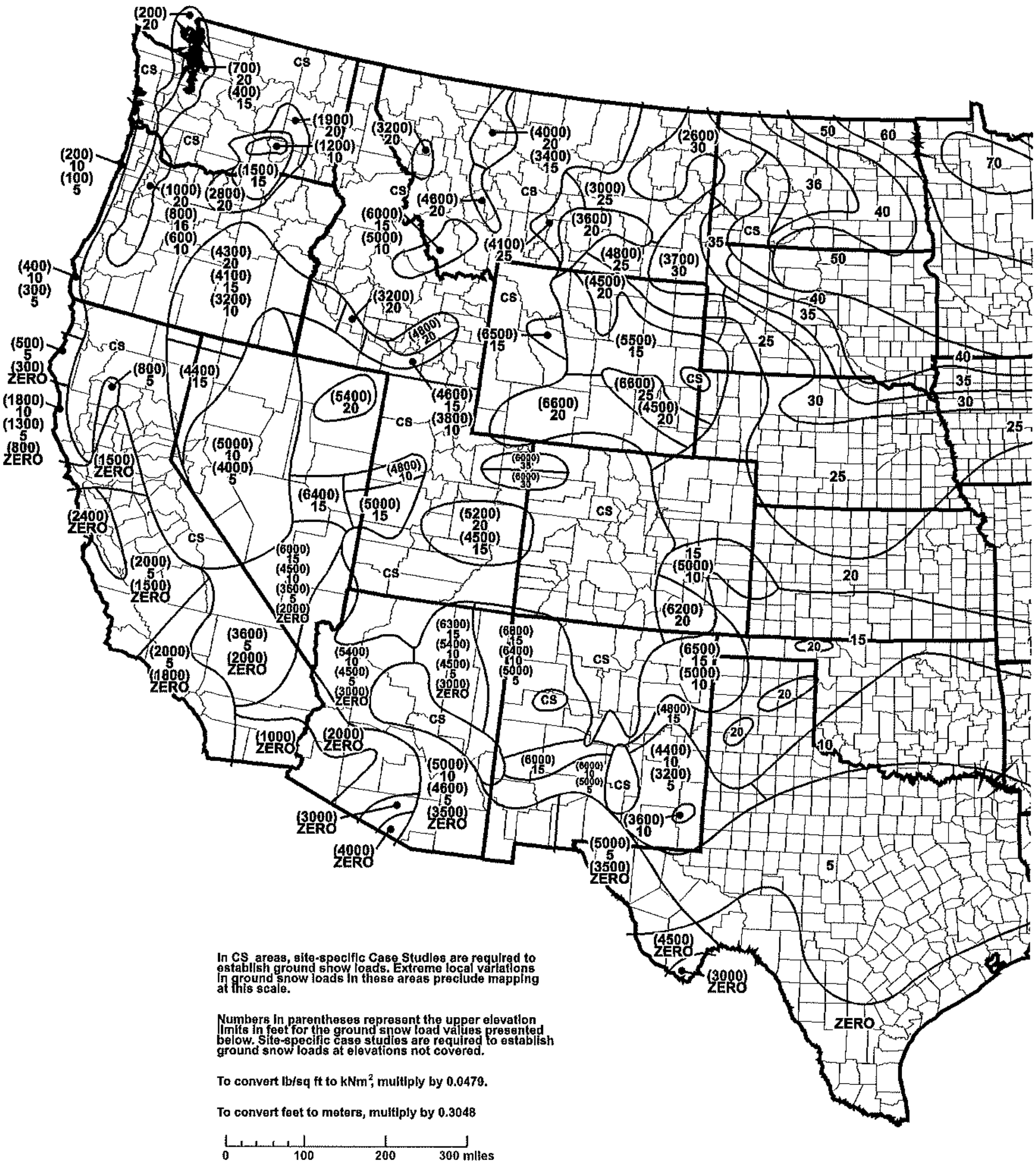


FIGURE 1608.2
GROUND SNOW LOADS, p_g, FOR THE UNITED STATES (psf)

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

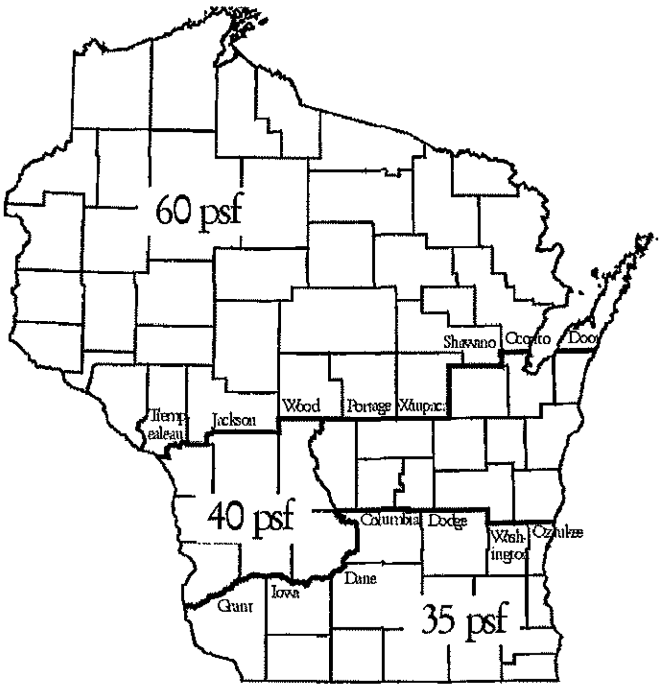


FIGURE 62.16–1
GROUND SNOW LOAD ZONES

1608.7 Drifts on lower roofs. In areas where the ground snow load, p_g, as determined by Section 1608.2, is equal to or greater than 5 pounds per square foot (0.240 kN/m^2), roofs shall be designed to sustain localized loads from snow drifts in accordance with Section 7.7 of ASCE 7.

1608.8 Roof projections. Drift loads due to mechanical equipment, penthouses, parapets and other projections above the roof shall be determined in accordance with Section 7.8 of ASCE 7.

1608.9 Sliding snow. The extra load caused by snow sliding off a sloped roof onto a lower roof shall be determined in accordance with Section 7.9 of ASCE 7.

SECTION 1609
WIND LOADS

1609.1 Applications. Buildings, structures and parts thereof shall be designed to withstand the minimum wind loads prescribed herein. Decreases in wind loads shall not be made for the effect of shielding by other structures. Wind pressures shall be assumed to come from any horizontal direction and to act normal to the surfaces considered.

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Section 6 of ASCE 7.

Exceptions:

1. Wind loads determined by the provisions of Section 1609.6.
2. Subject to the limitations of Section 1609.1.1.1, the provisions of SBCCI SSTD 10 shall be permitted for applicable Group R2 and R3 buildings.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of the *AF & PA Wood Frame Construction Manual for One and Two Family Dwellings, SBC High Wind Edition.*
4. Designs using NAAMM 1001 Guide Specifications for Design of Metal Flag Poles.

Comm 62.1609 (1) Alternative determination of wind loads. For buildings that meet all of the following conditions, wind

TABLE 1608.3.1
SNOW EXPOSURE FACTOR, C_e

TERRAIN CATEGORY[a]	EXPOSURE OF ROOF[a,b]		
	Fully exposed[c]	Partially exposed	Sheltered
A (see Section 1609.4)	N/A	1.1	1.3
B (see Section 1609.4)	0.9	1.0	1.2
C (see Section 1609.4)	0.9	1.0	1.1
D (see Section 1609.4)	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 radius of the site	0.7	0.8	N/A

For SI: 1 mile = 1609 344 m.

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.

b. Definitions of roof exposure are as follows:
1. Fully exposed shall mean roofs exposed on all sides with no shelter afforded by terrain, higher structures or trees. Roofs that contain several large pieces of mechanical equipment, parapets which extend above the height of the balanced snow load, h_b, or other obstructions are not in this category.
2. Partially exposed shall include all roofs except those designated as "fully exposed" or "sheltered."
3. Sheltered roofs shall mean those roofs located tight in among conifers that qualify as "obstructions."

c. Obstructions within a distance of 10 h_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 except for terrain category "A." Note that these are heights above the roof. Heights used to establish the terrain category in Section 1609.4 are heights above the ground.

TABLE 1608.3.2
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 ft^2 · hr · °F/Btu	1.1
Unheated structures	1.2
Continuously heated greenhouses[b] with a roof having a thermal resistance (*R*-value) less than 2.0 ft^2 · hr · °F/Btu	0.85

For SI: °C = [(°F)-32]/1.8, 1 British thermal unit per hour = 0.2931W.

a. The thermal condition shall be representative of the anticipated conditions during winters for the life of the structure.

b. A continuously heated greenhouse shall mean a greenhouse with a constantly maintained interior temperature of 50°F or more during winter months. Such greenhouse shall also have a maintenance attendant on duty at all times or a temperature alarm system to provide warning in the event of a heating system failure.

loads may be determined by applying only Table 6-2 in ASCE 7-98:

(a) The total building volume is less than 50,000 cubic feet.

(b) The building height is less than 30 feet (9144 mm).

(c) The wind exposure is Category C.

(d) Roof overhangs are designed to resist an uplift load of at least 30 pounds per square foot.

1609.1.1.1 Applicability. The provisions of SSTD 10 and the *AF & PA Wood Frame Construction Manual for One and Two Family Dwellings, SBC High Wind Edition* are applicable only to buildings located within Exposure A, B or C as defined in Section 1609.4. The provisions shall not apply to buildings sited on the upper half of an isolated hill, ridge or escarpment meeting the following conditions:

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

1609.1.2 Minimum wind loads. The wind loads used in the design of the main wind-force-resisting system shall not be less than 10 pounds per square foot (0.479 kN/m^2) multiplied by the area of the building or structure projected on a vertical plane normal to the wind direction. In the calculation of design wind loads for components and cladding for buildings, the algebraic sum of the pressures acting on opposite faces shall be taken into account. The design pressure for components and cladding of buildings shall not be less than 10 pounds per square foot (0.479 kN/m^2) acting in either direction normal to the surface. The design force for open buildings and other structures shall not be less than 10 pounds per square foot (0.479 kN/m^2) multiplied by the area A_f.

1609.1.3 Anchorage against overturning, uplift and sliding. Structural members and systems, and components and cladding in a building or structure shall be anchored to resist wind-induced overturning, uplift and sliding and to provide continuous load paths for these forces to the foundation. Where a portion of the resistance to these forces is provided by dead load, the dead load shall be taken as the minimum dead load likely to be in place during a design wind event. Where the alternate basic load combinations of Section 1605.3.2 are used, only two-thirds of the minimum dead load likely to be in place during a design wind event shall be used.

1609.1.4 Protection of openings. In wind-borne debris regions, glazing in the lower 60 feet (18 288 mm) in buildings shall be assumed to be openings unless such glazing is impact resistant or protected with an impact-resistant covering meeting the requirements of an approved impact- resisting standard or ASTM E 1996 and of ASTM E 1886 referenced therein as follows:

1. Glazed openings located within 30 feet (9144 mm) of grade shall meet the requirements of the Large Missile Test of ASTM E 1996.
2. Glazed openings located more than 30 feet (9144 mm) above grade shall meet the provisions of the Small Missile Test of ASTM E 1996.

Exception: Wood structural panels with a minimum thickness of $^7/_{16}$ inch (11.1 mm) and maximum panel span of 8 feet (2438 mm) are permitted for opening protection in one- and two-story buildings. Panels shall be precut to cover the glazed openings with attachment hardware provided. Attachments shall be designed to resist the components and cladding loads determined in accordance with the provisions of Section 1609.6.5. Attachment in accordance with Table 1609.1.4 is permitted for buildings with a mean roof height of 33 feet (10 058 mm) or less where wind speeds (3-second gust) do not exceed 130 miles per hour.

TABLE 1609.1.4
WINDBORNE DEBRIS PROTECTION FASTENING SCHEDULE FOR WOOD STRUCTURAL PANELS[a,b,c]

FASTENER TYPE	FASTENER SPACING (inches)			
	Panel span ≤ 2 feet	2 feet ≤ Panel span ≤ 4 feet	4 feet < Panel span ≤ 6 feet	6 feet < Panel span ≤ 8 feet
$2^1/_2$ #6 Wood Screws	16	16	12	9
$2^1/_2$ #8 Wood Screws	16	16	16	12

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound = 0.454 kg, 1 mile per hour = 0.44 m/s.

a. This table is based on a maximum wind speed (3-second gust) of 130 mph and mean roof height of 33 feet or less.

b. Fasteners shall be installed at opposing ends of the wood structural panel.

c. Where screws are attached to masonry or masonry/stucco, they shall be attached utilizing vibration-resistant anchors having a minimum withdrawal capacity of 490 pounds.

1609.1.5 Wind and seismic detailing. Lateral-force-resisting systems shall meet seismic detailing requirements and limitations prescribed in this code, even when wind code prescribed load effects are greater than seismic load effects.

1609.2 Definitions. The following words and terms shall, for the purposes of Section 1609.6, have the meanings shown herein.

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

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

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 feet (18 288 mm).
2. Mean roof height, h, does not exceed least horizontal dimension.

BUILDING, OPEN. A building having each wall at least 80 percent open. This condition is expressed for each wall by the equation:

$$A_o \geq 0.8 A_g \qquad \textbf{(Equation 16-11)}$$

where:

A_o = Total area of openings in a wall that receives positive external pressure, in square feet (m^2).

A_g = The gross area of that wall in which A_o is identified, in square feet (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; and
2. The total area of openings in a wall that receives positive external pressure exceeds 4 square feet (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:

$$A_o > 1.10 A_{oi} \qquad \textbf{(Equation 16-12)}$$

A_o > 4 square feet (0.37 m^2) or $> 0.01 A_g$, whichever is smaller, and $A_{oi}/A_{gi} \leq 0.20$ **(Equation 16-13)**

where:

A_o, A_g are as defined for an open building.

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

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

BUILDING, SIMPLE DIAPHRAGM. A building that complies with all of the following conditions:

1. Enclosed building,
2. Mean roof height h less than or equal to 60 feet (18 288 mm),
3. Mean roof height h does not exceed least horizontal dimension,
4. Building has an approximately symmetrical cross section,
5. Building has no expansion joints or structural separations within the building,
6. Wind loads are transmitted through floor and roof diaphragms to the vertical lateral-force-resisting systems,
7. If the building has moment resisting frames, roof slopes do not exceed 30 degrees (0.5235 rad).

COMPONENTS AND CLADDING. Elements of the building envelope that do not qualify as part of the main windforce-resisting system.

EFFECTIVE WIND AREA. The area used to determine GC_p. For component and cladding elements, the effective wind area in Tables 1609.6.2.1(2) and 1609.6.2.1(3) is the span length multiplied by an effective width that need not be less than one-third the span length. For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener.

HURRICANE-PRONE REGIONS. Areas vulnerable to hurricanes defined as:

1. The U.S. Atlantic Ocean and Gulf of Mexico coasts where the basic wind speed is greater than 90 mph and
2. Hawaii, Puerto Rico, Guam, Virgin Islands and American Samoa.

IMPORTANCE FACTOR, *I*. A factor that accounts for the degree of hazard to human life and damage to property.

MAIN WINDFORCE-RESISTING SYSTEM. An assemblage of structural elements assigned to provide support and stability for the overall structure. The system generally receives wind loading from more than one surface.

MEAN ROOF HEIGHT. The average of the roof eave height and the height to the highest point on the roof surface, except that eave height shall be used for roof angle of less than or equal to 10 degrees (0.1745 rad).

WIND-BORNE DEBRIS REGION. Areas within hurricane-prone regions within 1 mile (1.61 km) of the coastal mean high water line where the basic wind speed is 110 miles (48.4 m/s) per hour or greater; or where the basic wind speed is 120 miles (52.8 m/s) per hour or greater; or Hawaii.

1609.3 Basic wind speed. The basic wind speed, in miles per hour, for the determination of the wind loads shall be determined by Figure 1609 or by ASCE 7 Figure 6-1 when using the provisions of ASCE 7. Basic wind speed for the special wind regions indicated, near mountainous terrain, and near gorges, shall be in accordance with local jurisdiction requirements. Basic wind speeds determined by the local jurisdiction shall be in accordance with Section 6.5.4 of ASCE 7.

1609.3.1 Wind speed conversion. When required, the 3-second gust wind velocities of Figure 1609 shall be converted to fastest mile wind velocities using Table 1609.3.1.

1609.4 Exposure category. For each wind direction considered, an exposure category that adequately reflects the characteristics of ground surface irregularities shall be determined for the site at which the building or structure is to be constructed. For a site located in the transition zone between categories, the category resulting in the largest wind forces shall apply. Account shall be taken of variations in ground surface roughness that arise from natural topography and vegetation as well as from constructed features. For any given wind direction, the exposure in which a specific building or other structure is sited shall be assessed as being one of the following categories:

1. **Exposure A.** Large city centers with at least 50 percent of the buildings having a height in excess of 70 feet (21 356 mm). Use of this exposure category shall be limited to those areas for which terrain representative of Exposure A prevails in the upwind direction for a distance of at least 0.5 mile (0.8 km) or 10 times the height of the building or other structure, whichever is greater. Possible channeling effects or increased velocity pressures due to the building or structure being located in the wake of adjacent buildings shall be taken into account.
2. **Exposure B.** Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger. Exposure B shall be assumed unless the site meets the definition of another type exposure.
3. **Exposure C.** Open terrain with scattered obstructions, including surface undulations or other irregularities, having heights generally less than 30 feet (9144 mm) extending more than 1,500 feet (457.2 m) from the building site in any quadrant. This exposure shall also apply to any building located within Exposure B type terrain where the building is directly adjacent to open areas of Exposure C type terrain in any quadrant for a distance of more than 600 feet (182.9 m). This category includes flat open country, grasslands and shorelines in hurricane-prone regions.
4. **Exposure D.** Flat, unobstructed areas exposed to wind flowing over open water (excluding shorelines in hurricane-prone regions) for a distance of at least 1 mile (1.61 km). Shorelines in Exposure D include inland waterways, the Great Lakes and coastal areas of California, Oregon, Washington and Alaska. This exposure shall apply only to those buildings and other structures exposed to the wind coming from over the water. Exposure D extends inland from the shoreline a distance of 1,500 feet (460 m) or 10 times the height of the building or structure, whichever is greater.

1609.5 Importance factor. Buildings and other structures shall be assigned a wind load importance factor, I_w, in accordance with Table 1604.5.

1609.6 Simplified provisions for low-rise buildings.

1609.6.1 Scope. The procedures in Section 1609.6 shall be used for determining and applying wind pressures in the design of simple diaphragm buildings with flat, gabled and hipped roofs and having a mean roof height not exceeding the least horizontal dimension or 60 feet (18 288 mm), whichever is less, subject to the following limitations:

The provisions of Section 1609.6 shall not apply to buildings sited on the upper half of an isolated hill or escarpment meeting all the following conditions:

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

1609.6.2 Wind pressures.

1609.6.2.1 Load determination. Structural members, cladding, fasteners and systems providing for the structural integrity of the building shall be designed for the loads from Tables 1609.6.2.1(1), 1609.6.2.1(2) and 1609.6.2.1(3) using Figure 1609, multiplied by the appropriate height and exposure coefficient from Table 1609.6.2.1(4) and importance factor from Table 1604.5.

1609.6.2.2 Load case. Members that act as both part of the main windforce-resisting system and as components and cladding shall be designed for each separate load case.

1609.6.3 Edge strips and end zones. The width of edge strips (a) shall be 10 percent of the least horizontal dimension or 40 percent of the eave height, whichever is less but not less than either 4 percent of the least horizontal dimension or 3 feet (914 mm). End zones as shown in Figure 1609.6.(1) shall be twice the width of the edge strip (a).

TABLE 1609.3.1
EQUIVALENT BASIC WIND SPEEDS[a,b,c]

V_{3s}	85	90	100	105	110	120	125	130	140	145	150	160	170
V_{fm}	70	75	80	85	90	100	105	110	120	125	130	140	150

For SI: 1 mile per hour = 0.44 m/s.
a. Linear interpolation is permitted.
b. V_{3s} is the second gust wind speed (mph).
c. V_{fm} is the fastest mile wind speed (mph).

TABLE 1609.6.2.1(1)
MAIN WINDFORCE-RESISTING SYSTEM LOADS FOR A BUILDING WITH MEAN ROOF HEIGHT OF 30 FEET LOCATED IN EXPOSURE B[a] (psf)

BASIC WIND SPEED V (mph–3-second gust)	LOAD DIRECTION	ROOF ANGLE	HORIZONTAL LOADS[b]				VERTICAL LOADS						MAXIMUM HORIZONTAL WALL LOADS[d]			
			End zone		Interior zone		End zone[e]		Interior zone		Windward overhang		Zone			
			Wall	Roof[c]	Wall	Roof[c]	Windward roof	Leeward roof	Windward roof	Leeward roof	End zone	Interior zone	1E	4E	1	4
85	Transverse	0 to 5°	11.5	-5.9	7.6	-3.5	-13.8	-7.8	-9.6	-6.1	-19.3	-15.1	8.7	-6.7	6.4	-5.2
		20°	15.9	-4.2	10.6	-2.3	-13.8	-9.6	-9.6	-7.3	-19.3	-15.1	10.8	-9.0	7.8	-6.7
		30° < angle ≤ 45°	12.9	8.8	10.2	7.0	5.0	-7.8	4.3	-6.7	-4.5	-5.2	9.6	-7.3	8.2	-6.1
	Longitudinal	All angles	11.5	-5.9	7.6	-3.5	-13.8	-7.8	-9.6	-6.1	-19.3	-15.1	8.7	-6.7	6.4	-5.2
90	Transverse	0 to 5°	12.8	-6.7	8.5	-4.0	-15.4	-8.8	-10.7	-6.8	-21.6	-16.9	9.8	-7.5	7.2	-5.8
		20°	17.8	-4.7	11.9	-2.6	-15.4	-10.7	-10.7	-8.1	-21.6	-16.9	12.1	-10.1	8.8	-7.5
		30° < angle ≤ 45°	14.4	9.9	11.5	7.9	5.6	-8.8	4.8	-7.5	-5.1	-5.8	10.7	-8.1	9.1	-6.8
	Longitudinal	All angles	12.8	-6.7	8.5	-4.0	-15.4	-8.8	-10.7	-6.8	-21.6	-16.9	9.8	-7.5	7.2	-5.8
100	Transverse	0 to 5°	15.9	-8.2	10.5	-4.9	-19.1	-10.8	-13.3	-8.4	-26.7	-20.9	12.0	-9.3	8.8	-7.2
		20°	22.0	-5.8	14.6	-3.2	-19.1	-13.3	-13.3	-10.1	-26.7	-20.9	14.9	-12.5	10.8	-9.3
		30° < angle ≤ 45°	17.8	12.2	14.2	9.8	6.9	-10.8	5.9	-9.3	-6.3	-7.2	13.3	-10.1	11.3	-8.4
	Longitudinal	All angles	15.9	-8.2	10.5	-4.9	-19.1	-10.8	-13.3	-8.4	-26.7	-20.9	12.0	-9.3	8.8	-7.2
105	Transverse	0 to 5°	17.5	-9.1	11.6	-5.4	-21.0	-11.9	-14.6	-9.2	-29.4	-23.0	13.3	-10.3	9.7	-7.9
		20°	24.2	-6.4	16.1	-3.5	-21.0	-14.6	-14.6	-11.1	-29.4	-23.0	16.5	-13.8	11.9	-10.3
		30° < angle ≤ 45°	19.7	-13.4	15.6	10.8	7.6	-11.9	6.6	-10.3	-6.9	-7.9	14.6	-11.1	12.4	-9.2
	Longitudinal	All angles	17.5	-9.1	11.6	-5.4	-21.0	-11.9	-14.6	-9.2	-29.4	-23.0	13.3	-10.3	9.7	-7.9
110	Transverse	0 to 5°	19.2	-10.0	12.7	-5.9	-23.1	-13.1	-16.0	-10.1	-32.3	-25.3	14.6	-11.3	10.7	-8.7
		20°	26.6	-7.0	17.7	-3.9	-23.1	16.0	-16.0	-12.2	-32.3	-25.3	18.1	-15.1	13.1	-11.3
		30° < angle ≤ 45°	21.6	14.8	17.2	11.8	8.3	-13.1	7.2	-11.3	-7.6	-8.7	16.0	-12.2	13.7	-10.1
	Longitudinal	All angles	19.2	-10.0	12.7	-5.9	-23.1	-13.1	-16.0	-10.1	-32.3	-25.3	14.6	-11.3	10.7	-8.7
120	Transverse	0 to 5°	22.8	-11.9	15.1	-7.0	-27.4	-15.6	-19.1	-12.1	-38.4	-30.1	17.3	-13.4	12.7	-10.3
		20°	31.6	-8.3	21.1	-4.6	-27.4	-19.1	-19.1	-14.5	-38.4	-30.1	21.5	-18.0	15.6	-13.4
		30° < angle ≤ 45°	25.7	17.6	20.4	14.0	9.9	-15.6	8.6	-13.4	-9.0	-10.3	19.1	-14.5	16.2	-12.1
	Longitudinal	All angles	22.8	-11.9	15.1	-7.0	-27.4	-15.6	-19.1	-12.1	-38.4	-30.1	17.3	-13.4	12.7	-10.3
125	Transverse	0 to 5°	24.8	-12.9	16.4	-7.6	-29.8	-16.9	-20.7	-13.1	-41.7	-32.6	18.8	-14.5	13.8	-11.2
		20°	34.3	-9.1	22.9	-5.0	-29.8	-20.7	-20.7	-15.7	-41.7	-32.6	23.3	-19.5	16.9	-14.5
		30° < angle ≤ 45°	27.9	19.1	22.2	15.2	10.7	-16.9	9.3	-14.5	-9.8	-11.2	20.7	-15.7	17.6	-13.1
	Longitudinal	All angles	24.8	-12.9	16.4	-7.6	-29.8	-16.9	-20.7	-13.1	-41.7	-32.6	18.8	-14.5	13.8	-11.2

(continued)

TABLE 1609.6.2.1(1)—continued
MAIN WINDFORCE-RESISTING SYSTEM LOADS FOR A BUILDING WITH MEAN ROOF HEIGHT OF 30 FEET LOCATED IN EXPOSURE B[a] (psf)

BASIC WIND SPEED V (mph–3-second gust)	LOAD DIRECTION	ROOF ANGLE	HORIZONTAL LOADS[b]				VERTICAL LOADS						MAXIMUM HORIZONTAL WALL LOADS[d]			
			End zone		Interior zone		End zone		Interior zone		Windward overhang		Zone			
			Wall	Roof[c]	Wall	Roof[c]	Windward roof[e]	Leeward roof	Windward roof	Leeward roof	End zone	Interior zone	1E	4E	1	4
130	Transverse	0 to 5°	26.8	-13.9	17.8	-8.2	-32.3	-18.3	-22.4	-14.2	-45.1	-35.3	20.4	-15.7	14.9	-12.1
		20°	37.1	-9.8	24.7	-5.4	-32.2	-22.4	-22.4	-17.0	-45.1	-35.3	25.2	-21.1	18.3	-15.7
		30° < angle ≤ 45°	30.1	20.6	24.0	16.5	11.6	-18.3	10.0	-15.7	-10.6	-12.1	22.4	-17.0	19.1	-14.2
	Longitudinal	All angles	26.8	-13.9	17.8	-8.2	-32.2	-18.3	-22.4	-14.2	-45.1	-35.3	20.4	-15.7	14.9	-12.1
140	Transverse	0 to 5°	31.1	-16.1	20.6	-9.6	-37.3	-21.2	-26.0	-16.4	-52.3	-40.9	23.6	-18.2	17.3	-14.0
		20°	43.0	-11.4	28.7	-6.3	-37.3	-26.0	-26.0	-19.7	-52.3	-40.9	29.3	-24.5	21.2	-18.2
		30° < angle ≤ 45°	35.0	23.9	27.8	19.1	13.4	-21.2	11.7	-18.2	-12.3	-14.0	26.0	-19.7	22.1	-16.4
	Longitudinal	All angles	31.1	-16.1	20.6	-9.6	-37.3	-21.2	-26.0	-16.4	-52.3	-40.9	23.6	-18.2	17.3	-14.0
145	Transverse	0 to 5°	33.3	-17.3	22.1	-10.3	-40.1	-22.8	-27.9	-17.6	-56.1	-43.9	25.3	-19.6	18.6	-15.1
		20°	46.2	-12.2	30.8	-6.7	-40.1	-27.9	-27.9	-21.2	-56.1	-43.9	31.4	-26.3	22.8	-19.6
		30° < angle ≤ 45°	37.5	25.6	29.8	20.5	14.4	-22.8	12.5	-19.6	-13.1	-15.1	27.9	-21.2	23.7	-17.6
	Longitudinal	All angles	33.3	-17.3	22.1	-10.3	-40.1	-22.8	-27.9	-17.6	-56.1	-43.9	25.3	-19.6	18.6	-15.1
150	Transverse	0 to 5°	35.7	-18.5	23.7	-11.0	-42.9	-24.4	-29.8	-18.9	-60.0	-47.0	27.1	-20.9	19.9	-16.1
		20°	49.4	-13.0	32.9	-7.2	-42.9	-29.8	-29.8	-22.6	-60.0	-47.0	33.6	-28.1	24.4	-20.9
		30° < angle ≤ 45°	40.1	27.4	31.9	22.0	15.4	-24.4	13.4	-20.9	-14.1	-16.1	29.8	-22.6	25.4	-18.9
	Longitudinal	All angles	35.7	-18.5	23.7	-11.0	-42.9	-24.4	-29.8	-18.9	-60.0	-47.0	27.1	-20.9	19.9	-16.1
170	Transverse	0 to 5°	45.8	-23.8	30.4	-14.1	-55.1	-31.3	-38.3	-24.2	-77.1	-60.4	34.8	-26.9	25.6	-20.7
		20°	63.4	-16.7	42.3	-9.3	-55.1	-38.3	-38.3	-29.1	-77.1	-60.4	43.2	-36.1	31.3	-26.9
		30° < angle ≤ 45°	51.5	35.2	41.0	28.2	19.8	-31.3	17.2	-26.9	-18.1	-20.7	38.3	-29.1	32.6	-24.2
	Longitudinal	All angles	45.8	-23.8	30.4	-14.1	-55.1	-31.3	-38.3	-24.2	-77.1	-60.4	34.8	-26.9	25.6	-20.7

For SI: 1 foot = 304.8 mm, 1 degree = 0.01745 rad.

a. Presssures for roof angles between 5° and 20° and between 20° and 30° shall be interpolated from the table.

b. Pressures are the sum of the windward and leeward pressures and shall be applied to the windward elevation of the building in accordance with Figure 1609.6(3).

c. If pressure is less than 0, use 0.

d. "Max. Horizontal Wall Loads" are only for the design of wall elements which also support roof framing. As part of the MWFRS, these elements shall be designed for the interaction of vertical and horizontal loads or have independent mechanisms for each load. For interaction design of walls as MWFRS, the vertical roof loads shall be the "Vertical Loads" from Table 1609.6.2.1(1), and the horizontal loads shall be the "Max. Horizontal Wall Loads." The zone loads shall be applied as shown in Figure 1609.6(1) and as follows: 1E to the Windward Wall End Zone, 4E to the Leeward Wall End Zone, 1 to the Windward Wall Interior Zone, and 4 to the Leeward Wall Interior Zone.

e. Note that there are two load conditions between 20° and 30°. Negative pressure from 20° to 30° shall be interpolated using a pressure value of 0 for 30°. Positive pressures between 25° and 30° shall be interpolated using a pressure value of 0 for 25°.

TABLE 1609.6.2.1(2)
COMPONENT AND CLADDING LOADS FOR A BUILDING WITH A MEAN ROOF HEIGHT OF 30 FEET LOCATED IN EXPOSURE B[a] (psf)

	ZONE PER FIGURE 1609.6(2)	EFFECTIVE WIND AREA[a] (ft^2)	BASIC WIND SPEED V (mph–3-second gust)																					
			85		90		100		105		110		120		125		130		140		145		150		170	
Roof > 0 to 10 Degrees	1	10	10.0	-13.0	10.0	-14.6	10.0	-18.0	10.0	-19.8	10.0	-21.8	10.5	-25.9	11.4	-28.1	12.4	-30.4	14.3	-35.3	15.4	-37.8	16.5	-40.5	21.1	-52.0
	1	20	10.0	-12.7	10.0	-14.2	10.0	-17.5	10.0	-19.3	10.0	-21.2	10.0	-25.2	10.7	-27.4	11.6	-29.6	13.4	-34.4	14.4	-36.9	15.4	-39.4	19.8	-50.7
	1	50	10.0	-12.2	10.0	-13.7	10.0	-16.9	10.0	-18.7	10.0	-20.5	10.0	-24.4	10.0	-26.4	10.6	-28.6	12.3	-33.2	13.1	-35.6	14.1	-38.1	18.1	-48.9
	1	100	10.0	-11.9	10.0	-13.3	10.0	-16.5	10.0	-18.2	10.0	-19.9	10.0	-23.7	10.0	-25.7	10.0	-27.8	11.4	-32.3	12.2	-34.6	13.0	-37.0	16.7	-47.6
	2	10	10.0	-21.8	10.0	-24.4	10.0	-30.2	10.0	-33.3	10.0	-36.5	10.5	-43.5	11.4	-47.2	12.4	-51.0	14.3	-59.2	15.4	-63.5	16.5	-67.9	21.1	-87.2
	2	20	10.0	-19.5	10.0	-21.8	10.0	-27.0	10.0	-29.7	10.0	-32.6	10.0	-38.8	10.7	-42.1	11.6	-45.6	13.4	-52.9	14.4	-56.7	15.4	-60.7	19.8	-78.0
	2	50	10.0	-16.4	10.0	-18.4	10.0	-22.7	10.0	-25.1	10.0	-27.5	10.0	-32.7	10.0	-35.5	10.6	-38.4	12.3	-44.5	13.1	-47.8	14.1	-51.1	18.1	-65.7
	2	100	10.0	-14.1	10.0	-15.8	10.0	-19.5	10.0	-21.5	10.0	-23.6	10.0	-28.1	10.0	-30.5	10.0	-33.0	11.4	-38.2	12.2	-41.0	13.0	-43.9	16.7	-56.4
	3	10	10.0	-32.8	10.0	-36.8	10.0	-45.4	10.0	-50.1	10.0	-55.0	10.5	-65.4	11.4	-71.0	12.4	-76.8	14.3	-89.0	15.4	-95.5	16.5	102.2	21.1	-131.3
	3	20	10.0	-27.2	10.0	-30.5	10.0	-37.6	10.0	-41.5	10.0	-45.5	10.0	-54.2	10.7	-58.8	11.6	-63.6	13.4	-73.8	14.4	-79.1	15.4	-84.7	19.8	-108.7
	3	50	10.0	-19.7	10.0	-22.1	10.0	-27.3	10.0	-30.1	10.0	-33.1	10.0	-39.3	10.0	-42.7	10.6	-46.2	12.3	-53.5	13.1	-57.4	14.1	-61.5	18.1	-78.9
	3	100	10.0	-14.1	10.0	-15.8	10.0	-19.5	10.0	-21.5	10.0	-23.6	10.0	-28.1	10.0	-30.5	10.0	-33.0	11.4	-38.2	12.2	-41.0	13.0	-43.9	16.7	-56.4
Roof > 10 to 30 Degrees	1	10	10.0	-11.9	10.0	-13.3	10.4	-16.5	11.4	-18.2	12.5	-19.9	14.9	-23.7	16.2	-25.7	17.5	-27.8	20.3	-32.3	21.8	34.6	23.3	-37.0	30.0	-47.6
	1	20	10.0	-11.6	10.0	-13.0	10.0	-16.0	10.4	-17.6	11.4	-19.4	13.6	-23.0	14.8	-25.0	16.0	-27.0	18.5	-31.4	19.9	-33.7	21.3	-36.0	27.3	-46.3
	1	50	10.0	-11.1	10.0	-12.5	10.0	-15.4	10.0	-17.0	10.0	-18.6	11.9	-22.2	12.9	-24.1	13.9	-26.0	16.1	-30.2	17.3	-32.4	18.5	-34.6	23.8	-44.5
	1	100	10.0	-10.8	10.0	-12.1	10.0	-14.9	10.0	-16.5	10.0	-18.1	10.5	-21.5	11.4	-23.3	12.4	-25.2	14.3	-29.3	15.4	-31.4	16.5	-33.6	21.1	-43.2
	2	10	10.0	-25.1	10.0	-28.2	10.4	-34.8	11.4	-38.3	12.5	-42.1	14.9	-50.1	16.2	-54.3	17.5	-58.7	20.3	-68.1	21.8	-73.1	23.3	-78.2	30.0	-100.5
	2	20	10.0	-22.8	10.0	-25.6	10.0	-31.5	10.4	-34.8	11.4	-38.2	13.6	-45.4	14.8	-49.3	16.0	-53.3	18.5	-61.8	19.9	-66.3	21.3	-71.0	27.3	-91.2
	2	50	10.0	-19.7	10.0	-22.1	10.0	-27.3	10.0	-30.1	10.0	-33.0	11.9	-39.3	12.9	-42.7	13.9	-46.1	16.1	-53.5	17.3	-57.4	18.5	-61.4	23.8	-78.9
	2	100	10.0	-17.4	10.0	-19.5	10.0	-24.1	10.0	-26.6	10.0	-29.1	10.5	-34.7	11.4	-37.6	12.4	-40.7	14.3	-47.2	15.4	-50.6	16.5	-54.2	21.1	-69.6
	3	10	10.0	-25.1	10.0	-28.2	10.4	-34.8	11.4	-38.3	12.5	-42.1	14.9	-50.1	16.2	-54.3	17.5	-58.7	20.3	-68.1	21.8	-73.1	23.3	-78.2	30.0	-100.5
	3	20	10.0	-22.8	10.0	-25.6	10.0	-31.5	10.4	-34.8	11.4	-38.2	13.6	-45.4	14.8	-49.3	16.0	-53.5	18.5	-61.8	19.9	-66.3	21.3	-71.0	27.3	-91.2
	3	50	10.0	-19.7	10.0	-22.1	10.0	-27.3	10.0	-30.1	10.0	-33.0	11.9	-39.3	12.9	-42.7	13.9	-46.1	16.1	-53.5	17.3	-57.4	18.5	-61.4	23.8	-78.9
	3	100	10.0	-17.4	10.0	-19.5	10.0	-24.1	10.0	-26.6	10.0	-29.1	10.5	-34.7	11.4	-37.6	12.4	-40.7	14.3	-47.2	15.4	-50.6	16.5	-54.2	21.1	-69.6
Roof > 30 to 45 Degrees	1	10	11.9	-13.0	13.3	-14.6	16.5	-18.0	18.2	-19.8	19.9	-21.8	23.7	-25.9	25.7	-28.1	27.8	-30.4	32.3	-35.3	34.6	-37.8	37.0	-40.5	47.6	-52.0
	1	20	11.6	-12.3	13.0	-13.8	16.0	-17.1	17.6	-18.8	19.4	-20.7	23.0	-24.6	25.0	-26.7	27.0	-28.9	31.4	-33.5	33.7	-35.9	36.0	-38.4	46.3	-49.3
	1	50	11.1	-11.5	12.5	-12.8	15.4	-15.9	17.0	-17.5	18.6	-19.2	22.2	-22.8	24.1	-24.8	26.0	-26.8	30.2	-31.1	32.4	-33.3	34.6	-35.7	44.5	-45.8
	1	100	10.8	-10.8	12.1	-12.1	14.9	-14.9	16.5	-16.5	18.1	-18.1	21.5	-21.5	23.3	-23.3	25.2	-25.2	29.3	-29.3	31.4	-31.4	33.6	-33.6	43.2	-43.2
	2	10	11.9	-15.2	13.3	-17.0	16.5	-21.0	18.2	-23.2	19.9	-25.5	23.7	-30.3	25.7	-32.9	27.8	-35.6	32.3	-41.2	34.6	-44.2	37.0	-47.3	47.6	-60.8
	2	20	11.6	-14.5	13.0	-16.3	16.0	-20.1	17.6	-22.2	19.4	-24.3	23.0	-29.0	25.0	-31.4	27.0	-34.0	31.4	-39.4	33.7	-42.3	36.0	-45.3	46.3	-58.1
	2	50	11.1	-13.7	12.5	-15.3	15.4	-18.9	17.0	-20.8	18.6	-22.9	22.2	-27.2	24.1	-29.5	26.0	-32.0	30.2	-37.1	32.4	-39.8	34.6	-42.5	44.5	-54.6
	2	100	10.8	-13.0	12.1	-14.6	14.9	-18.0	16.5	-19.8	18.1	-21.8	21.5	-25.9	23.3	-28.1	25.2	-30.4	29.3	-35.3	31.4	-37.8	33.6	-40.5	43.2	-52.0
	3	10	11.9	-15.2	13.3	-17.0	16.5	-21.0	18.2	-23.2	19.9	-25.5	23.7	-30.3	25.7	-32.9	27.8	-35.6	32.3	-41.2	34.6	-44.2	37.0	-47.3	47.6	-60.8
	3	20	11.6	-14.5	13.0	-16.3	16.0	-20.1	17.6	-22.2	19.4	-24.3	23.0	-29.0	25.0	-31.4	27.0	-34.0	31.4	-39.4	33.7	-42.3	36.0	-45.3	46.3	-58.1
	3	50	11.1	-13.7	12.5	-15.3	15.4	-18.9	17.0	-20.8	18.6	-22.9	22.2	-27.2	24.1	-29.5	26.0	-32.0	30.2	-37.1	32.4	-39.8	34.6	-42.5	44.5	-54.6
	3	100	10.8	-13.0	12.1	-14.6	14.9	-18.0	16.5	-19.8	18.1	-21.8	21.5	-25.9	23.3	-28.1	25.2	-30.4	29.3	-35.3	31.4	-37.8	33.6	-40.5	43.2	-52.0
Wall	4	10	13.0	-14.1	14.6	-15.8	18.0	-19.5	19.8	-21.5	21.8	-23.6	25.9	-28.1	28.1	-30.5	30.4	-33.0	35.3	-38.2	37.8	-41.0	40.5	-43.9	52.0	-56.4
	4	20	12.4	-13.5	13.9	-15.1	17.2	-18.7	18.9	-20.6	20.8	-22.6	24.7	-26.9	26.8	-29.2	29.0	-31.6	33.7	-36.7	36.1	-39.3	38.7	-42.1	49.6	-54.1
	4	50	11.6	-12.7	13.0	-14.3	16.1	-17.6	17.8	-19.4	19.5	-21.3	23.2	-25.4	25.2	-27.5	27.2	-29.8	31.6	-34.6	33.9	-37.1	36.2	-39.7	46.6	-51.0
	4	100	11.1	-12.2	12.4	-13.6	15.3	-16.8	16.9	-18.5	18.5	-20.4	22.0	-24.2	23.9	-26.3	25.9	-28.4	30.0	-33.0	32.2	-35.4	34.4	-37.8	44.2	-48.6
	5	10	13.0	-17.4	14.6	-19.5	18.0	-24.1	19.8	-26.6	21.8	-29.1	25.9	-34.7	28.1	-37.6	30.4	-40.7	35.3	-47.2	37.8	-50.6	40.5	-54.2	52.0	-69.6
	5	20	12.4	-16.2	13.9	-18.2	17.2	-22.5	18.9	-24.8	20.8	-27.2	24.7	-32.4	26.8	-35.1	29.0	-38.0	33.7	-44.0	36.1	-47.2	38.7	-50.5	49.6	-64.9
	5	50	11.6	-14.7	13.0	-16.5	16.1	-20.3	17.8	-22.4	19.5	-24.6	23.2	-29.3	25.2	-31.8	27.2	-34.3	31.6	-39.8	33.9	-42.7	36.2	-45.7	46.6	-58.7
	5	100	11.1	-13.5	12.4	-15.1	15.3	-18.7	16.9	-20.6	18.5	-22.6	22.0	-26.9	23.9	-29.2	25.9	-31.6	30.0	-36.7	32.2	-39.3	34.4	-42.1	44.2	-54.1

For SI: 1 foot = 304.8 mm, 1 mile per hour = 0.44 m/s, 1 degree = 0.01745 rad.

a. For effective areas between those given above, the load is permitted to be interpolated; otherwise, use the load associated with the lower effective area.

TABLE 1609.6.2.1(3)
ROOF OVERHANG COMPONENT AND CLADDING DESIGN WIND PRESSURES FOR A BUILDING WITH A MEAN ROOF HEIGHT OF 30 FEET LOCATED IN EXPOSURE B[a] (psf)

	ZONE PER FIGURE 1609.6(2)	EFFECTIVE WIND AREA[a] (ft^2)	BASIC WIND SPEED V (mph–3-second gust)										
			90	100	105	110	120	125	130	140	145	150	170
Roof > 0 to 10 Degrees	2	10	-21.0	-25.9	-28.6	-31.4	-37.3	-40.5	-43.8	-50.8	-54.5	-58.3	-74.9
	2	20	-20.6	-25.5	-28.1	-30.8	-36.7	-39.8	-43.0	-49.9	-53.5	-57.3	-73.6
	2	50	-20.1	-24.9	-27.4	-30.1	-35.8	-38.8	-42.0	-48.7	-52.2	-55.9	-71.8
	2	100	-19.8	-24.4	-26.9	-29.5	-35.1	-38.1	-41.2	-47.8	-51.3	-54.9	-70.5
	3	10	-34.6	-42.7	-47.1	-51.6	-61.5	-66.7	-72.1	-83.7	-89.7	-96.0	-123.4
	3	20	-27.1	-33.5	-36.9	-40.5	-48.3	-52.4	-56.6	-65.7	-70.4	-75.4	-96.8
	3	50	-17.3	-21.4	-23.6	-25.9	-30.8	-33.4	-36.1	-41.9	-44.9	-48.1	-61.8
	3	100	-10.0	-12.2	-13.4	-14.8	-17.6	-19.1	-20.6	-23.9	-25.6	-27.4	-35.2
Roof > 10 to 30 Degrees	2	10	-27.2	-33.5	-37.0	-40.6	-48.3	-52.4	-56.7	-65.7	-70.5	-75.5	-96.9
	2	20	-27.2	-33.5	-37.0	-40.6	-48.3	-52.4	-56.7	-65.7	-70.5	-75.5	-96.9
	2	50	-27.2	-33.5	-37.0	-40.6	-48.3	-52.4	-56.7	-65.7	-70.5	-75.5	-96.9
	2	100	-27.2	-33.5	-37.0	-40.6	-48.3	-52.4	-56.7	-65.7	-70.5	-75.5	-96.9
	3	10	-45.7	-56.4	-62.2	-68.3	-81.2	-88.1	-95.3	-110.6	-118.6	-126.9	-163.0
	3	20	-40.5	-50.0	-55.1	-60.5	-72.0	-78.1	-84.5	-98.0	-105.1	-112.5	-144.4
	3	50	-33.6	-41.5	-45.7	-50.2	-59.7	-64.8	-70.1	-81.3	-87.2	-93.3	-119.9
	3	100	-28.4	-35.1	-38.7	-42.4	-50.5	-54.8	-59.3	-68.7	-73.7	-78.9	-101.3
Roof > 30 to 45 Degrees	2	10	-24.7	-30.5	-33.6	-36.9	-43.9	-47.6	-51.5	-59.8	-64.1	-68.6	-88.1
	2	20	-24.0	-29.6	-32.6	-35.8	-42.6	-46.2	-50.0	-58.0	-62.2	-66.5	-85.5
	2	50	-23.0	-28.4	-31.3	-34.3	-40.8	-44.3	-47.9	-55.6	-59.6	-63.8	-82.0
	2	100	-22.2	-27.4	-30.3	-33.2	-39.5	-42.9	-46.4	-53.8	57.7	-61.7	-79.3
	3	10	-24.7	-30.5	-33.6	-36.9	-43.9	-47.6	-51.5	-59.8	-64.1	-68.6	-88.1
	3	20	-24.0	-29.6	-32.6	-35.8	-42.6	-46.2	-50.0	-58.0	-62.2	-66.5	-85.5
	3	50	-23.0	-28.4	-31.3	-34.3	-40.8	-44.3	-47.9	-55.6	-59.6	-63.8	-82.0
	3	100	-22.2	-27.4	-30.3	-33.2	-39.5	-42.9	-46.4	-53.8	-57.7	-61.7	-79.3

For SI: 1 foot = 304.8 mm, 1 degree = 0.01745 rad, 1 mile per hour = 0.44 m/s.

a. For effective areas between those given above, the load is permitted to be interpolated; otherwise, use the load associated with the lower effective area.

TABLE 1609.6.2.1(4)
HEIGHT AND EXPOSURE ADJUSTMENT COEFFICIENTS[a]

MEAN ROOF (feet)	EXPOSURE		
	B	C	D
15	1.00	1.21	1.47
20	1.00	1.29	1.55
25	1.00	1.35	1.61
30	1.00	1.40	1.66
35	1.05	1.45	1.70
40	1.09	1.49	1.74
45	1.12	1.53	1.78
50	1.16	1.56	1.81
55	1.19	1.59	1.84
60	1.22	1.62	1.87

For SI: 1 foot = 304.8 mm.

a. All table values shall be adjusted for other exposures and heights by multiplying by the above coefficients.

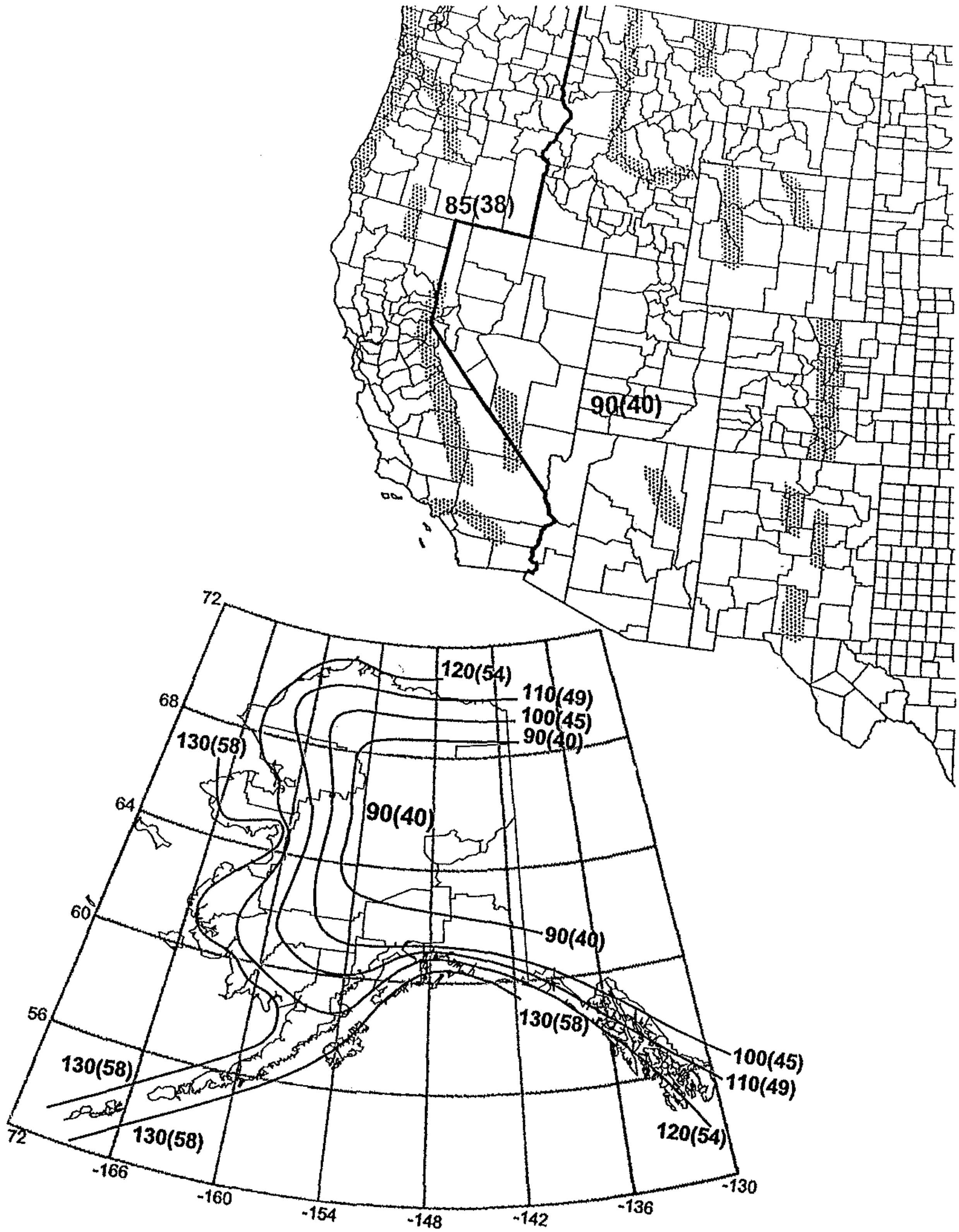
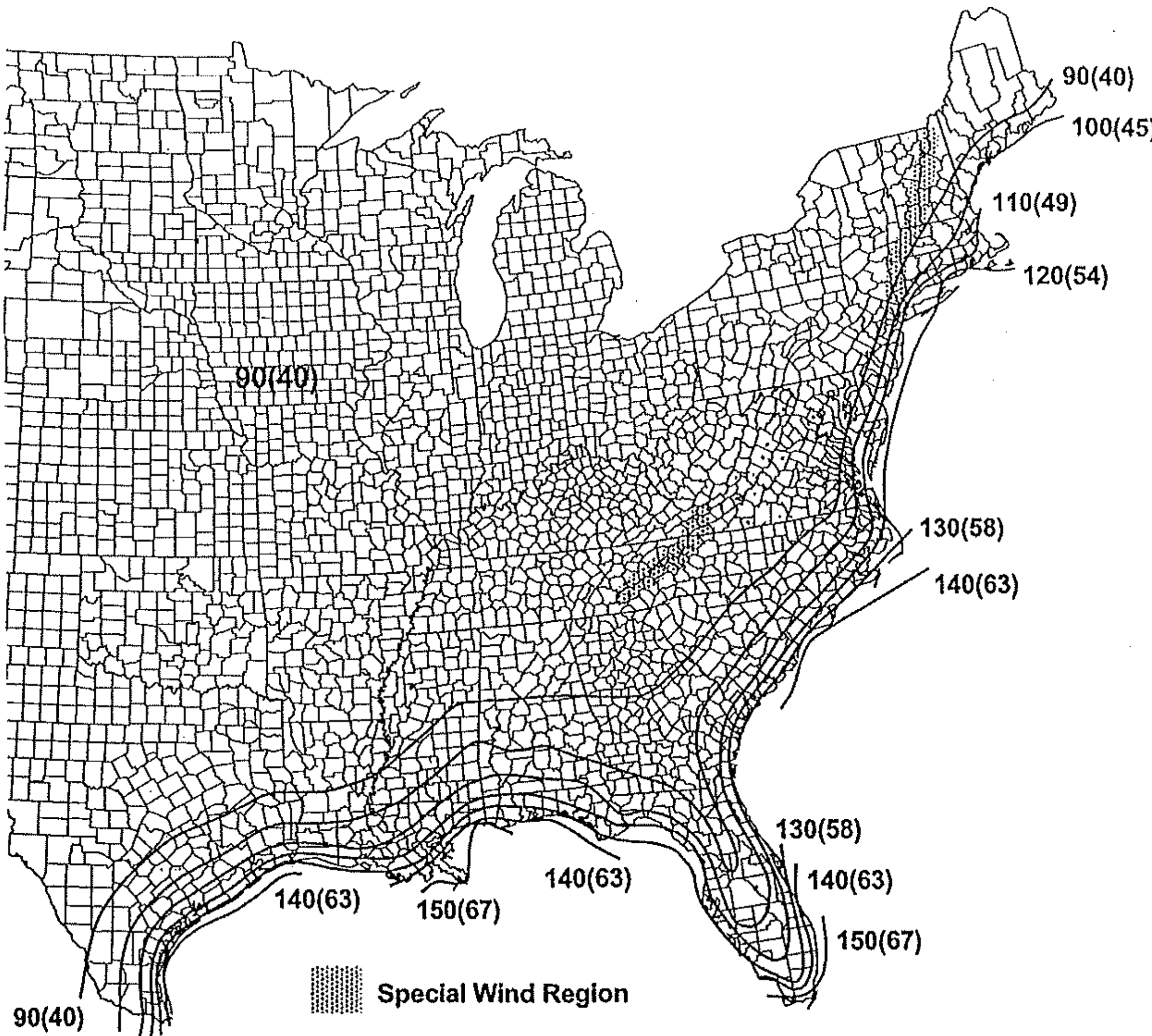


FIGURE 1609
BASIC WIND SPEED (3-SECOND GUST)



Location	V mph	(m/s)
Hawaii	105	(47)
Puerto Rico	145	(65)
Guam	170	(76)
Virgin Islands	145	(65)
American Samoa	125	(56)

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10 m) above ground for Exposure C category.
2. Linear interpolation between wind 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.

FIGURE 1609—continued
BASIC WIND SPEED (3-SECOND GUST)

1609.6.4 Main windforce-resisting system (MWFRS). All elements and connections of the MWFRS shall be designed for vertical and horizontal loads based on the combined leeward and windward wall pressures and roof pressures determined from Table 1609.6.2.1(1). Pressures shall be applied in accordance with the loading diagrams shown in Figure 1609.6(3) to the end zone and interior zone as shown in Figure 1609.6(1). The building shall be designed for all wind directions. For buildings having flat roofs a ridge line normal to the wind direction shall be assumed at the mid-length dimension of the roof for all directions considered. Each corner shall be considered in turn as the windward corner.

1609.6.4.1 Overhang loads. The pressures to be used for the effects of roof overhangs on MWFRS shall be taken from Table 1609.6.2.1(1) and includes the effect of the wind on both the bottom and top surfaces.

1609.6.5 Components and cladding. Pressure for wind-loading actions on components and cladding shall be determined from Table 1609.6.2.1(2) for enclosed portions of the building and Table 1609.6.2.1(3) for overhangs, based on the effective area for the element under consideration. The pressures in Table 1609.6.2.1(3) include internal pressure. The pressure shall be applied in accordance with the loading diagrams in Figure 1609.6(2).

1609.7 Roof systems.

1609.7.1 Roof deck. The roof deck shall be designed to withstand the wind pressures determined under either the provisions of Section 1609.6 for buildings with a mean roof height not exceeding 60 feet (18 288 mm) or Section 1609.1.1 for buildings of any height.

1609.7.2 Roof coverings. Roof coverings shall comply with Section 1609.7.1.

Exception: Rigid tile roof coverings that are air-permeable and installed over a roof deck complying with Section 1609.7.1 are permitted to be designed in accordance with Section 1609.7.3.

1609.7.3 Rigid tile. Wind loads on rigid tile roof coverings shall be determined in accordance with the following equation:

$$M_a = q_h C_L b L L_a [1.0 - Gc_p] \qquad \textbf{(Equation 16-14)}$$

For SI: $M_a = \dfrac{q_h C_L b L L_a [1.0 - Gc_p]}{1000}$

where:

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

C_L = Lift coefficient. The lift coefficient for concrete and clay tile shall be 0.2 or shall be determined by test in accordance with Section 1715.2.

GC_p = Roof pressure coefficient for each applicable roof zone determined from Section 6 of ASCE 7. Roof coefficients shall not be adjusted for internal pressure.

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

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

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

q_h = Wind velocity pressure psf (kN/m^2) determined from Section 6.5.10 of ASCE 7.

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

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

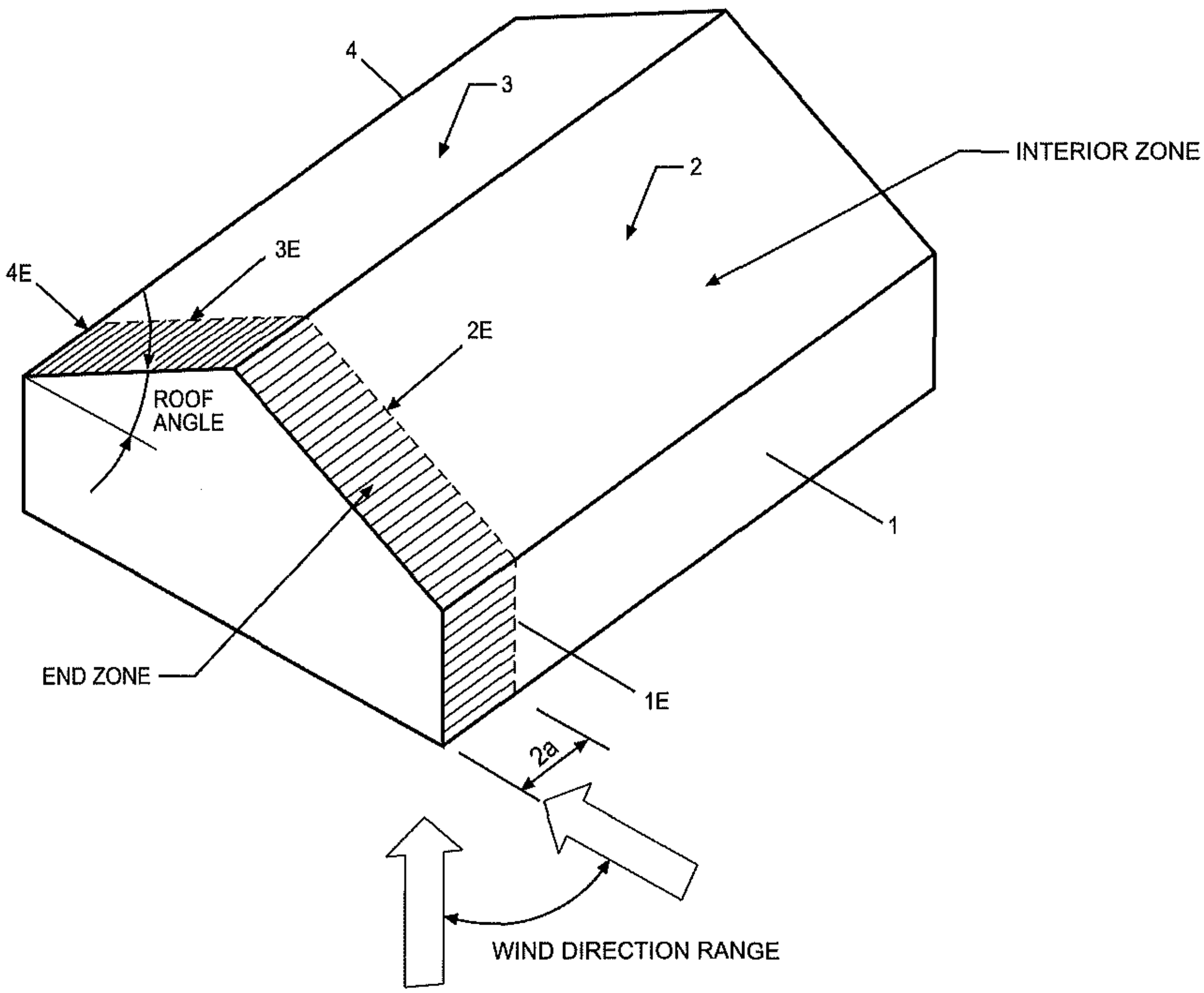
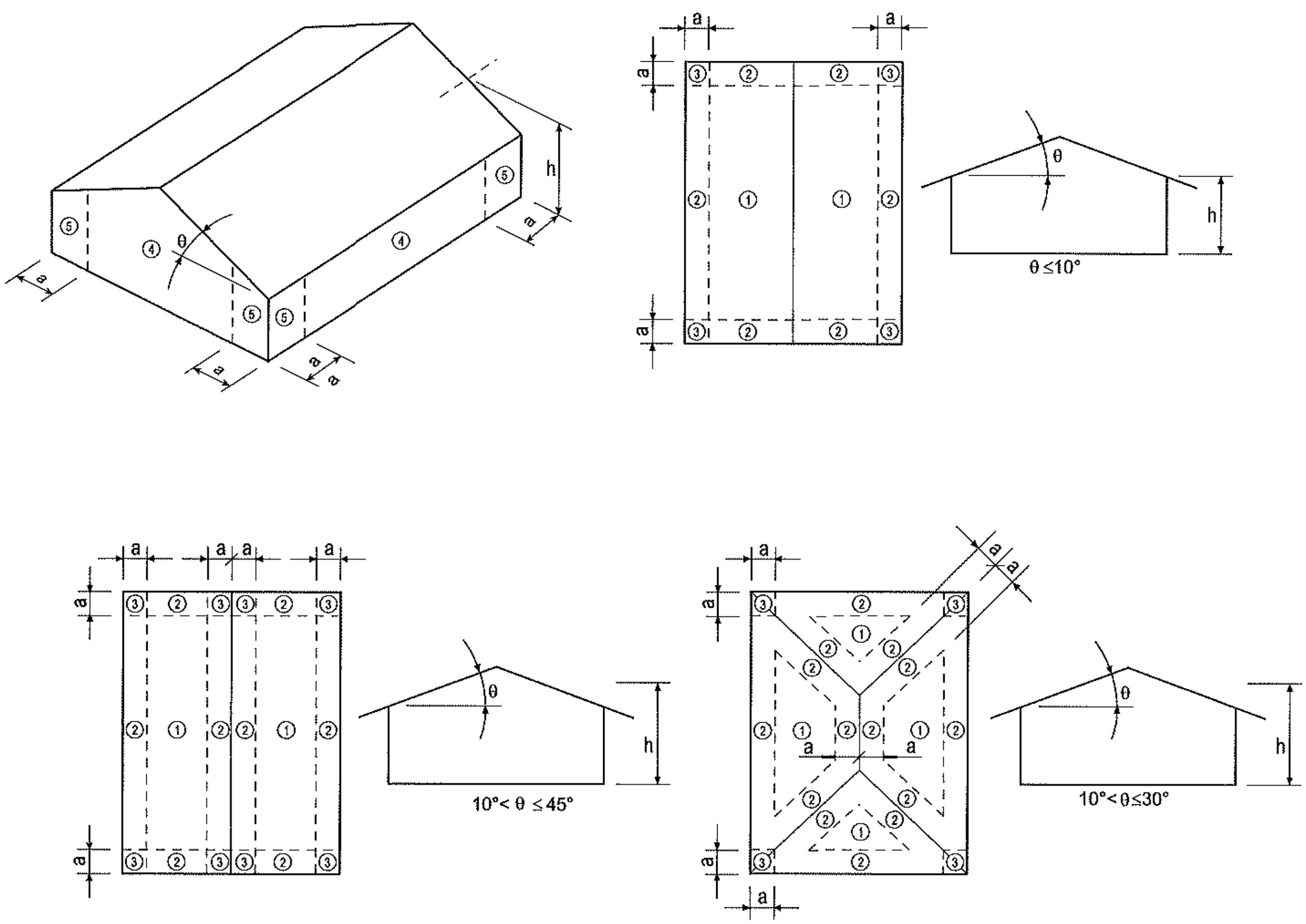
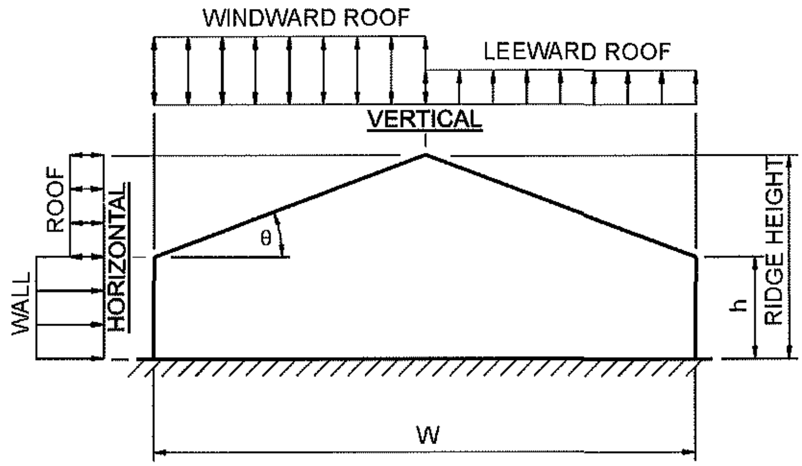


FIGURE 1609.6(1)
MAIN WINDFORCE (MWF) LOADING DIAGRAM

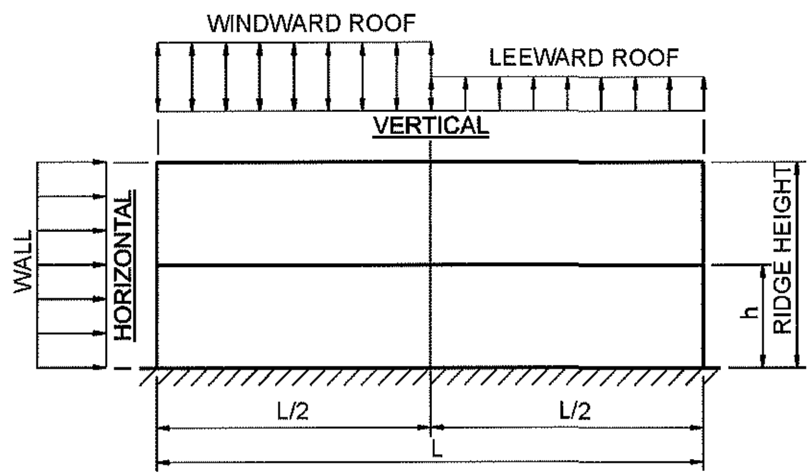


For SI: 1 degree = 0.01745 rad.

FIGURE 1609.6(2)
COMPONENT AND CLADDING LOADING DIAGRAMS



TRANSVERSE ELEVATION



LONGITUDINAL ELEVATION

FIGURE 1609.6(3)
APPLICATION OF MAIN WINDFORCE-RESISTING SYSTEM (MWFRS) LOADS FOR SIMPLE DIAPHRAGM BUILDINGS

SECTION 1610
SOIL LATERAL LOAD

1610.1 General. Basement and retaining walls shall be designed to resist lateral soil loads. Soil loads specified in Table 1610.1 shall be used as the minimum design lateral soil loads unless specified otherwise in a soil investigation report approved by the building official. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure load. Design lateral pressure shall be increased if soils with expansion potential are present at the site.

1610.2 Retaining walls. Retaining walls shall be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift. Retaining walls shall be designed for a safety factor of 1.5 against lateral sliding and overturning.

SECTION 1611
RAIN LOADS

1611.1 Design rain loads. Each portion of a roof shall be designed to sustain the load of rainwater that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow.

$$R = 5.2\,(d_s + d_h) \qquad \textbf{(Equation 16-15)}$$

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

TABLE 62.1610
SOIL LATERAL LOAD

DESCRIPTION OF BACKFILL MATERIAL[a]	UNIFIED SOIL CLASSIFICATION	ACTIVE CONDITION[b] DESIGN LATERAL SOIL LOAD (psf per foot of depth)	AT-REST CONDITION[c] DESIGN LATERAL SOIL LOAD (psf per foot of depth)
Well-graded clean gravels; gravel & sand mixes	GW	30	50
Poorly graded clean gravels; gravel & sand mixes	GP	30	50
Silty gravel, poorly graded gravel & sand mixes	GM	40	60
Clayey gravel, poorly graded gravel & clay mixes	GC	45	65
Well-graded clean sand; gravel & sand mixes	SW	30	50
Poorly graded clean sand; sand & gravel mixes	SP	30	50
Silty sands, poorly graded sand & silt mixtures	SM	45	65
Sand-silt-clay mix with plastic fines	SM-SC	45	65
Clayey sand, poorly graded sand & clay mixes	SC	60	100
Inorganic silts and clayey silts	ML	45	100
Mixture of inorganic silt and clay	ML-CL	60	100
Inorganic clays of medium plasticity	CL	60	100
Organic silt and silty clay, low plasticity	OL	Note d	Note d
Inorganic clayey silt, elastic silt	MH	Note d	Note d
Inorganic clays of high plasticity	CH	Note d	Note d
Organic clays and organic silty clay	OH	Note d	Note d

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

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

b. Where wall is expected to deflect a minimum of 0.001 times the retained soil height. Design lateral soil loads are for moist conditions for the specified soil at typical specified compacted densities. Actual field conditions shall govern. The lateral pressure of improperly drained, submerged, or saturated soils shall include the buoyant unit soil weight times appropriate K_a, plus the hydrostatic pressure. K_a is the coefficient of active earth pressure.

c. Where wall is expected to deflect less than 0.001 times the retained soil height. Design lateral soil loads are for moist conditions for the specified soil at typical specified compacted densities. Actual field conditions shall govern. The lateral pressure of improperly drained, submerged, or saturated soils shall include the buoyant unit soil weight times appropriate K_o, plus the hydrostatic pressure. K_o is the coefficient of earth pressure at rest.

d. Unsuitable as backfill material.

where:

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

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

R = Rain load on the undeflected roof, in pounds per square foot (kN/m²). When the phrase "undeflected roof" is used, deflections from loads (including dead loads) shall not be considered when determining the amount of rain on the roof.

1611.2 Ponding instability. Ponding refers to the retention of water due solely to the deflection of relatively flat roofs. Roofs with a slope less than one-fourth unit vertical in 12 units horizontal (2-percent slope) shall be investigated by structural analysis to ensure that they possess adequate stiffness to preclude progressive deflection (i.e., instability) as rain falls on them or meltwater is created from snow on them. The larger of snow load or rain load shall be used in this analysis. The primary drainage system within an area subjected to ponding shall be considered to be blocked in this analysis.

1611.3 Controlled drainage. Roofs equipped with hardware to control the rate of drainage shall be equipped with a secondary drainage system at a higher elevation that limits accumulation of water on the roof above that elevation. Such roofs shall be designed to sustain the load of rainwater that will accumulate on them to the elevation of the secondary drainage system plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow determined from Section 1611.1. Such roofs shall also be checked for ponding instability in accordance with Section 1611.2.

SECTION 1612
FLOOD LOADS
Deleted

SECTION 1613
EARTHQUAKE LOADS DEFINITIONS

1613.1 Definitions. The following words and terms shall, for the purposes of this section, have the meanings shown herein.

ACTIVE FAULT/ACTIVE FAULT TRACE. A fault for which there is an average historic slip rate of 1 mm per year or more and geologic evidence of seismic activity within Holocene (past 11,000 years) times. Active fault traces are designated by the appropriate regulatory agency and/or registered design professional subject to identification by a geologic report.

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

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

BOUNDARY ELEMENTS. Chords and collectors at diaphragm and shear wall edges, interior openings, discontinuities, and re-entrant corners.

BRITTLE. Systems, members, materials and connections that do not exhibit significant energy dissipation capacity in the inelastic range.

COLLECTOR. A diaphragm or shear wall element parallel to the applied load that collects and transfers shear forces to the vertical-force-resisting elements or distributes forces within a diaphragm or shear wall.

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

Component, equipment. A mechanical or electrical component or element that is part of a mechanical and/or electrical system within or without a building system.

Component, flexible. Component, including its attachments, having a fundamental period greater than 0.06 second.

Component, rigid. Component, including its attachments, having a fundamental period less than or equal to 0.06 second.

DESIGN EARTHQUAKE. The earthquake effects that buildings and structures are specifically proportioned to resist in Sections 1613 through 1622.

DESIGNATED SEISMIC SYSTEM. Those architectural, electrical, and mechanical systems and their components that require design in accordance with Section 1621 that have a component importance factor, I_p, greater than one.

DISPLACEMENT

Design Displacement. The design earthquake lateral displacement, excluding additional displacement due to actual and accidental torsion, required for design of the isolation system.

Total Design Displacement. The design earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for design of the isolation system.

Total Maximum Displacement. The maximum considered earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for verification of the stability of the isolation system or elements thereof, design of building separations, and vertical load testing of isolator unit prototype.

DISPLACEMENT RESTRAINT SYSTEM. A collection of structural elements that limits lateral displacement of seismically isolated structures due to the maximum considered earthquake.

EFFECTIVE DAMPING. The value of equivalent viscous damping corresponding to energy dissipated during cyclic response of the isolation system.

EFFECTIVE STIFFNESS. The value of the lateral force in the isolation system, or an element thereof, divided by the corresponding lateral displacement.

HAZARDOUS CONTENTS. A material that is highly toxic or potentially explosive and in sufficient quantity to pose a significant life-safety threat to the general public if an uncontrolled release were to occur.

INVERTED PENDULUM-TYPE STRUCTURES. Structures that have a large portion of their mass concentrated near the top and, thus, have essentially one degree of freedom in horizontal translation. The structures are usually T-shaped with a single column supporting the beams or framing at the top.

ISOLATION INTERFACE. The boundary between the upper portion of the structure, which is isolated, and the lower portion of the structure, which moves rigidly with the ground.

ISOLATION SYSTEM. The collection of structural elements that includes individual isolator units, structural elements that transfer force between elements of the isolation system and connections to other structural elements.

ISOLATOR UNIT. A horizontally flexible and vertically stiff structural element of the isolation system that permits large lateral deformations under design seismic load. An isolator unit may be used either as part of or in addition to the weight-supporting system of the building.

LOAD

Gravity Load (*W*). The total dead load and applicable portions of other loads as defined in Sections 1613 through 1622.

MAXIMUM CONSIDERED EARTHQUAKE. The most severe earthquake effects considered by this code.

NONBUILDING STRUCTURE. A structure, other than a building, constructed of a type included in Section 1622.

OCCUPANCY IMPORTANCE FACTOR. A factor assigned to each structure according to its Seismic Use Group as prescribed in Table 1604.5.

SEISMIC DESIGN CATEGORY. A classification assigned to a structure based on its Seismic Use Group and the severity of the design earthquake ground motion at the site.

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

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

SEISMIC RESPONSE COEFFICIENT. Coefficient, C_s, as determined from Section 1617.4.1.

SEISMIC USE GROUP. A classification assigned to a building based on its use as defined in Section 1616.2.

SHEAR WALL. A wall designed to resist lateral forces parallel to the plane of the wall.

SHEAR WALL-FRAME INTERACTIVE SYSTEM. A structural system that uses combinations of shear walls and frames designed to resist lateral forces in proportion to their rigidities, considering interaction between shear walls and frames on all levels.

SITE CLASS. A classification assigned to a site based on the types of soils present and their engineering properties as defined in Section 1615.1.5.

SITE COEFFICIENTS. The values of, F_a, and, F_v, indicated in Tables 1615.1.2-1 and 1615.1.2-2, respectively.

STORY DRIFT RATIO. The story drift divided by the story height.

TORSIONAL FORCE DISTRIBUTION. The distribution of horizontal seismic forces through a rigid diaphragm when the center of mass of the structure at the level under consideration does not coincide with the center of rigidity (sometimes referred to as a diaphragm rotation).

TOUGHNESS. The ability of a material to absorb energy without losing significant strength.

WIND-RESTRAINT SEISMIC SYSTEM. The collection of structural elements that provides restraint of the seismic- isolated structure for wind loads. The wind-restraint system may be either an integral part of isolator units or a separate device.

SECTION 1614
EARTHQUAKE LOADS—GENERAL

1614.1 [Comm 62.1614(1)] Scope. Every structure, and portion thereof, shall as a minimum, be designed and constructed to resist the effects of earthquake motions and assigned a Seismic Design Category as set forth in Section 1616.3. Structures determined to be in Seismic Design Category A, and the following structures, need only comply with Section 1616.4.

(a) Structures north of the 4% g contour line in IBC Figure 1615(2).

(b) Structures south of the 4% g contour line in IBC Figure 1615(2) that have a site class of A to C in IBC Table 1615.1.1.

(c) Structures south of the 4% g contour line in IBC Figure 1615(2) which are classified as Category IV in IBC Table 1604.5 and which have a site class of D, E or F in IBC Table 1615.1.1.

(2) Structures south of the 4% g contour line in IBC Figure 1615(2) which are classified as Category I, II or III in IBC Table 1604.5 and which have a site class of D, E or F in IBC Table 1615.1.1 shall comply with the applicable design requirements in IBC Sections 1616 through 1623.

Exceptions:

1. Detached Group R-3 dwellings as applicable in Section 101.2 in Seismic Design Categories A, B and C are exempt from requirements of Sections 1613 through 1622.
2. The seismic-force-resisting system of wood frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in Section 1616.1.
3. Agricultural storage structures intended only for incidental human occupancy are exempt from requirements of Sections 1613 through 1622.
4. Structures located where mapped short period spectral response acceleration, S_S, determined in accordance with Section 1615.1 is less than or equal to 0.15g and where the mapped spectral response acceleration at 1-second period, S_1, determined in accordance with Section 1615.1, is less than or equal to 0.04g shall only be required to comply with Section 1616.4.
5. Structures located where the short period design spectral response acceleration, S_{DS}, determined in accordance with Section 1615.1 is less than or equal to 0.167g and the design spectral response acceleration at 1-second period, S_{D1}, determined in accordance with Section 1615.1 is less than or equal to 0.067g, shall only be required to comply with Section 1616.4.

1614.1.1 Additions to existing buildings. An addition that is structurally independent from an existing structure shall be designed and constructed as required for a new structure in accordance with the seismic requirements for new structures. An addition that is not structurally independent from an existing structure shall be designed and constructed such that the entire structure conforms to the seismic force resistance requirements for new structures unless the following conditions are satisfied:

1. The addition conforms with the requirements for new structures; and
2. The addition does not increase the seismic forces in any structural element of the existing structure by more than 5 percent, unless the element has the capac-

ity to resist the increased forces determined in accordance with Sections 1613 through 1622.

1614.2 Change of occupancy. When a change of occupancy results in a structure being reclassified to a higher Seismic Use Group, the structure shall conform to the seismic requirements for a new structure.

> **Exception:** Specific detailing provisions required for a new structure are not required to be met where it can be shown an equivalent level of performance and seismic safety contemplated for a new structure is obtained. Such analysis shall consider the regularity, overstrength, redundancy and ductility of the structure within the context of the specific detailing provided.

1614.3 Alterations. Existing structures being altered need not comply with Sections 1613 through 1622 provided that the following conditions are met:

1. The alterations do not create a structural irregularity as defined in Section 1616.5 or make an existing structural irregularity more severe.
2. The alteration does not increase the seismic forces in any structural element of the existing structure by more than 5 percent, unless the capacity of the element subject to the increased forces is still in compliance with Sections 1613 through 1622.
3. The alteration does not decrease the seismic resistance of any structural element of the existing structure to less than that required for a new structure.
4. The alterations do not result in the creation of an unsafe condition.

1614.4 Quality assurance. A Quality Assurance Plan shall be provided where required by Chapter 17.

1614.5 Seismic and wind. When the code-prescribed wind design produces greater effects, the wind design shall govern, but detailing requirements and limitations prescribed in this and referenced sections shall be followed.

SECTION 1615
EARTHQUAKE LOADS—SITE GROUND MOTION

1615.1 General procedure for determining maximum considered earthquake and design spectral response accelerations. Ground motion accelerations, represented by response spectra and coefficients derived from these spectra, shall be determined in accordance with the general procedure of Section 1615.1 or the site-specific procedure of Section 1615.2. The site-specific procedure of Section 1615.2 shall be used for structures on sites classified as Site Class F, in accordance with Section 1615.1.1.

The mapped maximum considered earthquake spectral response acceleration at short periods, S_S, and at 1-second period, S_1, shall be determined from Figures 1615(1) through (10). Where a site is between contours, straight line interpolation or the value of the higher contour shall be used.

Comm 62.1615 Alternatives to contour lines in IBC Figures 1615(1) and 1615(2).

(1) The contour line in IBC Figure 1615(1) that extends through southern Rock, Walworth, and Kenosha Counties in Wisconsin may be ignored.

(2) The 4% g contour line in IBC Figure 1615(2) may be applied as occurring in the location shown in Figure 62.16–2.

Figure 62.16–2
Alternate 4% g Contour Location

The Site Class shall be determined in accordance with Section 1615.1.1. The maximum considered earthquake spectral response accelerations at short period and 1-second period adjusted for site class effects, S_{MS} and S_{M1}, shall be determined in accordance with Section 1615.1.2. The design spectral response accelerations at short period, S_{DS}, and at 1-second period, S_{D1}, shall be determined in accordance with Section 1615.1.3. The general response spectrum shall be determined in accordance with Section 1615.1.4.

> **Exception:** For structures located on sites with mapped spectral response acceleration at short period, S_S, less than or equal to 0.15g and mapped spectral response acceleration at 1-second period, S_1, less than or equal to 0.04g, the Site Class, maximum considered earthquake spectral response accelerations at short period and at 1-second period adjusted for site class effects (S_{MS} and S_{M1}), and the design spectral response accelerations at short period and at 1-second period (S_{DS} and S_{D1}) need not be determined. Such structures shall be categorized as Seismic Design Category A and need only comply with the requirements of Section 1616.4.

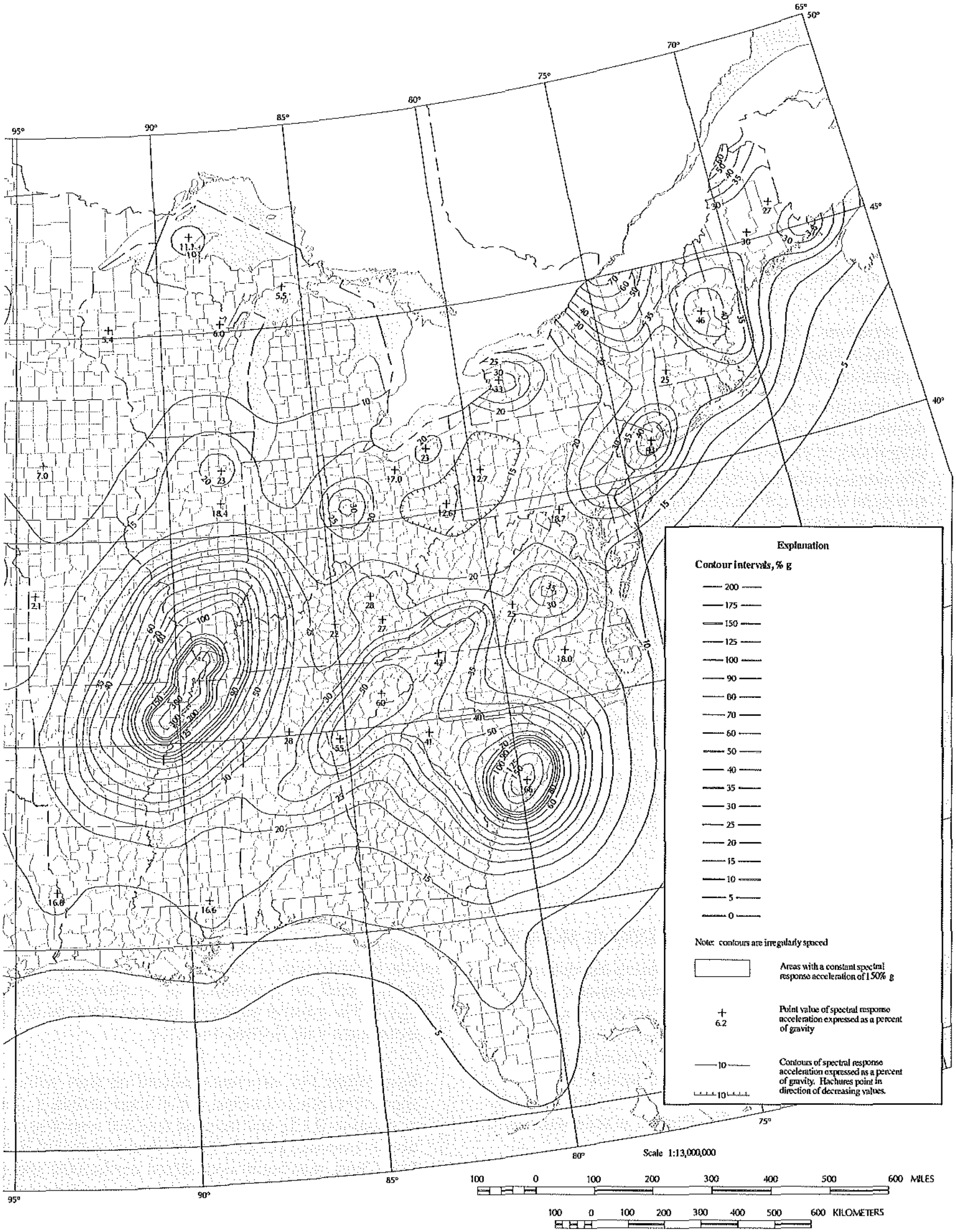


FIGURE 1615(1)
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5 PERCENT OF CRITICAL DAMPING), SITE CLASS B

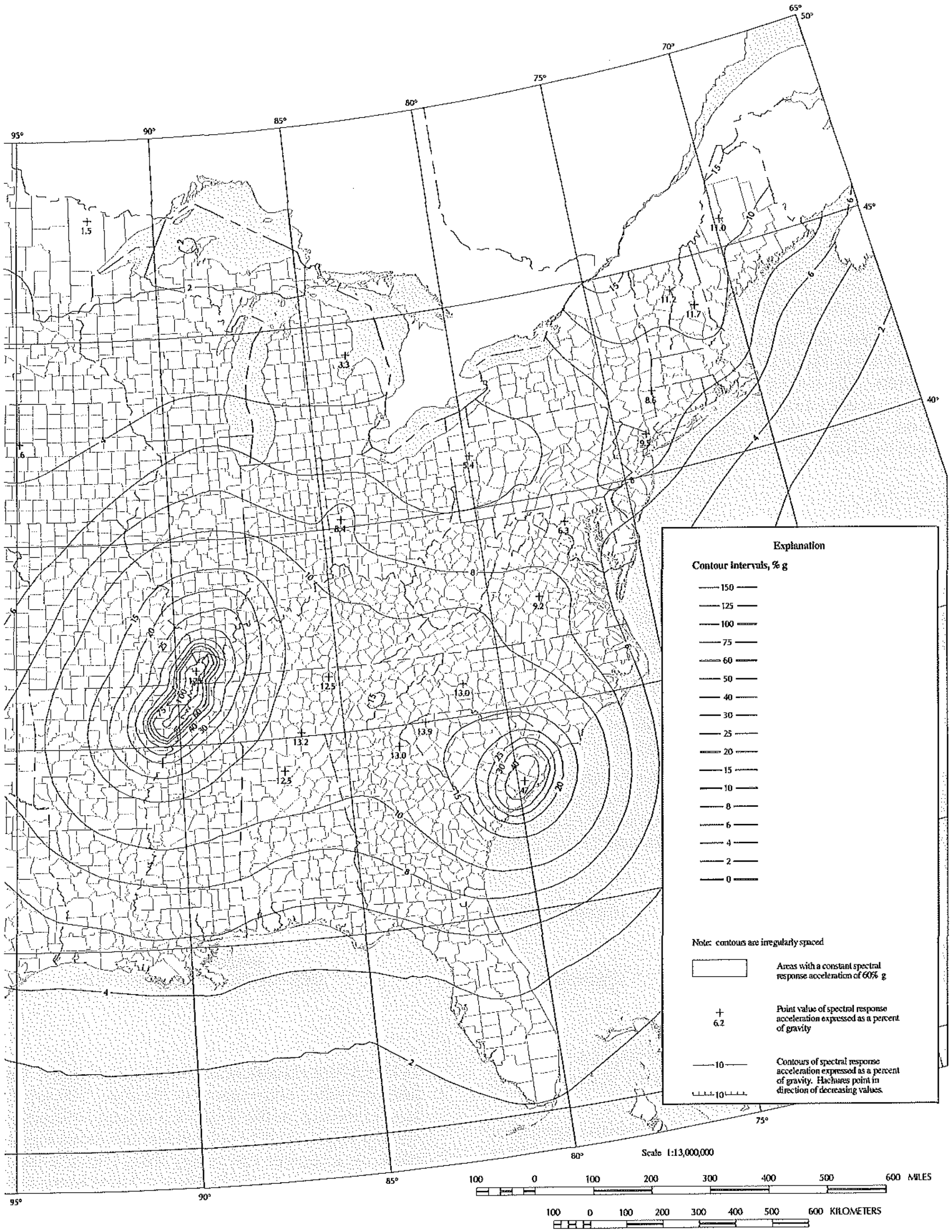


FIGURE 1615(2)
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5 PERCENT OF CRITICAL DAMPING), SITE CLASS B

1615.1.1 Site class definitions. The site shall be classified as one of the site classes defined in Table 1615.1.1. Where the soil shear wave velocity, $\bar{v}_s$, is not known, site class shall be determined, as permitted in Table 1615.1.1, from standard penetration resistance, $\bar{N}$, or from soil undrained shear strength, $\bar{s}_u$, calculated per Section 1615.1.5. Where site-specific data are not available to a depth of 100 feet (30 480 mm), appropriate soil properties are permitted to be estimated by the registered design professional preparing the soils report based on known geologic conditions.

When the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the building official determines that Site Class E or F soil is likely to be present at the site.

1615.1.2 Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. The maximum considered earthquake spectral response acceleration for short periods, S_{MS}, and at 1-second period, S_{M1}, adjusted for site class effects, shall be determined by Equations 16-16 and 16-17, respectively:

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

$$S_{M1} = F_v S_1 \qquad \textbf{(Equation 16-17)}$$

where:

F_a = Site coefficient defined in Table 1615.1.2(1).

F_v = Site coefficient defined in Table 1615.1.2(2).

S_S = The mapped spectral accelerations for short periods as determined in Section 1615.1.

S_1 = The mapped spectral accelerations for a 1-second period as determined in Section 1615.1.

1615.1.3 Design spectral response acceleration parameters. Five-percent damped design spectral response acceleration at short periods, S_{DS}, and at 1 second period, S_{D1}, shall be determined from Equations 16-18 and 16-19, respectively:

$$S_{DS} = \frac{2}{3} S_{MS} \qquad \textbf{(Equation 16-18)}$$

$$S_{D1} = \frac{2}{3} S_{M1} \qquad \textbf{(Equation 16-19)}$$

where:

S_{MS} = The maximum considered earthquake spectral response accelerations for short period as determined in Section 1615.1.2.

S_{M1} = The maximum considered earthquake spectral response accelerations for 1 second period as determined in Section 1615.1.2.

1615.1.4 General procedure response spectrum. The general design response spectrum curve shall be developed as indicated in Figure 1615.1.4 and as follows:

1. For periods less than or equal to T_0, the design spectral response acceleration, S_a, shall be given by Equation 16-20.
2. For periods greater than or equal to T_0 and less than or equal to the T_S, the design spectral response acceleration, S_a, shall be taken equal to S_{DS}.
3. For periods greater than T_S, the design spectral response acceleration, S_a, shall be given by Equation 16-21.

$$S_a = 0.6 \frac{S_{DS}}{T_O} T + 0.4 S_{DS} \qquad \textbf{(Equation 16-20)}$$

$$S_a = \frac{S_{D1}}{T} \qquad \textbf{(Equation 16-21)}$$

where:

S_{DS} = The design spectral response acceleration at short periods as determined in Section 1615.1.3.

S_{D1} = The design spectral response acceleration at 1 second period as determined in Section 1615.1.3.

T = Fundamental period (in seconds) of the structure (Section 1617.4.2).

T_O = $0.2\ S_{D1}/S_{DS}$

T_S = S_{D1}/S_{DS}

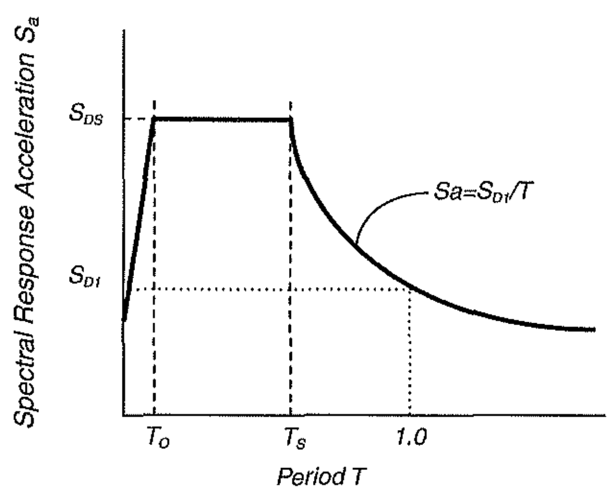


FIGURE 1615.1.4
DESIGN RESPONSE SPECTRUM

1615.1.5 Site classification for seismic design. The notations presented below apply to the upper 100 feet (30 480 mm) of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there are a total of n distinct layers in the upper 100 feet (30 480 mm). The symbol, i, then refers to any one of the layers between 1 and n.

where:

v_{si} = The shear wave velocity in feet per second (m/s).

TABLE 1615.1.1
SITE CLASS DEFINITIONS

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 feet, AS PER SECTION 1615.1.5		
		Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	Not applicable	Not applicable
B	Rock	$2{,}500 < \bar{v}_s \leq 5{,}000$	Not applicable	Not applicable
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \leq 2{,}500$	$\bar{N} > 50$	$\bar{s}_u \geq 2{,}000$
D	Stiff soil profile	$600 \leq \bar{v}_s \leq 1{,}200$	$15 \leq \bar{N} \leq 50$	$1{,}000 \leq \bar{s}_u \leq 2{,}000$
E	Soft soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$\bar{s}_u < 1{,}000$
E	—	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index $PI > 20$; 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ ft)		

For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929 m^2, 1 pound per square foot = 0.0479 kPa.

TABLE 1615.1.2(1)
VALUES OF SITE COEFFICIENT F_a AS A FUNCTION OF SITE CLASS AND MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS (S_S)[a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	Note b
F	Note b	Note b	Note b	Note b	Note b

a. Use straight line interpolation for intermediate values of mapped spectral acceleration at short period, S_s.
b. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values.

TABLE 1615.1.2(2)
VALUES OF SITE COEFFICIENT F_V AS A FUNCTION OF SITE CLASS AND MAPPED SPECTRAL RESPONSE ACCELERATION AT 1-SECOND PERIOD (S_1)[a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS				
	$S_s \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	Note b
F	Note b	Note b	Note b	Note b	Note b

a. Use straight line interpolation for intermediate values of mapped spectral acceleration at 1-second period, S_1.
b. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values.

d_i = The thickness of any layer between 0 and 100 feet (30 480 mm).

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$ **(Equation 16-22)**

$$\sum_{i=1}^{n} d_i = 100 \text{ feet } (30\,480 \text{ mm})$$

N_i is the Standard Penetration Resistance (ASTM D 1586-84) not to exceed 100 blows/foot (mm) as directly measured in the field without corrections.

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$ **(Equation 16-23)**

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$ **(Equation 16-24)**

where:

$$\sum_{i=1}^{m} d_i = d_s$$

Use only d_i and N_i for cohesionless soils.

d_s = The total thickness of cohesionless soil layers in the top 100 feet (30 480 mm).

s_{ui} = The undrained shear strength in pounds per square foot (kPa), not to exceed 5,000 pounds per square foot (240 kPa), ASTM D 2166-91 or D 2850-87.

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}}$$ **(Equation 16-25)**

where:

$$\sum_{i=1}^{k} d_i = d_c$$

d_c = The total thickness ($100 - d_s$) (For SI: $30\,480 - d_s$) of cohesive soil layers in the top 100 feet (30 480 mm).

PI = The plasticity index, ASTM D 4318.

w = The moisture content in percent, ASTM D 2216.

The shear wave velocity for rock, Site Class B, shall be either measured on site or estimated by a geotechnical engineer or engineering geologist/seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Site Class C.

The hard rock, Site Class A, category shall be supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 feet (30 480 mm), surficial shear wave velocity measurements may be extrapolated to assess $\bar{v}_s$.

The rock categories, Site Classes A and B, shall not be used if there is more than 10 feet (3048 mm) of soil between the rock surface and the bottom of the spread footing or mat foundation.

1615.1.5.1 Steps for classifying a site.

1. Check for the four categories of Site Class F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific evaluation.
2. Check for the existence of a total thickness of soft clay > 10 feet (3048 mm) where a soft clay layer is defined by: $\bar{s}_u$ < 500 pounds per square foot (25 kPa), $w \geq 40$ percent, and $PI > 20$. If these criteria are satisfied, classify the site as Site Class E.
3. Categorize the site using one of the following three methods with $\bar{v}_s$, $\bar{N}$, and $\bar{s}_u$ computed in all cases as specified.
   3.1. $\bar{v}_s$ for the top 100 feet (30 480 mm) (v_s method).
   3.2. $\bar{N}$ for the top 100 feet (30 480 mm) ($\bar{N}$ method).
   3.3. $\bar{N}_{ch}$ for cohesionless soil layers ($PI < 20$) in the top 100 feet (30 480 mm) and average, $\bar{s}_u$, for cohesive soil layers ($PI > 20$) in the top 100 feet (30 480 mm) ($\bar{s}_u$ method).

1615.2 Site-specific procedure for determining ground motion accelerations. A site-specific study shall account for the regional seismicity and geology; the expected recurrence rates and maximum magnitudes of events on known faults and source zones; the location of the site with respect to these; near

TABLE 1615.1.5
SITE CLASSIFICATION[a]

SITE CLASS	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
E	< 600 ft/s	< 15	< 1,000 psf
D	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
C	1,200 to 2,500 ft/s	> 50	> 2,000

For SI: 1 foot per second = 304.8 mm per second, 1 pound per square foot = 0.0479 kN/m^2.

a. If the $\bar{s}_u$ method is used and the $\bar{N}_{ch}$ and $\bar{s}_u$ criteria differ, select the category with the softer soils (for example, use Site Class E instead of D).

source effects if any; and the characteristics of subsurface site conditions.

1615.2.1 Probabilistic-maximum considered earthquake. Where site-specific procedures are used as required or permitted by Section 1615, the maximum considered earthquake ground motion shall be taken as that motion represented by an acceleration response spectrum having a 2-percent probability of exceedance within a 50-year period. The maximum considered earthquake spectral response acceleration at any period, S_{aM}, shall be taken from the 2-percent probability of exceedance within a 50-year period spectrum.

> **Exception:** Where the spectral response ordinates at 0.2 second or 1 second for a 5-percent damped spectrum having a 2-percent probability of exceedance within a 50-year period exceeds the corresponding ordinates of the deterministic limit of Section 1615.2.2, the maximum considered earthquake ground motion spectrum shall be taken as the lesser of the probabilistic maximum considered earthquake ground motion or the deterministic maximum considered earthquake ground motion spectrum of Section 1615.2.3, but shall not be taken as less than the deterministic limit ground motion of Section 1615.2.2.

1615.2.2 Deterministic limit on maximum considered earthquake ground motion. The deterministic limit for the maximum considered earthquake ground motion shall be the response spectrum determined in accordance with Figure 1615.2.2, where site coefficients, F_a and F_v, are determined in accordance with Section 1615.1.2, with the value of the mapped short period spectral response acceleration, S_S, taken as 1.5g and the value of the mapped spectral response acceleration at 1 second, S_1, taken as 0.6g.

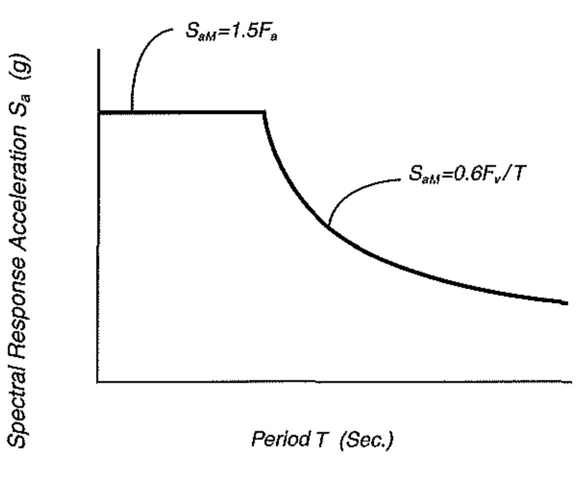


FIGURE 1615.2.2
DETERMINISTIC LIMIT ON MAXIMUM CONSIDERED EARTHQUAKE RESPONSE SPECTRUM

1615.2.3 Deterministic maximum considered earthquake ground motion. The deterministic maximum considered earthquake ground motion response spectrum shall be calculated as 150 percent of the median spectral response accelerations, S_{aM}, at all periods resulting from a characteristic earthquake on any known active fault within the region.

1615.2.4 Site-specific design ground motion. Where site-specific procedures are used to determine the maximum considered earthquake ground motion response spectrum, the design spectral response acceleration, S_a, at any period shall be determined from Equation 16-26:

$$S_a = \frac{2}{3} S_{aM} \qquad \textbf{(Equation 16-26)}$$

and shall be greater than or equal to 80 percent of the design spectral response acceleration, S_a, determined by the general response spectrum in Section 1615.1.4.

1615.2.5 Design spectral response coefficients. Where site-specific procedures are used as required or permitted by Section 1615, the design spectral response acceleration coefficient at short periods, S_{DS}, and the design spectral response acceleration at 1-second period, S_{D1}, shall be taken as the values of the design spectral response acceleration, S_a, obtained in accordance with Section 1615.2.4, at periods of 0.2 second and 1.0 second, respectively. The values so obtained shall not be taken as less than 80 percent of the values obtained from the general procedures of Section 1615.1.

SECTION 1616
EARTHQUAKE LOADS—CRITERIA SELECTION

1616.1 Structural design criteria. Each structure shall be assigned to a seismic design category in accordance with Section 1616.3. Seismic design categories are used in this code to determine permissible structural systems, limitations on height and irregularity, those components of the structure that must be designed for seismic resistance, and the types of lateral force analysis that must be performed.

Each structure shall be provided with complete lateral- and vertical-force-resisting systems capable of providing adequate strength, stiffness and energy dissipation capacity to withstand the design earthquake ground motions determined in accordance with Section 1615 within the prescribed deformation limits of Section 1617.3. The design ground motions shall be assumed to occur along any horizontal direction of a structure. A continuous load path, or paths, with adequate strength and stiffness to transfer forces induced by the design earthquake ground motions from the points of application to the final point of resistance, shall be provided.

Allowable Stress Design is permitted to be used to evaluate sliding, overturning and soil bearing at the soil-structure interface regardless of the design approach used in the design of the structure, provided load combinations of Section 1605.3 are utilized. When using Allowable Stress Design for proportioning foundations, the value of 0.2 $S_{DS}D$ in Equations 16-28, 16-29, 16-30 and 16-31 is permitted to be taken equal to zero. When the load combinations of Section 1605.3.2 are utilized, a one-third increase in soil allowable stresses is permitted for all load combinations that include W or E.

1616.2 Seismic use groups and occupancy importance factors. Each structure shall be assigned a seismic use group and a corresponding occupancy importance factor (I_E) as indicated in Table 1604.5.

1616.2.1 Seismic Use Group I. Seismic Use Group I structures are those not assigned to either Seismic Use Group II or III.

1616.2.2 Seismic Use Group II. Seismic Use Group II structures are those, the failure of which would result in a substantial public hazard due to occupancy or use as indicated by Table 1604.5, or as designated by the building official.

1616.2.3 Seismic Use Group III. Seismic Use Group III structures are those having essential facilities that are required for postearthquake recovery and those containing substantial quantities of hazardous substances, as indicated in Table 1604.5, or as designated by the building official.

Where operational access to a Seismic Use Group III structure is required through an adjacent structure, the adjacent structure shall conform to the requirements for Seismic Use Group III structures. Where operational access is less than 10 feet (3048 mm) from an interior lot line or less than 10 feet (3048 mm) from another structure, access protection from potential falling debris shall be provided by the owner of the Seismic Use Group III structure.

1616.2.4 Multiple occupancies. Where a structure is occupied for two or more occupancies not included in the same seismic use group, the structure shall be assigned the classification of the highest seismic use group corresponding to the various occupancies.

Where structures have two or more portions that are structurally separated in accordance with Section 1620, each portion shall be separately classified. Where a structurally separated portion of a structure provides required access to, required egress from, or shares life safety components with another portion having a higher seismic use group, both portions shall be assigned the higher seismic use group.

1616.3 Determination of seismic design category. All structures shall be assigned to a seismic design category based on their seismic use group and the design spectral response acceleration coefficients, S_{DS} and S_{D1}, determined in accordance with Section 1615.1.3 or 1615.2.5. Each building and structure shall be assigned to the most severe seismic design category in accordance with Table 1616.3(1) or 1616.3(2), irrespective of the fundamental period of vibration of the structure, T.

TABLE 1616.3(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT PERIOD RESPONSE ACCELERATIONS

VALUE OF S_{DS}	SEISMIC USE GROUP		
	I	II	III
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D[a]	D[a]	D[a]

a. Seismic Use Groups I and II structures located on sites with mapped maximum considered earthquake spectral response acceleration at 1-second period, S_1, equal to or greater than 0.75g, shall be assigned to Seismic Design Category E, and Seismic Use Group III structures located on such sites shall be assigned to Seismic Design Category F.

TABLE 1616.3(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	SEISMIC USE GROUP		
	I	II	III
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D[a]	D[a]	D[a]

a. Seismic Use Groups I and II structures located on sites with mapped maximum considered earthquake spectral response acceleration at 1-second period, S_1, equal to or greater than 0.75g, shall be assigned to Seismic Design Category E, and Seismic Use Group III structures located on such sites shall be assigned to Seismic Design Category F.

1616.3.1 Site limitation for Seismic Design Category E or F. A structure assigned to Seismic Design Category E or F shall not be sited over an identified active fault trace.

Exception: Detached Group R-3 as applicable in Section 101.2 of light-frame construction.

1616.4 Design requirements for Seismic Design Category A. Structures assigned to Seismic Design Category A need only comply with the requirements of Sections 1616.4.1 through 1616.4.4.

1616.4.1 Minimum lateral force. Structures shall be provided with a complete lateral-force-resisting system designed to resist the minimum lateral force, F_x, applied simultaneously at each floor level given by Equation 16-27:

$$F_x = 0.01\, w_x \qquad \textbf{(Equation 16-27)}$$

where:

F_x = The design lateral force applied at Level x.

w_x = The portion of the total gravity load of the structure, W, located or assigned to Level x.

W = The total dead load and other loads listed below:

1. In areas used for storage, a minimum of 25 percent of the reduced floor live load (floor live load in public garages and open parking structures need not be included).
2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 pounds per square foot (0.479 kN/m^2) of floor area, whichever is greater.
3. Total operating weight of permanent equipment.
4. Twenty percent of flat roof snow load where flat roof snow load exceeds 30 pounds per square foot (1.44 kN/m^2).

The direction of application of seismic forces used in design shall be that which will produce the most critical load effect in each component. The design seismic forces are permitted to be applied separately in each of two orthogonal directions and orthogonal effects are permitted to be neglected.

The effect of this lateral force shall be taken as E in the load combinations prescribed in Section 1605.2 for strength or load and resistance factor design methods, or Section 1605.3 for allowable stress design methods. Special seismic load combinations that include E_m need not be considered.

1616.4.2 Connections. All parts of the structure between separation joints shall be interconnected, and the connections shall be capable of transmitting the seismic force, F_p, induced in the connection by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure for F_p equal to 0.05 times the weight of the smaller portion. A positive connection for resisting horizontal forces acting on the member shall be provided for each beam, girder or truss to its support. The connection shall have strength sufficient to resist 5 percent of the dead and live load vertical reaction applied horizontally.

1616.4.3 Anchorage of concrete or masonry walls. See Section 1604.8.2.

1616.4.4 Conventional light-frame construction. Buildings constructed in compliance with Section 2308 are deemed to comply with Sections 1616.4.1, 1616.4.2 and 1616.4.3.

1616.5 Building configuration. Buildings shall be classified as regular or irregular based on the criteria in this section. Such classification shall be based on the plan and vertical configuration.

1616.5.1 Plan irregularity. Buildings having one or more of the features listed in Table 1616.5.1 shall be designated as having plan structural irregularity and shall comply with the requirements in the sections referenced in Table 1616.5.1.

1616.5.2 Vertical irregularity. Buildings having one or more of the features listed in Table 1616.5.2 shall be designated as having vertical irregularity and shall comply with the requirements in the sections referenced in Table 1616.5.2.

Exceptions:

1. Structural irregularities of Type 1a, 1b or 2 in Table 1616.5.2 do not apply where no story drift ratio under design lateral load is greater than 130 percent of the story drift ratio of the next story above. Torsional effects need not be considered in the calculation of story drifts for the purpose of this determination. The story drift ratio relationship for the top two stories of the building are not required to be evaluated.
2. Irregularities Types 1a, 1b and 2 of Table 1616.5.2 are not required to be considered for one-story buildings in any seismic design category or for two-story buildings in Seismic Design Category A, B, C or D.

1616.6 Analysis procedures. A structural analysis shall be made for all structures in accordance with the requirements of this section. The analysis shall form the basis for determining the seismic forces, E and E_m, to be applied in the load combinations of Section 1605 and shall form the basis for determining the design drift as required by Section 1617.3.

Exceptions:

1. Structures assigned to Seismic Design Category A.
2. Design drift need not be evaluated in accordance with Section 1617.3 when the simplified analysis method of Section 1617.5 is used.

1616.6.1 Simplified analysis. A simplified analysis, in accordance with Section 1617.5 shall be permitted to be used for any structure in Seismic Use Group I, subject to the following limitations, or a more rigorous analysis shall be made:

1. Buildings of light-framed construction not exceeding three stories in height, excluding basements.
2. Buildings of any construction other than light framed construction, not exceeding two stories in height, excluding basements, with flexible diaphragms at every level as defined in Section 1602.

1616.6.2 Seismic Design Category B or C. Except as permitted by Section 1616.6, the analysis procedures in Section 1617.4 shall be used for structures assigned to Seismic Design Category B or C (Section 1616) or a more rigorous analysis is permitted to be made.

TABLE 1616.5.1
PLAN STRUCTURAL IRREGULARITIES

	IRREGULARITY TYPE AND DESCRIPTION	REFERENCE SECTION	SEISMIC DESIGN CATEGORY[a] APPLICATION
1a	Torsional Irregularity—to be considered when diaphragms are not flexible as determined in Section 1602.1.1 Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure.	1617.4.4.5 1620.3.1 1616.6.3 Table 1616.6.3 1617.4.6.1	C, D, E and F D, E, and F D, E, and F D, E, and F C, D, E and F
1b	Extreme Torsional Irregularity—to be considered when diaphragms are not flexible as determined in Section 1602.1. Extreme torsional irregularity shall be considered to exist when the maximum story drift, computed and including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure.	1617.4.4.5 1620.3.1 1620.4.1 1616.6.3 Table 1616.6.3 1617.4.6.1	C, D, E and F D E and F D, E and F D, E, and F C, D, E and F
2	Re-entrant Corners Plan configurations of a structure and its lateral-force-resisting system contain re-entrant corners where both projections of the structure beyond a re-entrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.	1620.3.1	D, E and F
3	Diaphragm Discontinuity Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.	1620.3.1	D, E and F
4	Out-of-Plane Offsets Discontinuities in a lateral- force-resistance path, such as out-of-plane offsets of the vertical elements.	1620.3.1 1616.6.3 1620.1.9	D, E and F D, E and F B, C, D, E and F
5	Nonparallel Systems The vertical lateral-force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force-resisting system.	1620.2.2	C, D, E and F

a. Seismic Design Category is determined in accordance with Section 1616.

TABLE 1616.5.2
VERTICAL STRUCTURAL IRREGULARITIES

	IRREGULARITY TYPE AND DESCRIPTION	REFERENCE SECTION	SEISMIC DESIGN CATEGORY[a] APPLICATION
1a	Stiffness Irregularity—Soft Story A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.	1616.6.3 Table 1616.6.3	D, E, and F D, E, and F
1b	Stiffness Irregularity—Extreme Soft Story An extreme soft story is one in which the lateral stiffness is less than 60 percent of that in the story above or less than 70 percent of the average stiffness of the three stories above.	1620.4.1 1616.6.3 Table 1616.6.3	E and F D, E and F D, E and F
2	Weight (Mass) Irregularity Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	Table 1616.6.3	D, E and F
3	Vertical Geometric Irregularity Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any story is more than 130 percent of that in an adjacent story.	Table 1616.6.3	D, E and F
4	In-plane Discontinuity in Vertical Lateral-Force-Resisting Elements An in-plane offset of the lateral-force-resisting elements greater than the length of those elements or a reduction in stiffness of the resisting element in the story below.	1620.3.1 1616.6.3 1620.1.9	D, E and F D, E and F B, C, D, E and F
5	Discontinuity in Capacity—Weak Story A weak story is one in which the story lateral strength is less than 80 percent of that in the story above. The story strength is the total strength of seismic-resisting elements sharing the story shear for the direction under consideration.	1620.1.3 1616.6.3 1620.4.1	B, C, D, E and F D, E and F E and F

a. Seismic Design Category is determined in accordance with Section 1616.

1616.6.3 Seismic Design Category D, E or F. The analysis procedures identified in Table 1616.6.3 shall be used for structures assigned to Seismic Design Category D, E or F (see Section 1616), or a more rigorous analysis shall be made. For regular structures five stories or fewer in height having a period T, as determined in Section 1617.4.2, of 0.5 second or less, the design spectral response accelerations, S_{DS} and S_{D1}, need not exceed the values calculated using values of S_S and S_1, respectively, of 1.5g and 0.6g.

For the purposes of this section, structures shall be considered regular if they do not have plan irregularities 1a, 1b or 4 of Table 1616.5.1 or vertical irregularities 1a, 1b, 4 or 5 of Table 1616.5.2.

SECTION 1617
EARTHQUAKE LOADS—MINIMUM DESIGN LATERAL FORCE AND RELATED EFFECTS

1617.1 Seismic load effect E and E_m. Seismic load effect, E and E_m, for use in the load combinations of Section 1605 shall be determined as follows.

1617.1.1 Seismic load effect E. Where the effects of gravity and the seismic ground motion are additive, seismic load, E, for use in Formulas 16-5, 16-10, and 16-17 shall be defined by Equation 16-28:

$$E = \rho Q_E + 0.2 S_{DS} D \qquad \textbf{(Equation 16-28)}$$

where:

D = The effect of dead load.

E = The combined effect of horizontal and vertical earthquake-induced forces.

ρ = A reliability factor based on system redundancy obtained in accordance with Section 1617.2.

Q_E = The effect of horizontal seismic forces.

S_{DS} = The design spectral response acceleration at short periods obtained from Section 1615.1.3 or 1615.2.5.

Where the effects of gravity and seismic ground motion counteract, the seismic load, E, for use in Formulas 16-6, 16-12 and 16-18 shall be defined by Equation 16-29.

$$E = \rho Q_E - 0.2 S_{DS} D \qquad \textbf{(Equation 16-29)}$$

Design shall use the load combinations prescribed in Section 1605.2 for strength or load and resistance factor design methodologies, or Section 1605.3 for allowable stress design methods.

1617.1.2 Maximum seismic load effect, E_m. The maximum seismic load effect, E_m, shall be used in the special seismic load combinations in Section 1605.4.

Where the effects of the seismic ground motion and gravity loads are additive, seismic load, E_m, for use in Formula 16-19 shall be defined by Equation 16-30.

$$E_m = \Omega_0 Q_E + 0.2 S_{DS} D \qquad \textbf{(Equation 16-30)}$$

Where the effects of the seismic ground and gravity loads counteract, seismic load, E_m, for use in Formula 16-20 shall be defined by Equation 16-31.

$$E_m = \Omega_0 Q_E - 0.2 S_{DS} D \qquad \textbf{(Equation 16-31)}$$

where E, Q_E, S_{DS} are as defined above and Ω_0 is the system overstrength factor as given in Table 1617.6.

TABLE 1616.6.3
ANALYSIS PROCEDURES FOR SEISMIC DESIGN CATEGORIES D, E OR F

STRUCTURE DESCRIPTION	MINIMUM ALLOWABLE ANALYSIS PROCEDURE FOR SEISMIC DESIGN
1. Seismic Use Group I buildings of light-framed construction three stories or less in height and of other construction, two stories or less in height with flexible diaphragms at every level.	Simplified procedure of Section 1617.5.
2. Regular structures, other than those in Item 1 above, up to 240 feet in height.	Equivalent lateral-force procedure (Section 1617.4).
3. Structures that have vertical irregularities of Type 1a, 1b, 2 or 3 in Table 1616.5.2, or plan irregularities of Type 1a or 1b of Table 1616.5.1, and have a height exceeding five stories or 65 feet and structures exceeding 240 feet in height.	Modal analysis procedure (Section 1618).
4. Other structures designated as having plan or vertical irregularities.	Equivalent lateral-force procedure (Section 1617.4) with dynamic characteristics included in the analytical model.
5. Structures with all of the following characteristics: — located in an area with S_{D1} of 0.2 or greater, as determined in Section 1615.1.3; — located in an area assigned to Site Class E or F, in accordance with Section 1615.1.1 and; — with a natural period T of 0.7 second or greater, as determined in Section 1617.4.2.	Modal analysis procedure (Section 1618). A site-specific response spectrum shall be used but the design base shear shall not be less than that determined from Section 1617.4.1.

For SI: 1 foot = 304.8 mm.

The term $\Omega_0 Q_E$ need not exceed the maximum force that can be transferred to the element by the other elements of the lateral-force-resisting system.

Where allowable stress design methodologies are used with the special load combinations of Section 1605.4, design strengths are permitted to be determined using an allowable stress increase of 1.7 and a resistance factor, ϕ, of 1.0. This increase shall not be combined with increases in allowable stresses or load combination reductions otherwise permitted by this code or the material reference standard except that combination with the duration of load increases permitted in Chapter 23 is permitted.

1617.2 Redundancy. A redundancy coefficient, ρ, shall be assigned to all structures in accordance with this section, based on the extent of structural redundancy inherent in the lateral-force-resisting system.

1617.2.1 Seismic Design Category A, B or C. For structures assigned to Seismic Design Category A, B or C (see Section 1616), the value of the redundancy coefficient ρ is 1.0.

1617.2.2 Seismic Design Category D, E or F. For structures in Seismic Design Category D, E or F (see Section 1616), the redundancy coefficient, ρ, shall be taken as the largest of the values of, ρ_i, calculated at each story "i" of the structure in accordance with Equation 16-32 as follows:

$$\rho_i = 2 - \frac{20}{r_{\max_i}\sqrt{A_i}} \qquad \textbf{(Equation 16-32)}$$

For SI:

$$\rho_i = 2 - \frac{6.1}{r_{\max_i}\sqrt{A_i}}$$

where:

r_{max_i} = The ratio of the design story shear resisted by the most heavily loaded single element in the story to the total story shear, for a given direction of loading.

r_{max_i} = For braced frames the value, r_{max_i}, is equal to the lateral force component in the most heavily loaded brace element divided by the story shear.

r_{max_i} = For moment frames, r_{max_i}, shall be taken as the maximum of the sum of the shears in any two adjacent columns in a moment frame divided by the story shear. For columns common to two bays with moment-resisting connections on opposite sides at the level under consideration, it is permitted to use 70 percent of the shear in that column in the column shear summation.

r_{max_i} = For shear walls, r_{max_i}, shall be taken as the maximum value of the product of the shear in the wall or wall pier and $10/l_w$ ($3.3/l_w$ for SI), divided by the story shear, where l_w is the length of the wall or wall pier in feet (m).

r_{max_i} = For dual systems, r_{max_i}, shall be taken as the maximum value defined above, considering all lateral-load-resisting elements in the story. The lateral loads shall be distributed to elements based on relative rigidities considering the interaction of the dual system. For dual systems, the value of ρ need not exceed 80 percent of the value calculated above.

A_i = The floor area in square feet of the diaphragm level immediately above the story.

The value, ρ, shall not be less than 1.0, and need not exceed 1.5.

For structures with seismic-force-resisting systems in any direction comprised solely of special moment frames, the seismic-force-resisting system shall be configured such that the value of ρ calculated in accordance with this section does not exceed 1.25 for structures assigned to Seismic Design Category D, and does not exceed 1.1 for structures assigned to Seismic Design Category E or F.

For structures with vertical combinations of seismic-force resisting systems, the value, ρ, shall be determined independently for each seismic-force-resisting system. The reliability/redundancy factor of the lower portion shall not be less than the following:

$$\rho_L = \frac{R_L \rho_u}{R_u} \qquad \textbf{(Equation 16-33)}$$

where:

ρ_L = ρ of lower portion.

R_L = R of lower portion.

ρ_u = ρ of upper portion.

R_u = R of upper portion.

1617.3 Deflection and drift limits. The design story drift, Δ, as determined in Section 1617.4.6 or 1617.5.3, shall not exceed the allowable story drift, Δ_a, as obtained from Table 1617.3 for any story. All portions of the building shall be designed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact under total deflection, δ_x, as determined in Section 1617.4.6.1.

1617.4 Equivalent lateral force procedure for seismic design of buildings. See Section 1616.6 for limitations on the use of this procedure. For purposes of this analytical procedure, a building is considered to be fixed at the base.

1617.4.1 Seismic base shear. The seismic base shear, V, in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \qquad \textbf{(Equation 16-34)}$$

where:

C_s = The seismic response coefficient determined in accordance with Section 1617.4.1.1.

W = The effective seismic weight of the structure, including the total dead load and other loads listed below:

1. In areas used for storage, a minimum of 25 percent of the reduced floor live load (floor live load in public garages and open parking structures need not be included).

TABLE 1617.3
ALLOWABLE STORY DRIFT, Δ_a (inches)[a]

BUILDING	SEISMIC USE GROUP		
	I	II	III
Buildings, other than masonry shear wall or masonry wall frame buildings, four stories or less in height with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025\ h_{sx}$[b]	$0.020\ h_{sx}$	$0.015\ h_{sx}$
Masonry cantilever shear wall buildings[c]	$0.010\ h_{sx}$	$0.010\ h_{sx}$	$0.010\ h_{sx}$
Other masonry shear wall buildings	$0.007\ h_{sx}$	$0.007\ h_{sx}$	$0.007\ h_{sx}$
Masonry wall frame buildings	$0.013\ h_{sx}$	$0.013\ h_{sx}$	$0.010\ h_{sx}$
All other buildings	$0.020\ h_{sx}$	$0.015\ h_{sx}$	$0.010\ h_{sx}$

For SI: 1 inch = 25.4 mm.

a. There shall be no drift limit for single-story buildings with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.

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

c. Buildings in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 pounds per square foot (500 Pa/m²) of floor area, whichever is greater.
3. Total operating weight of permanent equipment.
4. Twenty percent of flat roof snow load where the flat roof snow load exceeds 30 pounds per square foot (1.44 kN/m²).

1617.4.1.1 Calculation of seismic response coefficient. The seismic response coefficient, C_s, shall be determined in accordance with the following equation:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} \qquad \textbf{(Equation 16-35)}$$

where:

I_E = The occupancy importance factor determined in accordance with Section 1616.2.

R = The response modification factor from Table 1617.6.

S_{DS} = The design spectral response acceleration at short period as determined in Section 1615.1.3.

The value of the seismic response coefficient, C_s, computed in accordance with Equation 16-35 need not exceed the following:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} \qquad \textbf{(Equation 16-36)}$$

but shall not be taken less than:

$$C_s = 0.044 S_{DS} I_E \qquad \textbf{(Equation 16-37)}$$

For buildings and structures in Seismic Design Category E or F, and those buildings and structures for which the 1-second spectral response, S_1, is equal to or greater than 0.6g, the value of the seismic response coefficient, C_s, shall not be taken as less than:

$$C_s = \frac{0.5 S_1}{R/I_E} \qquad \textbf{(Equation 16-38)}$$

where I and R are as defined above and:

S_{D1} = The design spectral response acceleration at 1-second period as determined from Section 1615.1.3.

S_1 = The maximum considered earthquake spectral response acceleration at 1-second period determined in accordance with Section 1615.1.

T = The fundamental period of the building (seconds) determined in Section 1617.4.2.

1617.4.2 Period determination. The fundamental period of the building, 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, or shall be taken as the approximate fundamental period, T_a, determined in accordance with the requirements of Section 1617.4.2. The calculated fundamental period, T, shall not exceed the product of the coefficient for upper limit on calculated period, C_u, from Table 1617.4.2 and the approximate fundamental period, T_a.

TABLE 1617.4.2
COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD

DESIGN SPECTRAL RESPONSE ACCELERATION AT 1-SECOND PERIOD, S_{D1}	COEFFICIENT C_u
≥ 0.4	1.2
0.3	1.3
0.2	1.4
0.15	1.5
≤ 0.1	1.7

1617.4.2.1 Approximate fundamental period. The approximate fundamental period, T_a, in seconds, shall be determined from the following equation:

$$T_a = C_T h_n^{3/4} \quad \textbf{(Equation 16-39)}$$

where:

C_T = building period coefficient 0.035 for moment-resisting frame systems of steel in which the frames resist 100 percent of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frames from deflecting when subjected to seismic forces (the metric coefficient is 0.085), 0.030 for moment-resisting frame systems of reinforced concrete in which the frames resist 100 percent of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frames from deflecting when subjected to seismic forces (the metric coefficient is 0.073), 0.030 for eccentrically braced steel frames (the metric coefficient is 0.073), 0.020 for all other building systems (the metric coefficient is 0.049), and

h_n = The height (ft or m) above the base to the highest level of the building.

Alternately, determination of the approximate fundamental period, T_a, in seconds, from the following formula for concrete and steel-moment resisting frame buildings not exceeding 12 stories in height and having a minimum story height of 10 feet (3048 mm) is permitted:

$$T_a = 0.1N \quad \textbf{(Equation 16-40)}$$

where:

N = Number of stories.

1617.4.3 Vertical distribution of seismic forces. The lateral force, F_x (kip or kN), induced at any level shall be determined from the following equations:

$$F_x = C_{vx} V \quad \textbf{(Equation 16-41)}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \quad \textbf{(Equation 16-42)}$$

where:

C_{vx} = Vertical distribution factor.

k = A distribution exponent related to the building period as follows:
For buildings having a period of 0.5 second or less, $k = 1$.
For buildings having a period of 2.5 seconds or more, $k = 2$.
For buildings having a period between 0.5 and 2.5 seconds, k shall be 2 or shall be determined by linear interpolation between 1 and 2.

h_i and h_x = The height (feet or m) from the base to Level i or x.

V = Total design lateral force or shear at the base of the building (kip or kN).

w_i and w_x = The portion of the total gravity load of the building, W, located or assigned to Level i or x.

1617.4.4 Horizontal shear distribution. The seismic design story shear in any story, V_x (kip or kN), shall be determined from the following equation:

$$V_x = \sum_{i=x}^{n} F_i \quad \textbf{(Equation 16-43)}$$

where:

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

1617.4.4.1 Rigid diaphragms. For rigid diaphragms as defined in Section 1602.1, the seismic design story shear, V_x, 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.

1617.4.4.2 Flexible diaphragms. For flexible diaphragms as defined in Section 1602.1, the seismic design story shear, V_x, shall be distributed to various vertical elements based on the tributary area of the diaphragm to each line of resistance. For the purpose of this section, the vertical elements of the lateral-force-resisting system are permitted to be considered to be in the same line of resistance if the maximum out-of-plane offset between such elements is less than 5 percent of the building dimension perpendicular to the direction of lateral load.

1617.4.4.3 Torsion. Where diaphragms are not flexible, the design shall include the torsional moment, M_t (kip · ft or kN · m), resulting from the difference in locations of the center of mass and the center of stiffness.

1617.4.4.4 Accidental torsion. Where diaphragms are not flexible, in addition to the torsional moment, the design also shall include accidental torsional moments, M_{ta} (kip · ft or kN · m), 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 building perpendicular to the direction of the applied forces.

1617.4.4.5 Dynamic amplification of torsion. For structures in Seismic Design Category C, D, E or F (Section 1616), where Type 1a or 1b plan torsional irregularity exists as defined in Table 1616.5.1, effects of torsional irregularity shall be accounted for by multiplying the sum of M_t plus M_{ta} (as determined in the preceding sections) at each level by a torsional amplification factor, A_x, determined from the following equation:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \quad \textbf{(Equation 16-44)}$$

where:

δ_{max} = The maximum displacement at Level x (inches or mm).

δ_{avg} = The average of the displacements at the extreme points of the structure at Level x (inches or mm).

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.

1617.4.5 Overturning. The building shall be designed to resist overturning effects caused by the seismic forces determined in Section 1617.4.3. At any story, the increment of overturning moment in the story under consideration shall be distributed to the various vertical-force-resisting elements in the same proportion as the distribution of the horizontal shears to those elements.

The overturning moments at Level x, M_x (kip · ft or kN · m), shall be determined from the following equation:

$$M_x = \tau \sum_{i=x}^{n} F_i (h_i - h_x) \qquad \textbf{(Equation 16-45)}$$

where:

F_i = The portion of the seismic base shear, V, induced at Level i.

h_i and h_x = The height (feet or m) from the base to Level i or x.

τ = The overturning moment reduction factor, determined as follows:

1. 1.0 for the top 10 stories.
2. 0.8 for the 20th story from the top and below.
3. value between 1.0 and 0.8 determined by a straight line interpolation for stories between the 20th and 10th stories below the top.

1617.4.6 Drift determination and P-delta effects. Frames and columns shall be designed to resist both brittle fracture and overturning instability during the maximum lateral excursion of each story, while supporting full dead and live load.

1617.4.6.1 Story drift determination. The design story drift, Δ, shall be computed as the difference of the deflections at the center of mass at the top and bottom of the story under consideration. Where allowable stress design is used, Δ shall be computed using earthquake forces without dividing by 1.4. For structures assigned to Seismic Design Category C, D, E or F (see Section 1616) having plan irregularity Types 1a or 1b of Table 1616.5.1, the design story drift, Δ, shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the story under consideration.

The deflections of Level x, δ_x (inches or mm), shall be determined in accordance with following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E} \qquad \textbf{(Equation 16-46)}$$

where:

C_d = The deflection amplification factor in Table 1617.6.

δ_{xe} = The deflections (inches or mm) determined by an elastic analysis of the seismic-force-resisting system.

I_E = The occupancy importance factor determined in accordance with Section 1616.2.

For determining compliance with the story drift limitation of Section 1617.3, the deflections of Level x, δ_x (inches or mm), shall be calculated as required in this section. For purposes of this drift analysis only, the upper bound limitation specified in Section 1617.4.2 on the computed fundamental period, T, in seconds, of the building, shall not apply.

The design story drift, Δ (inches or mm), shall be increased by the incremental factor relating to the P-delta effects, a_d, $1.0/(1 - \theta)$ as determined in Section 1617.4.6.2.

When calculating drift, the redundancy coefficient, ρ, shall be taken as 1.0.

1617.4.6.2 P-delta effects. P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered when the stability coefficient, θ, as determined by the following equation is equal to or less than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad \textbf{(Equation 16-47)}$$

where:

P_x = The total unfactored vertical design load at and above Level x (kip or kN); when calculating the vertical design load for purposes of determining P-delta, the individual load factors need not exceed 1.0;

Δ = The design story drift (inches or mm) occurring simultaneously with V_x;

V_x = The seismic shear force (kip or kN) acting between Level x and $x - 1$;

h_{sx} = The story height (inches or mm) below Level x; and

C_d = The deflection amplification factor in Table 1617.6.

The stability coefficient, θ, shall not exceed θ_{max} determined as follows:

$$\theta_{max} = \frac{0.5}{\beta C_d} \le 0.25 \qquad \textbf{(Equation 16-48)}$$

where:

β = The ratio of shear demand to shear capacity for the story between Level x and $x - 1$. Where the ratio, β, is not calculated, a value of $\beta = 1.0$ shall be used.

When the stability coefficient, θ, is greater than 0.10 but less than or equal to θ_{max}, interstory drifts and element forces shall be computed including P-delta effects. To obtain the story drift for including the P-delta effect, the design story drift determined in Section 1617.4.6.1 shall be multiplied by $1.0/(1-\theta)$.

Where θ is greater than θ_{max}, the structure is potentially unstable and shall be redesigned.

1617.5 Simplified analytical procedure for seismic design of buildings. See Section 1616.6 for limitations on the use of this procedure. For purposes of this analytical procedure, a building is considered to be fixed at the base.

1617.5.1 Seismic base shear. The seismic base shear, V, in a given direction shall be determined in accordance with the following equation:

$$V=\frac{1.2S_{DS}}{R}W \qquad \textbf{(Equation 16-49)}$$

where:

S_{DS} = The design elastic response acceleration at short period as determined in accordance with Section 1615.1.3.

R = The response modification factor from Table 1617.6.

W = The effective seismic weight of the structure, including the total dead load and other loads listed below:

1. In areas used for storage, a minimum of 25 percent of the reduced floor live load (floor live load in public garages and open parking structures need not be included).
2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 pounds per square foot of floor area, whichever is greater (0.48 kN/m^2).
3. Total weight of permanent operating equipment.
4. 20 percent of flat roof snow load where flat snow load exceeds 30 pounds per square foot (1.44 kN/m^2).

1617.5.2 Vertical distribution. The forces at each level shall be calculated using the following equation:

$$F_x=\frac{1.2S_{DS}}{R}w_x \qquad \textbf{(Equation 16-50)}$$

where:

w_x = The portion of the effective seismic weight of the structure, W, at Level x.

1617.5.3 Design drift. For the purposes of Sections 1617.3 and 1620.3.6 the design story drift, Δ, shall be taken as 1 percent of the story height unless a more exact analysis is provided.

1617.6 Seismic-force-resisting systems. The basic lateral and vertical seismic-force-resisting systems shall conform to one of the types indicated in Table 1617.6 subject to the limitations on height indicated in the table based on seismic design category as determined in Section 1616. The appropriate response modification coefficient, R, system overstrength factor, Ω_0, and deflection amplification factor, C_d, indicated in Table 1617.6 shall be used in determining the base shear, element design forces and design story drift.

For seismic-force-resisting systems not listed in Table 1617.6, analytical and test data shall be 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 1617.6 for equivalent response modification coefficient, R, system overstrength coefficient, Ω_0, and deflection amplification factor, C_d, values.

Exception: Structures assigned to Seismic Design Category A.

1617.6.1 Dual systems. For a dual system, the moment frame shall be capable of resisting at least 25 percent of the design forces. The total seismic force resistance is to be provided by the combination of the moment frame and the shear walls or braced frames in proportion to their stiffness.

1617.6.2 Combination along the same axis. For other than dual systems and shear wall-frame interactive systems, where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value, R, used for design in that direction shall not be greater than the least value for any of the systems utilized in that same direction.

Exception: For light frame, flexible diaphragm buildings, of Seismic Use Group I and two stories or less in height: Resisting elements are permitted to be designed using the least value of R for the different structural systems found on each independent line of resistance. The value of R used for design of diaphragms in such structures shall not be greater than the least value for any of the systems utilized in that same direction.

1617.6.3 Combinations of framing systems. Where different seismic-force-resisting systems are used along the two orthogonal axes of the structure, the appropriate response modification coefficient, R, system overstrength factor, Ω_0, and deflection amplification factor, C_d, indicated in Table 1617.6 for each system shall be used.

1617.6.3.1 Combination framing factor. The response modification coefficient, R, in the direction under consideration at any story shall not exceed the lowest response modification coefficient, R, for the seismic-force-resisting system in the same direction considered above that story, excluding penthouses. The system overstrength factor, Ω_0, in the direction under consideration at any story shall not be less than the largest value of this factor for the seismic-force-resisting system in the same direction considered above that story. In structures assigned to Seismic Design Category D, E or F, if a system with a response modification coefficient, R, with a value less than 5 is used as part of the seismic-force-resisting system in any direction of the structure, the lowest such value shall be used for the entire structure.

Exceptions:

1. Detached one- and two-family dwellings constructed of light framing.
2. The response modification coefficient, R, and system overstrength factor, Ω_0, for supported structural systems with a weight equal to or less than 10 percent of the weight of the structure are permitted to be determined independent of the values of these parameters for the structure as a whole.
3. The following two-stage static analysis procedure is permitted to be used provided the structure complies with the following:
   - 3.1. The flexible upper portion shall be designed as a separate structure using the appropriate values of R and ρ.
   - 3.2. The rigid lower portion shall be designed as a separate structure using the appropriate values of R and ρ. The reactions from the upper portion shall be those determined from the analysis of the upper portion amplified by the ratio of the, R/ρ, of the upper portion over, R/ρ, of the lower portion. This ratio shall not be less than 1.0.
   - 3.3. The lower portion shall have a stiffness at least 10 times the upper portion.
   - 3.4. The period of the entire structure shall not be greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.

1617.6.3.2 Combination framing detailing requirements. The detailing requirements of Section 1620 required by the higher response modification coefficient, R, shall be used for structural components common to systems having different response modification coefficients.

1617.6.4 System limitations for Seismic Design Category D, E or F. In addition to the system limitation indicated in Table 1617.6, structures assigned to Seismic Design Category D, E or F shall be subject to the following.

1617.6.4.1 Limited building height. For buildings that have steel-braced frames or concrete cast-in-place shear walls, the height limits in Table 1617.6 for Seismic Design Category D or E are increased to 240 feet (73 152 mm) and for Seismic Design Category F to 160 feet (48 768 mm) provided that the buildings are configured such that the braced frames or shear walls arranged in any one plane conform to the following:

1. The braced frames or shear walls in any one plane shall resist no more than 50 percent of the total seismic forces in each direction, neglecting torsional effects.
2. The seismic force in the braced frames or shear walls in any one plane resulting from torsional effects shall not exceed 20 percent of the total seismic force in the braced frames or shear walls.

1617.6.4.2 Interaction effects. Moment-resisting frames that are enclosed or adjoined by stiffer elements 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 consider and provide for the effect of these rigid elements on the structural system at deformations corresponding to the design story drift, Δ, as determined in Section 1617.4.6. In addition, the effects of these elements shall be considered when determining whether a structure has one or more of the irregularities defined in Section 1616.5.

1617.6.4.3 Deformational compatibility. Every structural component not included in the seismic-force- resisting system in the direction under consideration shall be designed to be adequate for vertical load-carrying capacity and the induced moments and shears resulting from the design story drift, Δ, as determined in accordance with Sections 1617.3 and 1617.4.6. Where allowable stress design is used, Δ shall be computed without dividing the earthquake force by 1.4. The moments and shears induced in components that are not included in the seismic-force-resisting system in the direction under consideration shall be calculated including the stiffening effects of adjoining rigid structural and nonstructural elements.

Exception: Reinforced concrete frame members not designed as part of the seismic-force-resisting system shall comply with Section 21.9 of ACI 318.

1617.6.4.4 Special moment frames. A special moment frame that is used but not required by Table 1617.6 is permitted to be discontinued and supported by a stiffer system with a lower response modification coefficient, R, provided the requirements of Sections 1620.1.3 and 1620.3.1 are met. Where a special moment frame is required by Table 1617.6, the frame shall be continuous to the foundation.

SECTION 1618
DYNAMIC ANALYSIS PROCEDURE FOR THE SEISMIC DESIGN OF BUILDINGS

1618.1 Dynamic analysis procedures. The following dynamic analysis procedures performed in accordance with the requirements of this section may be used in lieu of equivalent lateral force procedure of Section 1617.4:

1. Modal Response Spectral Analysis.
2. Linear Time-history Analysis.
3. Nonlinear Time-history Analysis.

See Section 1618.10 for Time-history analysis procedure.

1618.1.1 Modeling. A mathematical model of the building shall be constructed that represents the spatial distribution of mass and stiffness throughout the structure. For regular structures with independent orthogonal seismic-force-resisting systems, independent two-dimensional models may be constructed to represent each system. For irregular structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis shall be included at each level of the building. Where the diaphragms are not rigid compared to the vertical elements of the lateral-force-resisting system, the model shall include representation of the diaphragm's flexibility and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response. In addition, the model shall comply with the following:

1. Stiffness properties of concrete and masonry elements shall include the effects of cracked sections.
2. For steel moment frame systems, the contribution of panel zone deformations to overall story drift shall be included.

1618.2 Modes. An analysis shall be conducted to determine the natural modes of vibration for the building including the period of each mode, the modal shape vector, ϕ, the mass participation factor, and the modal mass. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual building mass in each of two orthogonal directions.

1618.3 Modal properties. The required periods, mode shapes, and participation factors of the building shall be calculated by established methods of structural analysis for the fixed base condition using the masses and elastic stiffnesses of the seismic-force-resisting system.

1618.4 Modal base shear. The portion of the base shear contributed by the m^{th} mode, V_m, shall be determined from the following equations:

$$V_m = C_{sm}\overline{W}_m \quad \textbf{(Equation 16-51)}$$

$$\overline{W}_m = \frac{\left(\sum_{i=1}^{n} w_i\phi_{im}\right)^2}{\sum_{i=1}^{n} w_i\phi_{im}^2} \quad \textbf{(Equation 16-52)}$$

where:

C_{sm} = The modal seismic response coefficient determined in Equation 16-53.

$\overline{W}_m$ = The effective modal gravity load.

w_i = The portion of the total gravity load, W, of the building at Level i, where W = the total dead load and other loads listed below:

1. In areas used for storage, a minimum of 25 percent of the reduced floor live load (floor live load in public garages and open parking structures need not be included).
2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 pounds per square foot (0.479 kN/m²) of floor area, whichever is greater.
3. Total operating weight of permanent equipment.
4. Twenty percent of flat roof snow load where the flat roof snow load exceeds 30 pounds per square foot (1.44 kN/m²).

ϕ_{im} = The displacement amplitude at the i^{th} level of the building when vibrating in its m^{th} mode.

The modal seismic response coefficient, C_{sm}, shall be determined by the following equation:

$$C_{sm} = \frac{S_{am}}{\left(\frac{R}{I_E}\right)} \quad \textbf{(Equation 16-53)}$$

where:

I_E = The occupancy importance factor determined in accordance with Section 1616.2.

S_{am} = The modal design spectral response acceleration at period T_m determined from either the general design response spectrum of Section 1615.1 or a site-specific response spectrum per Section 1615.2.

R = The response modification factor determined from Table 1617.6.

Exception: When the general design response spectrum of Section 1615.1 is used for buildings on Site Class D, E or F sites (see Section 1615.1.1), the modal seismic design coefficient, C_{sm}, for modes other than the fundamental mode that have periods less than 0.3 second is permitted to be determined by the following equation:

$$C_{sm} = \frac{0.4S_{DS}}{\left(\frac{R}{I_E}\right)}(1.0 + 5.0T_m) \quad \textbf{(Equation 16-54)}$$

where:

I_E = The occupancy importance factor determined in accordance with Section 1616.2.

R = The response modification factor determined from Table 1617.6.

S_{DS} = The design spectral response acceleration at short periods as defined in Section 1615.1.3.

T_m = The modal period of vibration (in seconds) of the mth mode of the building.

TABLE 1617.6
DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC-FORCE-RESISTING SYSTEMS

BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R^a	SYSTEM OVER-STRENGTH FACTOR, Ω_0^g	DEFLECTION AMPLIFICATION FACTOR, C_d^b	SYSTEM LIMITATIONS AND BUILDING HEIGHT LIMITATIONS (FEET) BY SEISMIC DESIGN CATEGORY AS DETERMINED IN SECTION 1616.3[c]				
					A or B	C	D[d]	E[e]	F[e]
1. Bearing Wall Systems									
A. Ordinary steel braced frames	(14)[j]2211	4	2	$3\frac{1}{2}$	NL	NL	160	160	160
B. Special reinforced concrete shear walls	1910.2.4	$5\frac{1}{2}$	$2\frac{1}{2}$	5	NL	NL	160	160	160
C. Ordinary reinforced concrete shear walls	1910.2.3	$4\frac{1}{2}$	$2\frac{1}{2}$	4	NL	NL	NP	NP	NP
D. Detailed plain concrete shear walls	1910.2.2	$2\frac{1}{2}$	$2\frac{1}{2}$	2	NL	NP	NP	NP	NP
E. Ordinary plain concrete shear walls	1910.2.1	$1\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{1}{2}$	NL	NP	NP	NP	NP
F. Special reinforced masonry shear walls	2106.1.1.5	5	$2\frac{1}{2}$	$3\frac{1}{2}$	NL	NL	160	160	100
G. Intermediate reinforced masonry shear walls	2106.1.1.4	$3\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{4}$	NL	NL	NP	NP	NP
H. Ordinary reinforced masonry shear walls	2106.1.1.2	$2\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{3}{4}$	NL	160	NP	NP	NP
I. Detailed plain masonry shear walls	2106.1.1.3	2	$2\frac{1}{2}$	$1\frac{3}{4}$	NL	NP	NP	NP	NP
J. Ordinary plain masonry shear walls	2106.1.1.1	$1\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{1}{4}$	NL	NP	NP	NP	NP
K. Light frame walls with shear panels—wood structural panels/sheet steel panels	2306.4.1/ 2211	6	3	4	NL	NL	65	65	65
L. Light frame walls with shear panels—all other materials	2306.4.5	2	$2\frac{1}{2}$	2	NL	NL	35	NP	NP
2. Building Frame Systems									
A. Steel eccentrically braced frames, moment-resisting, connections at columns away from links	(15)[j]	8	2	4	NL	NL	160	160	100
B. Steel eccentrically braced frames, nonmoment resisting, connections at columns away from links	(15)[j]	7	2	4	NL	NL	160	160	100
C. Special steel concentrically braced frames	(13)[j]	6	2	5	NL	NL	160	160	100
D. Ordinary steel concentrically braced frames	(14)[j]	5	2	$4\frac{1}{2}$	NL	NL	160	100	100
E. Special reinforced concrete shear walls	1910.2.4	6	$2\frac{1}{2}$	5	NL	NL	160	160	100
F. Ordinary reinforced concrete shear walls	1910.2.3	5	$2\frac{1}{2}$	$4\frac{1}{2}$	NL	NL	NP	NP	NP
G. Detailed plain concrete shear walls	1910.2.2	3	$2\frac{1}{2}$	$2\frac{1}{2}$	NL	NP	NP	NP	NP
H. Ordinary plain concrete shear walls	1910.2.1	2	$2\frac{1}{2}$	2	NP	NP	NP	NP	NP
I. Composite eccentrically braced frames	(14)[k]	8	2	4	NL	NL	160	160	100

(continued)

TABLE 1617.6—continued
DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC-FORCE-RESISTING SYSTEMS

BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R[a]	SYSTEM OVER-STRENGTH FACTOR, Ω_o[g]	DEFLECTION AMPLIFICATION FACTOR, C_d[b]	SYSTEM LIMITATIONS AND BUILDING HEIGHT LIMITATIONS (FEET) BY SEISMIC DESIGN CATEGORY[c] AS DETERMINED IN SECTION 1616.3[c]				
					A or B	C	D[d]	E[e]	F[e]
J. Composite concentrically braced frames	(13)[k]	5	2	$4\frac{1}{2}$	NL	NL	160	160	100
K. Ordinary composite braced frames	(12)[k]	3	2	3	NL	NL	NP	NP	NP
L. Composite steel plate shear walls	(17)[k]	$6\frac{1}{2}$	$2\frac{1}{2}$	$5\frac{1}{2}$	NL	NL	160	160	100
M. Special composite reinforced concrete shear walls with steel elements	(16)[k]	6	$2\frac{1}{2}$	5	NL	NL	160	160	100
N. Ordinary composite reinforced concrete shear walls with steel elements	(15)[k]	5	$2\frac{1}{2}$	$4\frac{1}{2}$	NL	NL	NP	NP	NP
O. Special reinforced masonry shear walls	2106.1.1.5	$5\frac{1}{2}$	$2\frac{1}{2}$	4	NL	NL	160	160	100
P. Intermediate reinforced masonry shear walls	2106.1.1.4	4	$2\frac{1}{2}$	$2\frac{1}{2}$	NL	NL	NP	NP	NP
Q. Ordinary reinforced masonry shear walls	2106.1.1.2	3	$2\frac{1}{2}$	$2\frac{1}{4}$	NL	160	NP	NP	NP
R. Detailed plain masonry shear walls	2106.1.1.3	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{4}$	NL	NP	NP	NP	NP
S. Ordinary plain masonry shear walls	2106.1.1.1	$1\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{1}{4}$	NL	NP	NP	NP	NP
T. Light frame walls with shear panels—wood structural panels/sheet steel panels	2306.4.1/ 2211	$6\frac{1}{2}$	$2\frac{1}{2}$	$4\frac{1}{2}$	NL	NL	65	65	65
U. Light frame walls with shear panels—all other materials	2306.4.5	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{2}$	NL	NL	35	NP	NP
3. Moment-resisting Frame Systems									
A. Special steel moment frames	(9)[j]	8	3	$5\frac{1}{2}$	NL	NL	NL	NL	NL
B. Special steel truss moment frames	(12)[j]	7	3	$5\frac{1}{2}$	NL	NL	160	100	NP
C. Intermediate steel moment frames	(10)[j]	6	3	5	NL	NL	160	100	NP[h]
D. Ordinary steel moment frames	(11)[j]	4	3	$3\frac{1}{2}$	NL	NL	35[h]	NP[h,i]	NP[h,i]
E. Special reinforced concrete moment frames	(21.1)[l]	8	3	$5\frac{1}{2}$	NL	NL	NL	NL	NL
F. Intermediate reinforced concrete moment frames	(21.1)[l]	5	3	$4\frac{1}{2}$	NL	NL	NP	NP	NP
G. Ordinary reinforced concrete moment frames	(21.1)[l]	3	3	$2\frac{1}{2}$	NL	NP	NP	NP	NP
H. Special composite moment frames	(9)[k]	8	3	$5\frac{1}{2}$	NL	NL	NL	NL	NL
I. Intermediate composite moment frames	(10)[k]	5	3	$4\frac{1}{2}$	NL	NL	NP	NP	NP
J. Composite partially restrained moment frames	(8)[k]	6	3	$5\frac{1}{2}$	160	160	100	NP	NP
K. Ordinary composite moment frames	(11)[k]	3	3	$2\frac{1}{2}$	NL	NP	NP	NP	NP
L. Masonry wall frames	2108.9.6 2106.1.1.6	$5\frac{1}{2}$	3	5	NL	NL	160	160	100

(continued)

TABLE 1617.6—continued
DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC-FORCE-RESISTING SYSTEMS

BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R^a	SYSTEM OVER-STRENGTH FACTOR, Ω_o[g]	DEFLECTION AMPLIFICATION FACTOR, C_d[b]	SYSTEM LIMITATIONS AND BUILDING HEIGHT LIMITATIONS (FEET) BY SEISMIC DESIGN CATEGORY AS DETERMINED IN SECTION 1616.3[c]				
					A or B	C	D[d]	E[e]	F[e]
4. Dual Systems with Special Moment Frames									
A. Steel eccentrically braced frames, moment-resisting connections, at columns away from links	(15)[j]	8	2 1/2	4	NL	NL	NL	NL	NL
B. Steel eccentrically braced frames, nonmoment-resisting connections, at columns away from links	(15)[j]	7	2 1/2	4	NL	NL	NL	NL	NL
C. Special steel concentrically braced frames	(13)[j]	8	2 1/2	6 1/2	NL	NL	NL	NL	NL
D. Ordinary steel concentrically braced frames	(14)[j]	6	2 1/2	5	NL	NL	NL	NL	NL
E. Special reinforced concrete shear walls	1910.2.4	8	2 1/2	6 1/2	NL	NL	NL	NL	NL
F. Ordinary reinforced concrete shear walls	1910.2.3	7	2 1/2	6	NL	NL	NP	NP	NP
G. Composite eccentrically braced frames	(14)[k]	8	2 1/2	4	NL	NL	NL	NL	NL
H. Composite concentrically braced frames	(13)[k]	6	2 1/2	5	NL	NL	NL	NL	NL
I. Composite steel plate shear walls	(17)[k]	8	2 1/2	6 1/2	NL	NL	NL	NL	NL
J. Special composite reinforced concrete shear walls with steel elements	(16)[k]	8	2 1/2	6 1/2	NL	NL	NL	NL	NL
K. Ordinary composite reinforced concrete shear walls with steel elements	(15)[k]	7	2 1/2	6	NL	NL	NP	NP	NP
L. Special reinforced masonry shear walls	2106.1.1.5	7	3	6 1/2	NL	NL	NL	NL	NL
M. Intermediate reinforced masonry shear walls	2106.1.1.4	6 1/2	3	5 1/2	NL	NL	NP	NP	NP
5. Dual Systems with Intermediate Moment Frames									
A. Special steel concentrically braced frames[f]	(13)[j]	6	2 1/2	5	NL	NL	160	100	NP
B. Ordinary steel concentrically braced frames[f]	(14)[j]	5	2 1/2	4 1/2	NL	NL	160	100	NP
C. Special reinforced concrete shear walls	1910.2.4	6	2 1/2	5	NL	NL	160	100	100
D. Ordinary reinforced concrete shear walls	1910.2.3	5 1/2	2 1/2	4 1/2	NL	NL	NP	NP	NP
E. Ordinary reinforced masonry shear walls	2106.1.1.2	3	3	2 1/2	NL	160	NP	NP	NP
F. Intermediate reinforced masonry shear walls	2106.1.1.4	5	3	4 1/2	NL	NL	NP	NP	NP
G. Composite concentrically braced frames	(13)[k]	5	2 1/2	4 1/2	NL	NL	160	100	NP
H. Ordinary composite braced frames	(12)[k]	4	2 1/2	3	NL	NL	NP	NP	NP
I. Ordinary composite reinforced concrete shear walls with steel elements	(15)[k]	5 1/2	2 1/2	4 1/2	NL	NL	NP	NP	NP

(continued)

TABLE 1617.6–continued
DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC-FORCE-RESISTING SYSTEMS

BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R^a	SYSTEM OVER-STRENGTH FACTOR, Ω_o^g	DEFLECTION AMPLIFICATION FACTOR, C_d^b	SYSTEM LIMITATIONS AND BUILDING HEIGHT LIMITATIONS (FEET) BY SEISMIC DESIGN CATEGORY AS DETERMINED IN SECTION 1616.3[c]				
					A or B	C	D[d]	E[e]	F[e]
J. Shear wall-frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls	21.1[l] 1910.2.3	$5^1/_2$	$2^1/_2$	5	NL	NP	NP	NP	NP
6. Inverted Pendulum Systems									
A. Cantilevered column systems		$2^1/_2$	2	$2^1/_2$	NL	NL	35	35	35
B. Special steel moment frames	(9)[j]	$2^1/_2$	2	$2^1/_2$	NL	NL	NL	NL	NL
C. Ordinary steel moment frames	(11)[j]	$1^1/_4$	2	$2^1/_2$	NL	NL	NP	NP	NP
D. Special reinforced concrete moment frames	21.1[l]	$2^1/_2$	2	$1^1/_4$	NL	NL	NL	NL	NL
7. Structural Steel Systems Not Specifically Detailed for Seismic Resistance	AISC—ASD AISC—LRFD AISI AISC—HSS	3	3	3	NL	NL	NP	NP	NP

For SI: 1 foot = 304.8 mm, 1 pound per square foot = 0.0479 KN/m^2.

a. Response modification coefficient, R, for use throughout.
b. Deflection amplification factor, C_d.
c. NL = not limited and NP = not permitted.
d. See Section 1617.6.4.1 for a description of building systems limited to buildings with a height of 240 feet or less.
e. See Section 1617.6.4.1 for building systems limited to buildings with a height of 160 feet or less.
f. Ordinary moment frame is permitted to be used in lieu of intermediate moment frame in Seismic Design Categories B and C.
g. The tabulated value of the overstrength factor, Ω_o, may be reduced by subtracting $^1/_2$ for structures with flexible diaphragms but shall not be taken as less than 2.0 for any structure.
h. Steel ordinary moment frames and intermediate moment frames are permitted in single story buildings up to a height of 60 feet, when the moment joints of field connections are constructed of bolted end plates and the dead load of the roof does not exceed 15 pounds per square foot. The dead weight of the portion of walls more than 35 feet above the base shall not exceed 15 pounds per square foot.
i. Steel ordinary moment frames are permitted in buildings up to a height of 35 feet, where the dead load of the walls, floors and roof does not exceed 15 pounds per square foot.
j. AISC Seismic Part I or Part III, Section number.
k. AISC Seismic Part II, Section number.
l. ACI 318, Section number.

1618.5 Modal forces, deflections and drifts. The modal force, F_{xm}, at each level shall be determined by the following equations:

$$F_{xm} = C_{vxm} V_m \qquad \textbf{(Equation 16-55)}$$

$$C_{vxm} = \frac{w_x \phi_{xm}}{\sum_{i=1}^{n} w_i \phi_{im}} \qquad \textbf{(Equation 16-56)}$$

where:

C_{vxm} = The vertical distribution factor in the m^{th} mode.

V_m = The total design lateral force or shear at the base in the m^{th} mode.

w_i, w_x = The portion of the total gravity load of the building, W, located or assigned to Level i or x.

ϕ_{im} = The displacement amplitude at the i^{th} level of the building when vibrating in its m^{th} mode.

ϕ_{xm} = The displacement amplitude at the x^{th} level of the building when vibrating in its m^{th} mode.

The modal deflection at each level, δ_{xm}, shall be determined by the following equations:

$$\delta_{xm} = \frac{C_d \delta_{xem}}{I_E} \qquad \textbf{(Equation 16-57)}$$

$$\delta_{xem} = \left(\frac{g}{4\pi^2}\right)\left(\frac{T_m^2 F_{xm}}{w_x}\right) \qquad \textbf{(Equation 16-58)}$$

where:

C_d = The deflection amplification factor determined from Table 1617.6.

F_{xm} = The portion of the seismic base shear in the m^{th} mode, induced at Level x.

g = The acceleration due to gravity (ft/s^2 or m/s^2).

I_E = The occupancy importance factor determined in accordance with Section 1616.2.

T_m = The modal period of vibration, in seconds, of the m^{th} mode of the building.

w_x = The portion of the total gravity load of the building, W, located or assigned to Level x.

δ_{xem} = The deflection of Level x in the mth mode at the center of the mass at Level x determined by an elastic analysis.

The modal drift in a story, Δ_m, shall be computed as the difference of the deflections, δ_{xm}, at the top and bottom of the story under consideration in accordance with Section 1617.4.6.1.

1618.6 Modal story shears and moments. The story shears, story overturning moments, and the shear forces and overturning moments in vertical elements of the structural system at each level due to the seismic forces determined from the appropriate formula in Section 1618.5 shall be computed for each mode by linear static methods.

1618.7 Design values. The design value for the modal base shear, V_t; each of the story shear, moment and drift quantities; and the deflection at each level shall be determined by combining their modal values as obtained from Sections 1618.5 and 1618.6. The combination shall be carried out by taking the square root of the sum of the squares of each of the modal values or by the complete quadratic combination (CQC) technique.

The base shear, V, using the equivalent lateral force procedure in Section 1617.4 shall be calculated using a fundamental period of the building, T, in seconds, of 1.2 times the coefficient for upper limit on the calculated period, C_u, taken from Table 1617.4.2, times the approximate fundamental period of the building, T_a, calculated in accordance with Section 1617.4.2.1. Where the thus calculated base shear, V, is greater than the modal base shear, V_t, the design story shears, moments, drifts and floor deflections shall be multiplied by C_m, the modification factor:

$$C_m = \frac{V}{V_t} \qquad \textbf{(Equation 16-59)}$$

where:

V = The equivalent lateral force procedure base shear, calculated in accordance with this section and Section 1617.4.

V_t = The modal base shear, calculated in accordance with this section.

The modal base shear, V_t, need not exceed the base shear calculated from the equivalent lateral force procedure in Section 1617.4. However, for buildings with a value of the design spectral response acceleration at 1 second period, S_{D1}, of 0.2 or greater, as determined in Section 1615.1.3, with a period T, as determined in Section 1617.4.2, of 0.7 second or greater, and located on Site Class E or F sites (Section 1615.1.1), the design base shear shall not be less than that determined using the equivalent lateral force procedure in Section 1617.4.

1618.8 Horizontal shear distribution. The distribution of horizontal shear shall be in accordance with the requirements of Section 1617.4.4 except that amplification of torsion per Section 1617.4.4.5 is not required for that portion of the torsion included in the modal analysis model.

1618.9 P-delta effects. The P-delta effects shall be determined in accordance with Section 1617.4.6.2. The story drifts and story shears shall be determined in accordance with Section 1617.4.6.1.

1618.10 Time-history analysis.

1618.10.1 Time history. Time-history analysis shall be performed with pairs of appropriate horizontal ground-motion time-history components that shall be selected and scaled from not less than three recorded events. Appropriate time histories shall have magnitudes, fault distance and source mechanisms that are consistent with those that control the maximum considered earthquake. Where three appropriate recorded ground-motion time-history pairs are not available, appropriate simulated ground-motion time-history pairs shall be used to make up the total number required. For each pair of horizontal ground-motion components, the square root of the sum of the squares (SRSS) of the 5 percent damped site-specific spectrum of the scaled horizontal components shall be constructed. The motions shall be scaled such that the average value of the SRSS spectra is not less than 1.4 times the 5 percent damped spectrum of two-thirds the maximum considered earthquake for periods from 0.2 T second to 1.5 T seconds. Each pair of time histories shall be applied simultaneously to the model considering torsional effects.

The parameter of interest shall be calculated for each time-history analysis. If three time-history analyses are performed, then the maximum response of the parameter of interest shall be used for design. If seven or more time-history analyses are performed, then the average value of the response parameter of interest may be used for design.

1618.10.2 Elastic time-history analysis. Elastic time-history analysis shall conform to Sections 1616.6, 1618.1, 1618.9 and the base shear scaled in accordance with Section 1618.7. Strength design shall be used to determine member capacities.

1618.10.3 Nonlinear time-history analysis.

1618.10.3.1 Nonlinear time history. Time histories shall be developed and results determined in accordance with the requirements of Section 1618.10.1. Capacities and characteristics of nonlinear elements shall be modeled consistent with test data or substantiated analysis, considering the importance factor. The responses shall not be reduced by R/I_E. The maximum inelastic response displacement shall comply with Table 1617.3. Strength design shall be used to determine member capacities.

1618.10.3.2 Design review. When nonlinear time-history analysis is used to justify a structural design, a design review of the lateral-force-resisting system shall be performed by an independent team of registered design professionals in the appropriate disciplines and others experienced in seismic analysis methods. The seismic-force-resisting system design review shall include, but not be limited to, the following:

1. Reviewing the development of site-specific spectra and ground-motion time histories.
2. Reviewing the preliminary design of the lateral-force-resisting system.
3. Reviewing the final design of the lateral-force-resisting system and all supporting analyses.

SECTION 1619 EARTHQUAKE LOADS SOIL-STRUCTURE INTERACTION EFFECTS

1619.1 Analysis procedure. If soil-structure interaction is considered in the determination of seismic design forces and corresponding displacements in the structure, the procedure given in Section 9.5.5 of ASCE 7 shall be used.

SECTION 1620 EARTHQUAKE LOADS—DESIGN, DETAILING REQUIREMENTS AND STRUCTURAL COMPONENT LOAD EFFECTS

1620.1 Structural component design and detailing. The design and detailing of the components of the seismic-force-resisting system shall comply with the requirements of this section in addition to the nonseismic requirements of this code.

Exception: Structures assigned to Seismic Design Category A.

Structures assigned to Seismic Design Category B (see Section 1616) shall conform to Sections 1620.1.1 through 1620.1.10.

1620.1.1 Second-order load effects. Where θ exceeds 0.10 as determined in Section 1617.4.6.2, second order load effects shall be included in the evaluation of component and connection strengths.

1620.1.2 Openings. Where openings occur in shear walls, diaphragms or other plate-type elements, reinforcement at the edges of the openings shall be designed to transfer the stresses into the structure. The edge reinforcement shall extend into the body of the wall or diaphragm a distance sufficient to develop the force in the reinforcement.

1620.1.3 Discontinuities in vertical system. Structures with a discontinuity in lateral capacity, vertical irregularity Type 5, as defined in Table 1616.5.2, shall not be over two stories or 30 feet (9144 mm) in height where the "weak" story has a calculated strength of less than 65 percent of the story above.

Exception: Where the "weak" story is capable of resisting a total seismic force equal to 75 percent of the deflection amplification factor, C_d, times the design force prescribed in Section 1617.4, the height limitation does not apply.

1620.1.4 Connections. All parts of the structure, except at separation joints, shall be interconnected and the connections shall be designed to resist 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 for the greater of:

$$F_p = 0.133\, S_{DS}\, w_p \qquad \textbf{(Equation 16-60)}$$

or

$$F_p = 0.05\, w_p \qquad \textbf{(Equation 16-61)}$$

where:

S_{DS} = The design, 5-percent damped, spectral response acceleration at short periods as defined in Section 1615.

w_p = The weight of the smaller portion.

A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder or truss to its support for a force, not less than 5 percent of the dead plus live load reaction.

1620.1.5 Diaphragms. Permissible deflection shall be that deflection up to which the diaphragm and any attached distributing or resisting element will maintain its structural integrity under design load conditions, such that the resisting element will continue to support design loads without danger to occupants of the structure.

Floor and roof diaphragms shall be designed to resist F_p as follows:

$$F_p = 0.2 I_E S_{DS} w_p + V_{px} \quad \textbf{(Equation 16-62)}$$

where:

F_p = The seismic force induced by the parts.

I_E = Occupancy importance factor (Table 1604.5).

S_{DS} = The short period site design spectral response acceleration coefficient (Section 1615).

w_p = The weight of the diaphragm and other elements of the structure attached to the diaphragm.

V_{px} = The portion of the seismic shear force at the level of the diaphragm, required to be transferred to the components of the vertical seismic-force-resisting system because of the offsets or changes in stiffness of the vertical components above or below the diaphragm.

Diaphragms shall provide for both shear and bending stresses resulting from these forces. Diaphragms shall have ties or struts to distribute the wall anchorage forces into the diaphragm. Diaphragm connections shall be positive, mechanical or welded-type connections.

1620.1.6 Collector elements. Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces. Collector elements, splices and their connections to resisting elements shall have the design strength to resist the special load combinations of Section 1605.4.

Exception: In structures or portions thereof braced entirely by light-frame shear walls, collector elements, splices and connections to resisting elements need only have the strength to resist the load combinations of Section 1605.2 or 1605.3.

1620.1.7 Bearing walls and shear walls. Bearing walls and shear walls and their anchorage shall be designed for an out-of-plane force, F_p, that is the greater of 10 percent of the weight of the wall, or the quantity given by Equation 16-63:

$$F_p = 0.40 I_E S_{DS} w_w \quad \textbf{(Equation 16-63)}$$

where:

I_E = Occupancy importance factor (Table 1604.5).

S_{DS} = The short period site design spectral response acceleration coefficient (Section 1615.1.3 or 1615.2.5).

w_w = The weight of the wall.

In addition, concrete and masonry walls shall be anchored to the roof and floors and members that provide lateral support for the wall or that are supported by the wall. The anchorage shall provide a direct connection between the wall and the supporting construction capable of resisting the greater of the force, F_p, as given by Equation 16-63 or (400 $S_{DS} I_E$) pounds per linear foot of wall. For SI: 5838 $S_{DS} I_E$ N/m. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet (1219 mm). Parapets shall conform to the requirements of Section 1621.2.1.

1620.1.8 Inverted pendulum-type structures. Supporting columns or piers of inverted pendulum-type structures shall be designed for the bending moment calculated at the base determined using the procedures given in Section 1617.4 and varying uniformly to a moment at the top equal to one-half the calculated bending moment at the base.

1620.1.9 Elements supporting discontinuous walls or frames. Columns or other elements supporting discontinuous walls or frames of structures having plan irregularity Type 4 of Table 1616.5.1 or vertical irregularity Type 4 of Table 1616.5.2 shall have the design strength to resist special seismic load combinations of Section 1605.4.

Exceptions:

1. The quantity, E_m, in Section 1617.1.2 need not exceed the maximum force that can be transmitted to the element by the lateral-force-resisting system at yield.
2. Concrete slabs supporting light-frame walls.

1620.1.10 Direction of seismic load. The direction of application of seismic forces used in design shall be that which will produce the most critical load effect in each component. The requirement will be deemed satisfied if the design seismic forces are applied separately and independently in each of the two orthogonal directions.

1620.2 Seismic Design Category C. Structures assigned to Seismic Design Category C (see Section 1616) shall conform to the requirements of Section 1620.1 for Seismic Design Category B and to Sections 1620.2.1 through 1620.2.2.

1620.2.1 Anchorage of concrete or masonry walls. Concrete or masonry walls shall be anchored to floors and roofs and members that provide out-of-plane lateral support for the wall or that are supported by the wall. The anchorage shall provide a positive direct connection between the wall and floor or roof capable of resisting the horizontal forces specified in Equation 16-64 for structures with flexible diaphragms or in Section 1621.1.4 for structures with diaphragms that are not flexible.

$$F_p = 1.2 S_{DS} I_E w_w \quad \textbf{(Equation 16-64)}$$

where:

F_p = The design force in the individual anchors.

I_E = Occupancy importance factor per Section 1616.2.

S_{DS} = The design earthquake spectral response acceleration at short period per Section 1615.1.3.

w_w = The weight of the wall tributary to the anchor.

Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragms. Where added chords are used to form subdiaphragms, such chords shall transmit the anchorage forces to the main crossties. The maximum length-to-width ratio of the structural subdiaphragm shall be $2^1/_2$ to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components. Connections shall extend into the diaphragms a sufficient distance to develop the force transferred into the diaphragm.

In wood diaphragms, the continuous ties shall be in addition to the diaphragm sheathing. Anchorage shall not be accomplished by use of toenails or nails subject to withdrawal, nor shall wood ledgers or framing be used in cross-grain bending or cross-grain tension. The diaphragm sheathing shall not be considered effective as providing the ties or struts required by this section.

In metal deck diaphragms, the metal deck shall not be used as the continuous ties required by this section in the direction perpendicular to the deck span.

Diaphragm-to-wall anchorage using embedded straps shall be attached to or hooked around the reinforcing steel or otherwise terminated so as to directly transfer force to the reinforcing steel.

1620.2.2 Direction of seismic load. For structures that have plan structural irregularity Type 5 in Table 1616.5.1, the critical direction requirement of Section 1620.1.10 shall be deemed satisfied if components and their foundations are designed for the following orthogonal combination of prescribed loads.

One hundred percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction. The combination requiring the maximum component strength shall be used. Alternatively, the effects of the two orthogonal directions are permitted to be combined on a square root of the sum of the squares (SRSS) basis. When the square root of the sum of the squares method of combining directional effects is used, each term computed shall be assigned the sign that will result in the most conservative result.

1620.3 Seismic Design Category D. Structures assigned to Seismic Design Category D shall conform to the requirements of Section 1620.2 for Seismic Design Category C and to Sections 1620.3.1 through 1620.3.6.

1620.3.1 Plan or vertical irregularities. For buildings having a plan structural irregularity of Type 1a, 1b, 2, 3 or 4 in Table 1616.5.1 or a vertical structural irregularity of Type 4 in Table 1616.5.2, the design forces determined from Section 1617.4.1 shall be increased 25 percent for connections of diaphragms to vertical elements and to collectors, and for connections of collectors to the vertical elements.

1620.3.2 Vertical seismic forces. Horizontal cantilever and horizontal prestressed components shall be designed to resist the vertical component of earthquake ground motion. This requirement is considered to be met if:

1. The load combinations used in designing such components include E as defined in Equation 16-29; and
2. Such components are designed to resist, in addition to the applicable load combinations of Section 1605, a minimum net upward force of 0.2 times the dead load.

1620.3.3 Diaphragms. The deflection in the plane of the diaphragm shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached elements to maintain structural integrity under the individual loading and continue to support the prescribed loads.

Floor and roof diaphragms shall be designed to resist design seismic forces determined in accordance with Equation 16-65 as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad \textbf{(Equation 16-65)}$$

where:

F_i = The design force applied to Level i.

F_{px} = The diaphragm design force.

w_i = The weight tributary to Level i.

w_{px} = The weight tributary to the diaphragm at Level x.

The force determined from Equation 16-65 need not exceed 0.3 $S_{DS}I_E w_{px}$ but shall not be less than 0.15 $S_{DS}I_E w_{px}$, where S_{DS} is the design spectral response acceleration at short period determined in Section 1615.1.3 and I_E is the occupancy importance factor determined in Section 1616.2. When the diaphragm is required to transfer design seismic force from the vertical-resisting elements above the diaphragm to other vertical-resisting elements below the diaphragm due to offsets in the placement of the elements or to changes in relative lateral stiffness in the vertical elements, these forces shall be added to those determined from Equation 16-65 and to the upper and lower limits on that formula.

1620.3.4 Collector elements. Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing resistance to those forces.

Collector elements, splices and their connections to resisting elements shall resist the forces determined in accordance with Equation 16-65. In addition, collector elements, splices and their connections to resisting elements shall have the design strength to resist the earthquake loads as defined in the Special Load Combinations of Section 1605.4.

Exception: In structures, or portions thereof, braced entirely by light-frame shear walls, collector elements, splices and their connections to resisting elements need only be designed to resist forces in accordance with Equation 16-65.

1620.3.5 Direction of seismic load. The independent orthogonal procedure given in Section 1620.1.10 will not be satisfactory for the critical direction requirement for any structure. The orthogonal combination procedure in Section 1620.2.2 will be deemed satisfactory for any structure.

1620.3.6 Building separations. All structures shall be separated from adjoining structures. Separations shall allow for the displacement δ_M. Adjacent buildings on the same property shall be separated by at least, δ_{MT}, where

$$\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2} \qquad \textbf{(Equation 16-66)}$$

and δ_{M1} and δ_{M2} are the displacements of the adjacent buildings.

When a structure adjoins a property line not common to a public way, that structure shall also be set back from the property line by at least the displacement, δ_M, of that structure.

Exception: Smaller separations or property line setbacks shall be permitted when justified by rational analyses based on maximum expected ground motions.

1620.4 Seismic Design Category E or F. Structures assigned to Seismic Design Category E or F (Section 1616) shall conform to the requirements of Section 1620.3 for Seismic Design Category D and to Section 1620.4.1.

1620.4.1 Plan or vertical irregularities. Structures having plan irregularity Type 1b of Table 1616.5.1 or vertical irregularities Type 1b or 5 of Table 1616.5.2 shall not be permitted.

SECTION 1621
ARCHITECTURAL, MECHANICAL AND ELECTRICAL COMPONENT SEISMIC DESIGN REQUIREMENTS

1621.1 Component design. Architectural, mechanical, electrical and nonstructural systems, components, and elements permanently attached to structures, including supporting structures and attachments (hereinafter referred to as "components"), and nonbuilding structures that are supported by other structures, shall meet the requirements of this section.

Architectural, mechanical, electrical and other nonstructural components in structures shall be designed and constructed to resist equivalent static forces and displacements determined herein.

1621.1.1 Applicability to components. For the purposes of this chapter, unless otherwise noted, components shall be considered to have the same seismic design category as that of the structure that they occupy or to which they are attached, as determined in Section 1616.

Exception: The following components are exempt from the requirements of Section 1621:

1. Components in Seismic Design Category A.
2. Other than parapets supported by bearing walls or shear walls, architectural components in Seismic Design Category B when the component importance factor, I_p, is equal to 1.00.
3. Mechanical and electrical components in Seismic Design Category B.
4. Mechanical and electrical components in Seismic Design Category C when the component importance factor, I_p, is equal to 1.00.
5. Mechanical and electrical components in all seismic design categories, where I_p = 1.0 and flexible connections between the components and associated duct work, piping and conduit are provided, that are mounted at 4 feet (1219 mm) or less above a floor level and weigh 400 pounds (1780 N) or less, and are not critical to the continued operation of the structure.
6. Mechanical and electrical components in Seismic Design Category D, E or F that weigh 20 pounds (89 N) or less, where I_p = 1.0 and flexible connections between the components and associated duct work, piping and conduit are provided.

The interrelationship of components and their effect on each other shall be considered so that the failure of any essential or nonessential architectural, mechanical, or electrical component shall not cause the failure of another essential architectural, mechanical, or electrical component.

1621.1.2 Applicability to supported nonbuilding structures. The minimum seismic design forces for nonbuilding structures that are supported by other structures shall be determined in accordance with Section 1621. Nonbuilding structures supported at grade shall be designed in accordance with Section 1622. Nonbuilding structures supported by other structures shall meet the requirements of Section 1621.1.4 with R_p equal to the value of R specified in Section 1622 and a_p = 2.5 for nonbuilding structures with flexible component dynamic characteristics and a_p = 1.0 for nonbuilding structures with rigid component dynamic characteristics. The distribution of lateral forces for the supported nonbuilding structure and other applicable requirements specified in Section 1622 shall apply to nonbuilding structures.

Exception: For structures in Seismic Design Category D, E or F, as determined in Section 1616, if the combined weight of the supported components and nonbuilding structures with flexible component dynamic characteristics exceeds 25 percent of the weight of the structure, the structure shall be designed considering interaction effects between the structure and the supported items.

1621.1.3 Component force transfer. Components shall be attached such that the component forces are transferred to the structure of the building. Component seismic attachments shall be bolted, welded or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity. The construction documents shall include sufficient information relating to the attachments to verify compliance.

1621.1.4 Seismic forces. Seismic forces, F_p, shall be determined in accordance with Equation 16-67:

$$F_p = \frac{0.4 a_p S_{DS} W_p}{\left(\frac{R_p}{I_p}\right)}\left(1 + 2\frac{z}{h}\right) \qquad \textbf{(Equation 16-67)}$$

and F_p is not required to be taken as greater than:

$$F_p = 1.6 S_{DS} I_P W_P \quad \textbf{(Equation 16-68)}$$

and F_p shall not be taken as less than:

$$F_p = 0.3 S_{DS} I_P W_P \quad \textbf{(Equation 16-69)}$$

where:

a_p = Component amplification factor that varies from 1.00 to 2.50 (select appropriate value from Table 1621.2 or 1621.3).

F_p = Seismic design force centered at the component's center of gravity and distributed relative to component's mass distribution.

I_p = Component importance factor that is either 1.00 or 1.50, as determined in Section 1621.1.6.

h = Average roof height of structure relative to the base elevation.

R_p = Component response modification factor that varies from 1.0 to 5.0 (select appropriate value from Table 1621.2 or 1621.3).

S_{DS} = Design spectral response acceleration at short period, as determined in Section 1615.1.3.

W_p = Component operating weight.

z = Height in structure at point of attachment of component. For items at or below the base, z shall be taken as 0. For items at or above the roof, z is not required to be taken as greater than the roof height h.

The force, F_p, shall be applied independently longitudinally and laterally in combination with service loads associated with the component. Component earthquake effects shall be determined for combined horizontal and vertical load effects as indicated in Section 1617.1 substituting F_p for the term Q_E. The redundancy based reliability coefficient, ρ, required in Section 1617.1 is permitted to be taken as equal to 1.

When positive and negative wind loads exceed F_p for nonstructural exterior walls, these wind loads shall govern the design.

1621.1.5 Seismic relative displacements. Seismic relative displacements, D_p, shall be determined in accordance with the following equations. For two connection points on the same structure, A, or the same structural system, one on Level x and the other at Level y, D_p shall be determined as:

$$D_p = \delta_{xA} - \delta_{yA} \quad \textbf{(Equation 16-70)}$$

D_p is not required to be taken as greater than:

$$D_p = (X - Y)\frac{\Delta_{aA}}{h_{sx}} \quad \textbf{(Equation 16-71)}$$

For two connection points on separate structures, A and B, or separate structural systems, one at Level x and the other at Level y, D_p shall be determined as:

$$D_p = |\delta_{xA}| + |\delta_{yB}| \quad \textbf{(Equation 16-72)}$$

D_p is not required to be taken as greater than:

$$D_p = \frac{X\Delta_{aA}}{h_{sx}} + \frac{Y\Delta_{aB}}{h_{sx}} \quad \textbf{(Equation 16-73)}$$

where:

D_p = Relative seismic displacement that the component must be designed to accommodate.

h_{sx} = Story height used in the definition of the allowable drift, Δ_a, in Table 1617.3.

δ_{xA} = Deflection at structure Level x of structure A, determined by an elastic analysis as defined in Sections 1617.4.2 through 1617.4.6 and multiplied by the C_d factor determined from Table 1617.6.

δ_{yA} = Deflection at structure Level y of structure A, determined by an elastic analysis as defined in Sections 1617.4.2 through 1617.4.6 and multiplied by the C_d factor determined from Table 1617.6.

δ_{yB} = Deflection at structure Level y of structure B, determined by an elastic analysis as defined in Sections 1617.4.2 through 1617.4.6 and multiplied by the C_d factor determined from Table 1617.6.

X = Height of upper support attachment at Level x as measured from the base.

Y = Height of lower support attachment at Level y as measured from the base.

Δ_{aA} = Allowable story drift for structure A as defined in Table 1617.3.

Δ_{aB} = Allowable story drift for structure B as defined in Table 1617.3.

The effects of seismic relative displacements shall be considered in combination with displacements caused by other loads.

1621.1.6 Component importance factor. The component importance factor, I_p, shall be as follows:

$I_p = 1.5$ Life-safety component is required to function after an earthquake.

$I_p = 1.5$ Component contains hazardous or flammable material in quantities that exceed the exempted amounts for an open system listed in Chapter 4.

$I_p = 1.5$ Storage racks in occupancies open to the general public (e.g., warehouse retail stores).

$I_p = 1.0$ All other components.

In addition, for structures in Seismic Use Group III (Section 1616.2.3):

$I_p = 1.5$ Components needed for continued operation of the facility or whose failure could impair the continued operation of the facility.

1621.1.7 Component anchorage. Components shall be anchored in accordance with the following:

1. The force in the connected part shall be determined based on the prescribed forces for the component specified in Section 1621.1.4. Where component anchorage is provided by shallow expansion anchors, shallow chemical anchors, or shallow (low

deformability) cast-in-place anchors, a value of R_p = 1.50 shall be used in Section 1621.1.4 to determine the forces on the connected part.

2. Anchors embedded in concrete or masonry shall be proportioned to carry the lesser of the following:
    - 2.1. The design strength of the connected part.
    - 2.2. 1.3 times the force in the connected part due to the prescribed forces.
    - 2.3. The maximum force that can be transferred to the connected part by the component structural system.
3. Determination of forces in anchors shall include the expected conditions of installation including eccentricities and prying effects.
4. Determination of force distribution of groups of anchors at one location shall include the stiffness of the connected system and its ability to redistribute loads to other anchors in the group beyond yield.
5. Powder-driven fasteners shall not be used for tension load applications in Seismic Design Category D, E or F, as determined in Section 1616, unless approved for such loading.
6. The design strength of anchors in concrete shall be determined in accordance with the provisions of Section 1913.
7. For additional requirements for anchors to steel, see Chapter 22.
8. For additional requirements for anchors in masonry, see Chapter 21.
9. For additional requirements for anchors in wood, see Chapter 23.

1621.1.8 Quality assurance; special inspection and testing. A quality assurance plan shall be prepared when required by Section 1705. Special inspection and testing for seismic concerns shall be performed in accordance with Sections 1704 and 1708, respectively.

1621.2 Architectural component design. Architectural systems, components or elements (hereinafter referred to as "components") listed in Table 1621.2 and their attachments shall meet the requirements of this section.

1621.2.1 Architectural component forces and displacements. Architectural components shall meet the force requirements of Section 1621.1.4 and Table 1621.2 and the displacement requirements of Section 1621.1.5.

Exception: Components supported by chains or otherwise suspended from the structural system above are not required to meet the lateral seismic force requirements and seismic relative displacement requirements of this section provided that they are capable of moving a minimum of 12 inches (305 mm) or swing 45 degrees off vertical, without damage or contacting an obstruction. The gravity design load for these items shall be three times their operating load.

1621.2.2 Architectural component deformation. Architectural components shall be designed for the seismic relative displacement requirements of Section 1621.1.5. Architectural components shall be designed for vertical deflection due to joint rotation of cantilever structural members.

1621.2.3 Exterior wall elements and connections. Exterior nonstructural wall panels or elements that are attached to or enclose the structure shall be designed to resist the forces prescribed by Equation 16-67 and shall accommodate movements of the structure resulting from response to the design basis ground motion, the relative seismic displacement, D_p, as determined in Section 1621.1.5, or temperature changes, whichever is greater. Such elements shall be supported by means of positive and direct structural supports or by mechanical connections and fasteners in accordance with the following requirements:

1. Connections and panel joints shall allow for a relative movement between stories of not less than the relative seismic displacement for the story, D_p, or 0.5 inch (12.7 mm), whichever is greater.
2. Connections to permit movement in the plane of the panel for story drift shall be sliding connections using slotted or oversize holes, connections that permit movement by bending of steel, or other connections that provide equivalent sliding or ductile capacity.
3. Bodies of connections shall have sufficient ductility and rotation capacity to preclude fracture of the concrete or brittle failures at or near welds.
4. Fasteners in the connecting system such as bolts, inserts, welds, dowels and the body of the connectors shall be designed for the force, F_p, determined by Equation 16-67, with values of R_p and a_p taken from Table 1621.2 applied at the center of mass of the panel.
5. Anchorage using flat straps embedded in concrete shall be attached to or hooked around reinforcing steel, or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

1621.2.4 Out-of-plane bending. Transverse or out-of-plane bending or deformation of a component or system that is subjected to forces as determined in Section 1621.1.4 shall not exceed the deflection capability of the component or system.

1621.2.5 Suspended ceilings. Suspended ceilings shall meet the requirements of either industry standard construction as modified in Section 1621.2.5.2 or integral construction as specified in Section 1621.2.5.3.

Exception: Ceilings in buildings with an I_p equal to 1.0 located in Seismic Design Category C and less than three stories in height.

TABLE 1621.2
ARCHITECTURAL COMPONENTS COEFFICIENTS

ARCHITECTURAL COMPONENT OR ELEMENT	COMPONENT AMPLIFICATION FACTOR a_p[a]	COMPONENT RESPONSE MODIFICATION FACTOR R_p
1. Interior nonstructural walls and partitions (see also Section 1621.2.7)		
a. Plain (unreinforced) masonry walls	1.0	1.25
b. Other walls and partitions	1.0	2.5
2. Cantilever elements (unbraced or braced to structural frame below its center of mass)		
a. Parapets and cantilever interior nonstructural walls	2.5	2.5
b. Chimneys and stacks when laterally braced or supported by the structural frame	2.5	2.5
3. Cantilever elements (braced to structural frame above its center of mass)		
a. Parapets	1.0	2.5
b. Chimneys and stacks	1.0	2.5
c. Exterior nonstructural walls	1.0	2.5
4. Exterior nonstructural wall elements and connections (see also Section 1621.2.3)		
a. Wall element	1.0	2.5
b. Body of wall panel connections	1.0	2.5
c. Fasteners of the connecting system	1.25	1.0
5. Veneer		
a. Limited deformability elements and attachments	1.0	2.5
b. Low deformability elements or attachments	1.0	1.25
6. Penthouses (except when framed by an extension of the building frame)	2.5	3.5
7. Ceilings (see also Section 1621.2.5)	1.0	2.5
8. Cabinets		
a. Storage cabinets and laboratory equipment	1.0	2.5
9. Access floors (see also Section 1621.2.6)		
a. Special access floors (designed in accordance with Section 1621.2.6.1)	1.0	2.5
b. All other	1.0	1.25
10. Appendages and ornamentations	2.5	2.5
11. Signs and billboards	2.5	2.5
12. Other rigid components		
a. High deformability elements and attachments	1.0	3.5
b. Limited deformability elements and attachments	1.0	2.5
c. Low deformability materials and attachments	1.0	1.25
13. Other flexible components		
a. High deformability elements and attachments	1.0	3.5
b. Limited deformability elements and attachments	2.5	2.5
c. Low deformability materials and attachments	2.5	1.25

a. Where justified by detailed dynamic analyses, a lower value for a_p is permitted, but shall not be less than 1. The reduced value of a_p shall be between 2.5, assigned to flexible or flexibly attached equipment, and 1, assigned to rigid or rigidly attached equipment.

1621.2.5.1 Seismic forces. Suspended ceilings shall be designed to meet the force requirements of Section 1621.1.4. The weight of the ceiling, W_p, shall include the ceiling grid and panels; light fixtures if attached to, clipped to, or laterally supported by the ceiling grid; and other components that are laterally supported by the ceiling. W_p shall not be less than 4 pounds per square foot (192 N/m²). The seismic force, F_p, shall be transmitted through the ceiling attachments to the building structural elements or the ceiling-structure boundary.

Design of anchorage and connections shall be in accordance with Section 1621.1.7.

1621.2.5.2 Industry standard construction. Unless designed in accordance with Section 1621.2.5.3, suspended ceilings shall be designed and constructed in accordance with the following.

1621.2.5.2.1 Seismic Design Category C. Suspended ceilings in Seismic Design Category C, as determined in Section 1616, shall be designed and installed in accordance with CISCA 0-2, except that seismic forces shall be determined in accordance with Sections 1621.1.4 and 1621.2.5.1. Sprinkler heads and other penetrations shall have a minimum of 0.25-inch (6.4 mm) clearance on all sides.

1621.2.5.2.2 Seismic Design Category D, E or F. Suspended ceilings in Seismic Design Category D, E or F, as determined in Section 1616, shall be designed and installed in accordance with CISCA 3-4, and the following:

1. A heavy-duty T-bar grid system shall be used.
2. The width of the perimeter supporting closure angle shall be not less than 2 inches (51 mm). In each orthogonal horizontal direction, one end of the ceiling grid shall be attached to the closure angle. The other end in each horizontal direction shall have a 0.75-inch (19.1 mm) clearance from the wall and shall rest upon and be free to slide on a closure angle.
3. For ceiling areas exceeding 1,000 square feet (93 m²) horizontal restraint of the ceiling to the structure shall be provided. The horizontal restraints shall be designed to minimize diaphragm loads.

   Exception: Rigid braces may be used instead of diagonal splay wires. Braces and attachments to the structure above shall be adequate to limit relative lateral deflections at point of attachment of ceiling grid to less than 0.25 inch (6.4 mm) for the loads prescribed in Section 1621.1.4.

4. For ceiling areas exceeding 2,500 square feet (232 m²), a seismic separation joint or full height partition shall be provided unless analyses are performed that demonstrate ceiling system penetrations and closure angles provide sufficient clearance to accommodate the additional movement.
5. Except where rigid braces are used to limit lateral deflections, sprinkler heads and other penetrations shall have a 2-inch (51 mm) oversize ring, sleeve or adapter through the ceiling tile to allow for free movement of at least 1 inch (25 mm) in all horizontal directions. Alternatively, a swing joint that can accommodate 1 inch (25 mm) of ceiling movement in all horizontal directions shall be provided at the top of the sprinkler head extension.
6. Changes in ceiling plane elevation shall be provided with positive bracing.
7. Cable trays and electrical conduits shall be independently supported and braced independently of the ceiling.
8. Suspended ceiling shall be subject to the special inspection requirements of Section 1704.

1621.2.5.3 Integral ceiling/sprinkler construction. Where the requirements of Section 1621.2.5.2 are not met, the sprinkler system and ceiling grid shall be designed and tied together as an integral unit. Such a design shall consider the mass and flexibility of all elements involved, including ceiling system, sprinkler system, light fixtures and mechanical (HVAC) appurtenances. The design shall be performed by a registered design professional.

1621.2.6 Access floors. Access floors shall be designed to meet the force requirements of Section 1621.1.4 and the additional requirements of this section. The weight of the access floor, W_p, shall include the weight of the floor system, 100 percent of the weight of equipment fastened to the floor, and 25 percent of the weight of equipment supported by, but not fastened to, the floor. The seismic force, F_p, shall include the effect of overturning of equipment fastened to the access floor panels and shall be transmitted from the top surface of the access floor to the supporting structure.

When checking individual pedestals for overturning effects, the maximum concurrent axial load shall not exceed the portion of W_p assigned to the pedestal under consideration.

1621.2.6.1 Special access floors. Access floors shall be considered to be "special access floors" if they comply with the following:

1. Connections transmitting seismic loads consist of mechanical fasteners, concrete anchors, welding or bearing. Design load capacities comply with recognized design codes and/or certified test results.
2. Seismic loads are not transmitted by friction produced solely by the effects of gravity, powder-actuated fasteners (shot pins) or adhesives.
3. The bracing system is designed to resist the destabilizing effects of individual members buckling in compression.
4. Bracing and pedestals are of structural or mechanical shape produced to ASTM specifications that

specify minimum mechanical properties. Electrical tubing shall not be used for bracing or pedestals.

5. Floor stringers that are designed to carry axial seismic loads and that are mechanically fastened to the supporting pedestals are used.

1621.2.7 Partitions. Partitions that are tied to the ceiling and partitions greater than 6 feet (1829 mm) in height shall be laterally braced to the building structure. Such bracing shall be independent of any ceiling splay bracing. Bracing shall be spaced to limit horizontal deflection at the partition head to be compatible with ceiling deflection requirements as determined in Section 1621.2.5 for suspended ceilings and Section 1621.2.1 for other systems.

Exception: Partitions not taller than 9 feet (2743 mm) when the horizontal seismic load does not exceed 5 pounds per square foot (0.240 kN/m^2) required in Section 1607.13.

1621.2.8 Steel storage racks. Steel storage racks supported above grade shall be designed to meet the general requirements of Sections 1622.1, 1622.2, and the specific requirements of Section 1622.3.4. Steel storage racks supported at grade shall be designed in accordance with Section 1622.3.4.

1621.3 Mechanical and electrical component design. Attachments and equipment supports for the mechanical and electrical systems, components or elements (hereinafter referred to as "components") shall meet the requirements of Sections 1621.3.1 through 1621.3.14.4.

1621.3.1 Mechanical and electrical component forces and displacements. Mechanical and electrical components shall meet the force and seismic relative displacement requirements of Sections 1621.1.4, 1621.1.5 and Table 1621.3. Powder-actuated fasteners (shot pins) shall not be used for component anchorage in tension applications in Seismic Design Category D, E or F. Components mounted on vibration isolation systems shall have a bumper restraint or snubber in each horizontal direction. The lateral design force for the restraint shall be taken as $2F_p$.

Exception: Components supported by chains or similarly suspended from above are not required to meet the lateral seismic force requirements and seismic relative displacement requirements of this section provided that they cannot be damaged or cannot damage any other component when subject to seismic motion and they have ductile or articulating connections to the structure at the point of attachment. The gravity design load for these items shall be three times their operating load.

1621.3.2 Mechanical and electrical component period. The fundamental period of the mechanical and electrical component (and its attachment to the building), T_p, shall be determined by the following formula provided that the component and attachment can be reasonably represented analytically by a simple spring and mass single-degree-of-freedom system:

$$T_p = 2\pi\sqrt{\frac{W_p}{K_p g}} \qquad \textbf{(Equation 16-74)}$$

where:

g = Acceleration of gravity in inches/sec^2 (mm/s^2).

K_p = Stiffness of resilient support system of the component and attachment, determined in terms of load per unit deflection at the center of gravity of the component.

T_p = Component fundamental period.

W_p = Component operating weight.

Alternatively, the fundamental period of the component in seconds, T_p, shall be determined from experimental test data or by analysis.

1621.3.3 Mechanical and electrical component attachments. The stiffness of mechanical and electrical component attachments shall be designed such that the load path for the component performs its intended function.

1621.3.4 Component supports. Mechanical and electrical component supports and the means by which they are attached to the component shall be designed for the forces determined in Section 1621.1.4 and in conformance with the requirements of this code applying to the materials comprising the means of attachment. Such supports include, but are not limited to, structural members, braces, frames, skirts, legs, saddles, pedestals, cables, guys, stays, snubbers and tethers. Component supports are permitted to be forged or cast as a part of the mechanical or electrical component. If standard or proprietary supports are used, they shall be designed by either load rating (i.e., testing) or for the calculated seismic forces. The stiffness of the support shall be designed such that the seismic load path for the component performs its intended function.

Component supports shall be designed to accommodate the seismic relative displacements between points of support determined in accordance with Section 1621.2.5.

The means by which supports are attached to the component, except when integral (i.e., cast or forged), shall be designed to accommodate both the forces and displacements determined in accordance with Sections 1621.1.4 and 1621.1.5. If the value of $I_p = 1.5$ for the component, the local region of the support attachment point to the component shall be designed to resist the effect of the load transfer on the component wall.

1621.3.5 Deleted.

1621.3.6 Utility and service lines at structure interfaces. At the interface of adjacent structures or portions of the same structure that may move independently, utility lines shall be provided with adequate flexibility to accommodate the anticipated differential movement between the ground and the structure. Differential displacements shall be determined in accordance with Section 1621.1.5.

1621.3.7 Site-specific considerations. Designated seismic systems in Seismic Use Group III, as defined in Section 1616.2.3 shall be able to continue functioning if utility service is interrupted.

1621.3.8 Storage tanks. Storage shall be designed to meet the general requirements of Sections 1622.1 and 1622.2 and the specific requirements of Section 1622.4.3.

TABLE 1621.3
MECHANICAL AND ELECTRICAL COMPONENTS COEFFICIENTS

MECHANICAL AND ELECTRICAL COMPONENT OR ELEMENT	COMPONENT AMPLIFICATION FACTOR a_p[a]	COMPONENT RESPONSE MODIFICATION FACTOR R_p
1. General mechanical		
a. Boilers and furnaces	1.0	2.5
b. Pressure vessels on skirts and free-standing	2.5	2.5
c. Stacks	2.5	2.5
d. Cantilevered chimneys	2.5	2.5
e. Other	1.0	2.5
2. Manufacturing and process machinery		
a. General	1.0	2.5
b. Conveyors (nonpersonnel)	2.5	2.5
3. Piping systems		
a. High deformability elements and attachments	1.0	3.5
b. Limited deformability elements and attachments	1.0	2.5
c. Low deformability elements or attachments	1.0	1.25
4. HVAC system equipment		
a. Vibration isolated	2.5	2.5
b. Nonvibration isolated	1.0	2.5
c. Mounted in-line with ductwork	1.0	2.5
d. Other	1.0	2.5
5. Elevator components	1.0	2.5
6. Escalator components	1.0	2.5
7. Trussed towers (free-standing or guyed)	2.5	2.5
8. General electrical		
a. Distributed systems (bus ducts, conduit, cable tray)	1.0	3.5
b. Equipment	1.0	2.5
9. Lighting fixtures	1.0	1.25

a. Where justified by detailed dynamic analyses, a lower value of a_p is permitted, but shall not be less than 1. The reduced value of a_p shall be between 2.5, assigned to flexible or flexibly attached equipment, and 1, assigned to rigid or rigidly attached equipment.

1621.3.9 HVAC ductwork. Attachments and supports for HVAC ductwork systems shall be designed to meet the force and displacement requirements of Sections 1621.1.4 and 1621.1.5 and the additional requirements of this section. Ductwork systems designated as having an I_p greater than 1.0 shall be designed to meet the force and displacement requirements of Sections 1621.1.4 and 1621.1.5. Where HVAC ductwork runs between structures that could displace relative to one another and for seismically isolated structures where the HVAC ductwork crosses the seismic isolation interface, the HVAC ductwork shall be designed to accommodate the seismic relative displacements specified in Section 1621.1.5.

Seismic restraints are not required for HVAC ducts with $I_p = 1.0$, provided that either of the following conditions are met for the full length of each duct run:

1. HVAC ducts are suspended from hangers and hangers are 12 inches (305 mm) or less in length from the top of the duct to the supporting structure and the hangers are detailed to avoid significant bending of the hangers and their connections.
2. HVAC ducts have a cross-sectional area of less than 6 square feet (0.557 m^2).

HVAC duct systems fabricated and installed in accordance with the SMACNA duct construction standards (SMACNA-HVAC and SMACNA-Seismic) and including Appendix B of the SMACNA Seismic Restraint Manual Guidelines for Mechanical Systems shall be deemed to meet the lateral bracing requirements of this section.

Equipment items installed in-line with the duct system (e.g., fans, heat exchangers and humidifiers) with an operating weight greater than 75 pounds (334 N) shall be supported and laterally braced independently of the duct system and shall meet the force requirements of Section 1621.1.4. Appurtenances such as dampers, louvers and diffusers shall be positively attached with mechanical fasteners. Unbraced piping attached to in-line equipment shall be provided with adequate flexibility to accommodate differential displacements.

1621.3.10 Piping systems. Attachments and supports for piping systems shall be designed to meet the force and displacement requirements of Sections 1621.1.4 and 1621.1.5 and the additional requirements of this section. Piping systems designated as having I_p greater than 1.0 shall themselves be designed to meet the force and displacement requirements of Sections 1621.1.4 and 1621.1.5 and the following requirements. Where piping systems are attached to structures that could displace relative to one another and for seismically isolated structures where the piping system crosses the seismic isolation interface, the piping system shall be designed to accommodate the seismic relative displacements specified in Section 1621.1.5.

Seismic effects that shall be considered in the design of a piping system include the dynamic effects of the piping system, its contents and its supports. The interaction between the piping system and the supporting structures, including other mechanical and electrical equipment, shall be considered.

See Section 1621.3.14 for elevator system piping requirements.

1621.3.10.1 Fire-protection sprinkler systems. Fire-protection sprinkler systems designed and constructed in accordance with NFPA 13, shall be deemed to meet the force, displacement and other requirements of this section provided that the seismic design force and displacement calculated in accordance with NFPA 13 when multiplied by a factor of 1.4 are determined to be not less than that prescribed by this code.

1621.3.10.2 Other piping systems. The following documents shall be used for the seismic design of the respective systems. The seismic design force and displacement used shall not be less than that determined using Section 1621.1.

ASME B31.1 Power Piping
ASME B31.3 Process Piping
ASME B31.4 Liquid Transportation Systems for Hydrocarbons, Liquid Petroleum Gas, Anhydrous Ammonia, and Alcohols
ASME B31.5 Refrigeration Piping
ASME B31.9 Building Services Piping
ASME B31.11 Slurry Transportation Piping System
ASME B31.8 Gas Transmission and Distribution Piping Systems

Exception: Piping systems designated as having an I_p greater than 1.0 shall not be designed using the simplified analysis procedures of ASME 31.9, Section 919.4.1(a).

The following requirements shall be met for piping systems designated as having an I_p greater than 1.0:

1. Under design loads and displacements, piping shall not be permitted to impact other components.
2. Piping shall accommodate the effects of relative displacement that can occur between piping support points on the structure or the ground, other mechanical and/or electrical equipment, and other piping.

1621.3.10.2.1 Supports and attachments for other piping. In addition to meeting the force, displacement and other requirements of this section, attachments and supports for piping shall be subject to the following other requirements and limitations:

1. Attachments shall be designed in accordance with Section 1621.1.7.
2. Seismic supports are not required for:
   - 2.1. Piping supported by rod hangers provided that hangers in the pipe run are 12 inches (305 mm) or less in length from the top of the pipe to the supporting structure and the pipe can accommodate the expected deflections. Rod hangers shall not be constructed in a manner that would subject the rod to bending moments.

2.2. High deformability piping in Seismic Design Category D, E or F, as determined in Section 1616, designated as having an I_p greater than 1.0 and a nominal pipe size of 1 inch (25 mm) or less where provisions are made to protect the piping from impact or to avoid the impact of larger piping or other mechanical equipment.

2.3. High deformability piping in Seismic Design Category C, as determined in Section 1616, designated as having an I_p greater than 1.0 and a nominal pipe size of 2 inches (51 mm) or less where provisions are made to protect the piping from impact or to avoid the impact of larger piping or other mechanical equipment.

2.4. High deformability piping in Seismic Design Category D, E or F designated as having an I_p equal to 1.0 and a nominal pipe size of 3 inches (76 mm) or less.

3. Seismic supports shall be constructed so that support engagement is maintained.

1621.3.11 Boilers and pressure vessels. Attachments and supports for boilers and pressure vessels shall be designed to meet the force and displacement requirements of Sections 1621.1.4 and 1621.1.5 and the additional requirements of this section. Boilers and pressure vessels designated as having an $I_p = 1.5$ themselves shall be designed to meet the force and displacement requirements of Sections 1621.1.4 and 1621.1.5.

Seismic effects that shall be considered in the design of a boiler or pressure vessel include the dynamic effects of the boiler or pressure vessel, its contents, and its supports; sloshing of liquid contents; loads from attached components such as piping; and the interaction between the boiler or pressure vessel and its support.

1621.3.11.1 ASME boilers and pressure vessels. Boilers or pressure vessels designed in accordance with ASME BPVC, shall be deemed to meet the force, displacement and other requirements of this section. In lieu of the specific force and displacement provisions provided in ASME BPVC, the force and displacement requirements of Sections 1621.1.4 and 1621.1.5 shall be used.

1621.3.11.2 Other boilers and pressure vessels. Boilers and pressure vessels designated as having an I_p = 1.5, but not constructed in accordance with Section 1621.3.11.1 shall meet the following provisions:

1. The design strength for seismic loads in combination with other service loads and appropriate environmental effects shall not exceed the following:

   1.1. For boilers and pressure vessels constructed with ductile materials (e.g., steel, aluminum or copper), 90 percent of the material minimum specified yield strength.

   1.2. For threaded connections in boilers or pressure vessels or their supports constructed with ductile materials, 70 percent of the material minimum specified yield strength.

   1.3. For boilers and pressure vessels constructed with nonductile materials (e.g., plastic, cast iron, or ceramics), 25 percent of the material minimum specified tensile strength.

   1.4. For threaded connections in boilers or pressure vessels or their supports constructed with nonductile materials, 20 percent of the material minimum specified tensile strength.

2. Where boiler and pressure vessel components are constructed of nonductile materials or in cases where material ductility is reduced (e.g., low temperature applications), the load effects shall include possible seismic impact from other components.

3. Boilers and pressure vessels shall be designed to resist interaction effects between them and other constructions.

1621.3.11.3 Supports and attachments for boilers and pressure vessels. Attachments and supports for boilers and pressure vessels shall meet the following requirements:

1. Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and designed and constructed in accordance with the appropriate requirements of this code for the material being used.

2. Attachments embedded in concrete shall be able to support for cyclic loads.

3. Seismic supports shall be constructed so that support engagement is maintained.

1621.3.12 Mechanical equipment attachments and supports. Attachments and supports for mechanical equipment not covered in the preceding sections shall be designed to meet the force and displacement requirements of Sections 1621.1.4 and 1621.1.5 and the additional requirements of this section. Components of mechanical equipment designated as having an I_p greater than 1.0, which contain hazardous or flammable materials in quantities that exceed the exempted amounts for an open system listed in Chapter 4, shall be designed to meet the force and displacement requirements of Sections 1621.1.4 and 1621.1.5 and the additional requirements of this section.

Seismic effects that shall be considered in the design of mechanical equipment, attachments and their supports include the dynamic effects of the equipment, its contents, and its supports. The interaction between the equipment and the supporting structures, including other mechanical and electrical equipment, shall also be included.

1621.3.12.1 Mechanical equipment. Mechanical equipment having an I_p greater than 1.0 shall meet the following requirements:

1. For equipment components vulnerable to impact, equipment components constructed of nonductile materials, or in cases where material ductility is reduced (e.g., low temperature applications), seismic impact shall be prevented.
2. The design shall include the effect of loadings imposed on the equipment by attached utility or service lines due to differential motions of points of support from separate structures.

Components of mechanical equipment designated as having an I_p greater than 1.0, which contain hazardous or flammable materials in quantities that exceed the exempted amounts for an open system listed in Chapter 4, shall be designed for seismic loads. The design strength for seismic loads in combination with other service loads and environmental effects such as corrosion shall be based on the following:

1. For mechanical equipment constructed with ductile materials (e.g., steel, aluminum, or copper), 90 percent of the equipment material minimum specified yield strength.
2. For threaded connections in equipment constructed with ductile materials, 70 percent of the material minimum specified yield strength.
3. For mechanical equipment constructed with nonductile materials (e.g., plastic, cast iron or ceramics), 25 percent of the equipment material minimum tensile strength.
4. For threaded connections in equipment constructed with nonductile materials, 20 percent of the material minimum specified yield strength.

1621.3.12.2 Attachments and supports for mechanical equipment. Attachments and supports for mechanical equipment shall meet the following requirements:

1. Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and designed and constructed in accordance with the requirements of this code for the material in question.
2. Friction clips shall not be used for anchorage attachment.
3. Expansion anchors shall not be used for non- vibration-isolated mechanical equipment rated over 10 horsepower (7.45 kW).

   Exception: Use of undercut expansion anchors is permitted.
4. Supports shall be specifically evaluated if weak-axis bending of cold-formed support steel is relied on for the seismic load path.
5. Components mounted on vibration isolation systems shall have a bumper restraint or snubber in each horizontal direction, and vertical restraints shall be provided where required to resist overturning. Isolator housings and restraints shall be constructed of ductile materials. (See additional design force requirements in Table 1621.2.) A viscoelastic pad or similar material of appropriate thickness shall be used between the bumper and equipment item to limit the impact load.
6. Seismic supports shall be constructed so that support engagement is maintained.

1621.3.13 Electrical equipment attachments and supports. Attachments and supports for electrical equipment shall be designed to meet the force and displacement requirements of Sections 1621.1.4 and 1621.1.5 and the additional requirements of this section. Electrical equipment designated as having an I_p greater than 1.0 shall itself be designed to meet the force and displacement requirements of Sections 1621.1.4 and 1621.1.5 and the additional following requirements.

Seismic effects that shall be considered in the design of other electrical equipment include the dynamic effects of the equipment, its contents and its supports. The interaction between the equipment and the supporting structures, including other mechanical and electrical equipment, shall be considered. Where conduit, cable trays or similar electrical distribution components are attached to structures that could displace relative to one another and for seismically isolated structures where the conduit or cable trays cross the seismic isolation interface, the conduit or cable trays shall be designed to accommodate the seismic relative displacement specified in Section 1621.1.5.

1621.3.13.1 Electrical equipment. Electrical equipment designated as having an I_p greater than 1.0 shall meet the following requirements:

1. Seismic impact between the equipment and other components shall be prevented.
2. The design load shall include the effects of loading on the equipment imposed by attached utility or service lines that are also attached to separate structures.
3. Batteries on racks shall have wrap-around restraints designed to prevent batteries from falling off the rack. Spacers shall be used between restraints and cells to prevent damage to cases. Racks shall have sufficient lateral load capacity.
4. Internal coils of dry-type transformers shall be positively attached to their supporting substructure within the transformer enclosure.
5. Slide-out components in electrical control panels, computer equipment, etc., shall have a latching mechanism to hold contents in place.
6. Electrical cabinet design shall conform to NEMA 250 and NEMA ICS6 standards. Cut-outs in the lower shear panel that do not appear to have been made by the manufacturer and are judged to significantly reduce the strength of the cabinet are not permitted unless analysis demonstrates that the remaining strength is sufficient.

7. The attachment of additional external items weighing more than 100 pounds (445 N) is not permitted unless such items were either provided by the manufacturer or analysis shows that their effects are supported by the design.

1621.3.13.2 Attachments and supports for electrical equipment. Attachments and supports for electrical equipment shall meet the following requirements:

1. Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and designed and constructed in accordance the requirements of this code for the appropriate material.
2. Friction clips shall not be used for anchorage attachment.
3. Oversized plate washers extending to the equipment wall shall be used at bolted connections through the base sheet metal if the base is not reinforced with stiffeners or not capable of transferring the required loads.
4. Weak-axis bending of light gage support steel is not permitted in the seismic load path unless analysis shows that it can support the design load.
5. The supports for linear electrical equipment such as cable trays, conduit and bus ducts shall be designed to meet the force and displacement requirements of Sections 1621.1.4 and 1621.1.5 if any of the following situations apply:
   5.1. Supports are cantilevered up from the floor.
   5.2. Supports include bracing to limit deflection.
   5.3. Supports are constructed as rigid welded frames.
   5.4. Attachments into concrete utilize nonexpanding insets, shot pins or cast-iron embedments.
   5.5. Attachments utilize spot welds, plug welds, or minimum size welds as defined by AISC LRFD.
6. Components mounted on vibration isolation systems shall have a bumper restraint or snubber in each horizontal direction, and vertical restraints shall be provided where required to resist overturning. Isolator housings and restraints shall be constructed of ductile materials. (See additional design force requirements in Table 1621.3.) A viscoelastic pad or similar material of appropriate thickness shall be used between the bumper and the equipment item to limit the impact load.

1621.3.14 Elevator design requirements. Elevators shall meet the force and displacement requirements of Section 1621.3.1 unless exempted by Section 1621. Elevators designed in accordance with the seismic requirements of ASME A17.1 shall be deemed to meet the seismic force requirements of this section, provided that they meet the requirements provided in Sections 1621.3.14.1 through 1621.3.14.4.

1621.3.14.1 Elevators and hoistway structural systems. Elevators and hoistway structural systems shall be designed to meet the force and displacement requirements of Section 1621.3.1.

1621.3.14.2 Elevator machinery and controller supports and attachments. Elevator machinery and controller supports and attachments shall be designed to meet the force and displacement provisions of Section 1621.3.1.

1621.3.14.3 Seismic controls. Seismic switches shall be provided for nonexempt elevators that operate at a speed of 150 feet per minute (46 m/min) or greater. Seismic switches shall provide an electrical signal indicating that structural motions are of such a magnitude that the operation of elevators may be impaired. Upon activation of the seismic switch, elevator operations shall conform to provisions in ASME A17.1 except as noted below. The seismic switch shall be located at or above the highest floor serviced by the elevators. The seismic switch shall have two horizontal perpendicular axes of sensitivity. Its trigger level shall be set to 30 percent of the acceleration of gravity.

In facilities where the loss of the use of an elevator is a life-safety issue, the elevator is permitted to be used after the seismic switch has triggered provided that:

1. The elevator shall operate no faster than the service speed.
2. The elevator shall be operated remotely from top to bottom and back to top to verify that it is operable.

1621.3.14.4 Retainer plates. Retainer plates are required at the top and bottom of the car and its counterweight.

SECTION 1622
NONBUILDING STRUCTURES SEISMIC DESIGN REQUIREMENTS

1622.1 Nonbuilding structures. The requirements of this section apply to self-supporting structures that carry gravity loads that are not defined as buildings, vehicular or railroad bridges, nuclear power generation plants, offshore platforms, or dams. Where the building official has approved the use of specific industry standards for seismic design of nonbuilding structures, those standards shall be applicable within the limitations of the requirements of this section. Nonbuilding structures that are beyond the scope of this section shall be designed by specific industry standards approved by the building official.

Nonbuilding structures shall be designed and detailed to provide sufficient stiffness, strength and ductility to resist the effects of seismic ground motion specified in Section 1622. Design shall conform to the applicable requirements of other sections of this code except as modified by Section 1622. Applicable strength and other design criteria shall be obtained from other sections of this code or its referenced standards. When applicable strength and other design criteria are not contained in or referenced by this code, such criteria shall be obtained from standards approved for use for this purpose by the building official. Where acceptance criteria are defined in terms of allowable stresses as opposed to strength, the design seismic forces calculated in accordance with Section 1622 shall be reduced by a factor of 1.4. Allowable stress increases used in the approved standards are permitted. Detailing shall be in accordance with the approved standards.

1622.1.1 Nonbuilding structures supported by other structures. If a nonbuilding structure is supported above the base by another structure and the weight of the nonbuilding structure is less than 25 percent of the combined weight of the nonbuilding structure and the supporting structure, the design seismic forces of the supported nonbuilding structure shall be determined in accordance with the requirements of Section 1621.1.2.

If the weight of a nonbuilding structure is 25 percent or more of the combined weight of the nonbuilding structure and the supporting structure, the design seismic forces of the nonbuilding structure shall be determined based on the combined nonbuilding structure and supporting structural system. For supported nonbuilding structures that have component dynamic characteristics that are not rigid as defined in Section 1622.2.5, the combined system *R* factor shall be a maximum of 3. For supported nonbuilding structures that have rigid component dynamic characteristics, the combined system *R* factor shall be the value of the supporting structural system. The supported nonbuilding structure and attachments shall be designed for the forces determined for the nonbuilding structure in a combined systems analysis.

1622.1.2 Architectural, mechanical and electrical components. Architectural, mechanical and electrical components supported by nonbuilding structures shall be designed in accordance with Section 1621.

1622.2 Seismic design requirements. Design of nonbuilding structures to resist seismic loads shall conform to this section.

1622.2.1 Weight. The weight, *W*, for nonbuilding structures shall include dead loads as defined for buildings in Section 1617.4.1. For purposes of calculating design seismic forces in nonbuilding structures, *W* also shall include normal operating contents for items such as tanks, vessels, and bins and the contents of piping. *W* shall include snow and ice loads when these loads constitute 25 percent or more of *W*.

1622.2.2 Fundamental period. The fundamental period of the nonbuilding structure shall be determined by methods as prescribed in Section 1617.4.2 or by using other rational methods.

1622.2.3 Drift limits. The drift limitations of Section 1617.3 need not apply to nonbuilding structures if a rational analysis indicates they may be exceeded without adversely affecting structural stability. P-delta effects shall be considered when critical to the function or stability of the nonbuilding structure.

1622.2.4 Seismic requirements of materials. The seismic requirements of materials as contained in Chapters 19 through 23 shall be applicable unless specifically exempted in this section.

1622.2.5 Minimum seismic forces. Nonbuilding structures shall be designed to resist minimum seismic lateral forces not less than the requirements of Section 1617.4.1 and the following:

1. The response modification coefficient, *R*, shall be the lesser of the values given in Table 1622.2.5(1) or the values in Table 1617.6.
2. For nonbuilding systems with response modification coefficients, *R*, provided in Table 1622.2.5(1), the minimum value specified in Equation 16-37 shall be replaced by the following:

   $$C_s = 0.14 S_{DS} I \qquad \textbf{(Equation 16-75)}$$

   and the minimum value specified in Equation 16-38 shall be replaced by the following:

   $$C_s = \frac{0.8 S_1 I}{R} \qquad \textbf{(Equation 16-76)}$$

3. The overstrength factor, Ω_0, shall be as given in Table 1622.2.5(1).
4. The importance factor, *I*, shall be as given in Table 1622.2.5(2).
5. The height limitations shall be as given in Table 1622.2.5(1).
6. The vertical distribution of the lateral seismic forces in nonbuilding structures covered by this section shall be determined in accordance with the requirements of Section 1617.4.3 or 1618.5 or with an approved standard.

Exceptions:

1. At sites where the design spectral response acceleration at short period, S_{DS}, as determined in Section 1615.1.3, is greater than or equal to 0.50, irregular structures per Section 1616.5 that cannot be modeled as a single mass shall use the procedures of Section 1618.
2. When an approved standard provides a basis for the earthquake-resistant design of a specific nonbuilding structure, such a standard is permitted to be used subject to the following limitations:
   - 2.1. Where seismic force is determined in accordance with an approved national standard, it shall not be taken as less than 80 percent of that in Section 1622.2.5.
   - 2.2. The seismic ground acceleration and seismic coefficients shall be in conformance with the requirements of Sections 1615 and 1616, respectively.
   - 2.3. The value for total lateral force and total base overturning moment used in design shall not be less than 80 percent of the base shear value and overturning moment, each adjusted for the effects of soil-structure interaction that would be obtained using Section 1617 of this code.
3. The base shear is permitted to be reduced in accordance with Section 1619 to account for the effects of soil structure interaction. In no case shall the reduced base shear be less than 0.7*V*.

1622.2.6 Rigid nonbuilding structures. Nonbuilding structures that have a fundamental period, T, less than 0.06 second, including their anchorages, shall be designed for the lateral force obtained from the following:

$$V = 0.3 S_{DS} W I \qquad \textbf{(Equation 16-77)}$$

where:

I = The importance factor as defined in Table 1622.2.5(2).

S_{DS} = The site design response acceleration as determined from Section 1615.1.

V = The total design lateral seismic base shear force applied to a nonbuilding structure.

W = Nonbuilding structure operating weight as defined in Section 1622.2.1.

The force shall be distributed with height in accordance with Section 1617.4.3.

1622.2.7 Deflection limits and structure separation. Deflection limits and structure separation shall be determined in accordance with Section 1617.3 unless specifically amended.

1622.3 Nonbuilding structures similar to buildings. Nonbuilding structures that have structural systems that are designed and constructed in a manner similar to buildings and have a dynamic response similar to buildings shall be designed for seismic loads as for buildings or comparable material, with the exceptions contained in this section.

1622.3.1 Design basis. This general category of nonbuilding structures shall be designed in accordance with Sections 1616.1 and 1622.2. The lateral force design procedure for nonbuilding structures similar to buildings shall be selected in accordance with the force and detailing requirements of Section 1616.1.

Exception: Intermediate moment frames of reinforced concrete in Seismic Use Group I or II are permitted to be used at sites where the seismic coefficient S_{DS}, as determined in Section 1615.1 or 1615.2, is greater than or equal to 0.50 provided that both of the following conditions are met:

1. The nonbuilding structure is less than 50 feet (15 240 mm) in height.
2. $R \leq 3.0$ is used for design.

1622.3.2 Determination of seismic effect *E*. The seismic effect, E, to be used in the load combinations specified in Section 1605 shall be determined in accordance with Section 1617.1.

1622.3.3 Pipe racks. Pipe racks supported at the base shall be designed to meet the force requirements of Section 1617.4 or 1617.5. Friction resulting from gravity loads shall not be considered to provide resistance to seismic forces.

Displacements of the pipe rack and potential for interaction effects (pounding of the piping system) shall be considered using the amplified deflections obtained from the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad \textbf{(Equation 16-78)}$$

where:

C_d = The deflection amplification factor in Table 1622.2.5(1).

I = Occupancy importance factor determined from Table 1622.2.5(2).

δ = The deflections determined using the prescribed seismic design forces.

Exception: The occupancy importance factor, I, used in the computation of δ_{xe} shall be determined from Table 1622.2.5(2).

The seismic design of piping systems and their attachments is governed by Section 1621.

1622.3.4 Steel storage racks. Steel storage racks shall be designed in accordance with Section 2210 and Sections 1622.3.4.1 through 1622.3.4.4 or, alternatively, with the method detailed in Section 2.7, Earthquake Forces of the RMI Standard Specification for the Design Testing and Utilization of Industrial Steel Storage Racks. When determining the value of C_S, for steel storage racks suppported at the base in Section 2.7.3 of the RMI Standard specification, the value of C_a is taken as equal to $S_{DS}/2.5$ and the value of C_V is taken as equal to S_{D1} and where the value of I_p shall not be taken as less than that required by Section 1621.1.6. In addition, the value of C_S in the RMI Standard specification shall not be taken as less than 0.14 S_{DS}.

Also, for storage racks supported above grade, the value of C_S in the RMI Standard specification shall not be less than the value determined for F_P in accordance with Section 1621.1.4 of this code, with R_P taken as equal to R and a_p taken as equal to 2.5.

1622.3.4.1 General. Storage racks shall meet the force requirements of Section 1621.1.4, with R_P taken as equal to R, and a_P taken as equal to 2.5.

Exception: Storage racks supported at the base need not meet the force requirements of Section 1621.1.4 provided that they are designed as structures with an R of 4 or less that meet the requirements of Section 1616.1. Higher values of R are permitted provided that the detailing requirements of AISC Seismic are met.

1622.3.4.2 Operating weight. Storage racks shall be designed for each of the following conditions of operating weight, W or W_p:

1. Weight of the rack plus every storage level loaded to 67 percent of its rated load capacity.
2. Weight of the rack plus the highest storage level only loaded to 100 percent of its rated load capacity.

The design shall consider the actual height of the center of mass of each storage load component.

1622.3.4.3 Vertical distribution of seismic forces. For storage racks, the vertical distribution of seismic forces shall be as specified in Section 1617.4.3 and with the following:

1. The base shear V of the structure shall be the base shear of the storage rack when loaded in accordance with Section 1621.2.8.
2. The base of the structure shall be the floor supporting the storage rack. Each storage level of the rack shall be treated as a level of the structure with heights, h_i and h_x, measured from the base of the structure.
3. The factor, k, shall be taken as 1.0.
4. The importance factor, I, shall be taken from Section 1621.1.6.

1622.3.4.4 Seismic displacements. Storage rack installations shall accommodate the seismic displacement of storage racks and their contents relative to adjacent or attached components and elements. The assumed total relative displacement for storage racks shall not be less than 5 percent of the height consideration above the base.

1622.3.5 Electrical power-generating facilities. As used in this section, the term "electrical power-generating facilities" means power plants that generate electricity by steam turbines, combustion turbines, diesel generators or similar turbo machinery. The seismic design of electrical power-generating facilities shall be designed as for building structures using the appropriate factors contained in Section 1622.

1622.3.6 Structural towers for tanks and vessels. The seismic design of structural towers that support tanks and vessels shall meet the requirements of Section 1622.1.1. In addition, the following requirements shall be met:

1. The distribution of the lateral base shear from the tank or vessel onto the supporting structure shall accommodate the relative stiffness of the tank and resisting structural elements.
2. The distribution of the vertical reactions from the tank or vessel onto the supporting structure shall accommodate the relative stiffness of the tank and resisting structural elements. When the tank or vessel is supported on grillage beams, the calculated vertical reaction due to weight and overturning shall be increased at least 20 percent to account for nonuniform support. The grillage beam and vessel attachment shall be designed for this increased design value.
3. Seismic displacements of the tank and vessel shall consider the deformation of the support structure when determining P-delta effects or evaluating required clearances to prevent pounding of the tank on the structure.
4. When the sloshing period of the stored liquid is within 70 percent to 150 percent of the fundamental period of the supporting structure, the effects of sloshing shall be included in the design of the tank and supporting structure.

1622.3.7 Piers and wharves. As used in this section, the terms "piers" and "wharves" are defined as structures located in waterfront areas that project into a body of water or parallel the shoreline. The seismic design of piers and wharves shall be as for buildings. Seismic forces on elements below the water level shall include the inertial force of the mass of the displaced water. The additional seismic mass equal to the mass of the displaced water shall be included as a lumped mass on the submerged element, and shall be added to the calculated seismic forces of the pier or wharf structure. Seismic dynamic forces from the soil shall be determined by a registered design professional. The design shall account for the effect of liquefaction on piers and wharfs.

1622.4 Nonbuilding structures not similar to buildings. Nonbuilding structures that have structural systems that are designed and constructed in a manner such that the dynamic response is not similar to buildings shall be designed for seismic resistance in compliance with this section.

1622.4.1 Determination of seismic effect, E. The seismic effect, E, to be used in the load combinations specified in Section 1605 shall be determined in accordance with Section 1617.1, with the exception that the redundancy-based reliability coefficient, ρ, shall be taken as 1.0.

1622.4.2 Earth-retaining structures. The seismic forces and design methodology shall be determined in accordance with a geotechnical analysis prepared by a registered design professional.

The seismic use group shall be determined by the proximity of the retaining wall to other nonbuilding structures or buildings. If failure of the retaining wall would affect an adjacent structure, the Seismic Use Group shall not be less than that of the adjacent structure, as determined in Section 1616. Earth-retaining walls are permitted to be designed for seismic loads as either yielding or nonyielding walls. Cantilevered reinforced concrete retaining walls shall be assumed to be yielding walls and shall be designed as simple flexural wall elements.

1622.4.3 Tanks and vessels. This section applies to tanks and vessels storing liquids, gases, and granular solids. Tanks and vessels covered herein include reinforced concrete, prestressed concrete, steel, and fiber-reinforced plastic materials. Tanks and vessels shall be designed in accordance with this code and shall be designed to resist seismic lateral forces determined from a substantiated analysis approved by the building official.

TABLE 1622.2.5(1) SEISMIC COEFFICIENTS FOR NONBUILDING STRUCTURES

NONBUILDING STRUCTURE TYPE	RESPONSE MODIFICATION COEFFICIENT R	SYSTEM OVER-STRENGTH FACTOR Ω_o	DEFLECTION AMPLIFICATION FACTOR C_d	STRUCTURAL SYSTEM AND HEIGHT LIMITS[c] (feet) Seismic design category as determined in Section 1616: B	C	D	E or F
1. Nonbuilding frame systems:							
a. Concentric braced frame of steel	See Table 1617.6			NL	NL	NL	NL
b. Special concentric braced frames of steel				NL	NL	NL	NL
2. Moment-resisting frame systems:							
a. Special moment frames of steel	See Table 1617.6			NL	NL	NL	NL
b. Ordinary moment frames of steel				NL	NL	50	50
c. Special moment frames of concrete				NL	NL	NL	NL
d. Intermediate moment frames of concrete				NL	NL	50	50
3. Ordinary moment frames of concrete				NL	50	NP	NP
4. Steel storage racks	4	2	$3\frac{1}{2}$	NL	NL	NL	NL
5. Elevated tanks, vessels, bins or hoppers[a]							
a. On braced legs	3	2	$2\frac{1}{2}$	NL	NL	NL	NL
b. On unbraced legs	3	2	$2\frac{1}{2}$	NL	NL	NL	NL
c. Irregular braced legs single pedestal or skirt supported	2	2	2	NL	NL	NL	NL
d. Welded steel	2	2	2	NL	NL	NL	NL
e. Concrete	2	2	2	NL	NL	NL	NL
6. Horizontal, saddle supported welded steel vessels	3	2	$2\frac{1}{2}$	NL	NL	NL	NL
7. Tanks or vessels supported on structural towers similar to buildings	3	2	2	NL	NL	NL	NL
8. Flat bottom, ground supported tanks, or vessels:							
a. Anchored (welded or bolted steel)	3	2	$2\frac{1}{2}$	NL	NL	NL	NL
b. Unanchored (welded or bolted steel)	$2\frac{1}{2}$	2	2	NL	NL	NL	NL
9. Reinforced or prestressed concrete:							
a. Tanks with reinforced nonsliding base	2	2	2	NL	NL	NL	NL
b. Tanks with anchored flexible base	3	2	2	NL	NL	NL	NL
10. Tanks with unanchored and unconstrained:							
a. Flexible base	$1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{2}$	NL	NL	NL	NL
b. Other material	$1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{2}$	NL	NL	NL	NL
11. Cast-in-place concrete silos, stacks and chimneys having walls continuous to the foundation	3	$1\frac{3}{4}$	3	NL	NL	NL	NL
12. Other reinforced masonry structures	3	2	$2\frac{1}{2}$	NL	NL	50	50
13. Other nonreinforced masonry structures	$1\frac{1}{4}$	2	$1\frac{1}{2}$	NL	50	50	50
14. Other steel and reinforced concrete distributed mass cantilever structures not covered herein including stacks, chimneys, silos, and skirt-supported vertical vessels	3	2	$2\frac{1}{2}$	NL	NL	NL	NL
15. Trussed towers (freestanding or guyed), guyed stacks and chimneys	3	2	$2\frac{1}{2}$	NL	NL	NL	NL
16. Cooling towers:							
a. Concrete or steel	$3\frac{1}{2}$	$1\frac{3}{4}$	3	NL	NL	NL	NL
b. Wood frame	$3\frac{1}{2}$	3	3	NL	NL	50	50
17. Telecommunication towers							
a. Truss: Steel	3	$1\frac{1}{2}$	3	NL	NL	NL	NL
b. Pole: Steel	$1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{2}$	NL	NL	NL	NL
Wood	$1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{2}$	NL	NL	NL	NL
Concrete	$1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{2}$	NL	NL	NL	NL
c. Frame: Steel	3	$1\frac{1}{2}$	$1\frac{1}{2}$	NL	NL	NL	NL
Wood	$2\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{2}$	NL	NL	NL	NL
Concrete	2	$1\frac{1}{2}$	$1\frac{1}{2}$	NL	NL	NL	NL
18. Amusement structures and monuments	2	2	2	NL	NL	NL	NL
19. Inverted pendulum-type structures (not elevated tank)[b]	2	2	2	NL	NL	NL	NL
20. Signs and billboards	$3\frac{1}{2}$	$1\frac{3}{4}$	3	NL	NL	NL	NL
21. Other self-supporting structures, tanks or vessels not covered above	$1\frac{1}{4}$	2	$2\frac{1}{2}$	NL	50	50	50

For SI: 1 foot = 304.8 mm.
NL = No limit.
NP = Not permitted.

a. Support towers similar to building-type structures, including those with irregularities (see Section 1616.5 for definition of irregular structures) shall comply with the requirements of Section 1617.6.3 for Seismic Design Category F structures.
b. Light posts, stoplight, etc.
c. Above base.

**TABLE 1622.2.5(2)
IMPORTANCE FACTOR (*I*) AND SEISMIC USE GROUP CLASSIFICATION
FOR NONBUILDING STRUCTURES**

IMPORTANCE FACTOR	I = 1.0	I = 1.25	I = 1.5
Seismic Use Group as Determined in Section 1616.2	I	II	III
Hazard	H-I	H-II	H-III
Function	F-I	–	F-III

H-I = The stored product is biologically or environmentally benign; low fire or low physical hazard.
H-II = The stored product is rated low explosion, moderate fire or moderate physical hazard as determined by the authority having jurisdiction.
H-III = The stored product is rated high or moderate explosion hazard, high fire hazard or high physical hazard as determined by the authority having jurisdiction.
F-I = Nonbuilding structures not classified as F-III.
F-III = Seismic Use Group III nonbuilding structures or designated ancillary nonbuilding structures (such as communication towers, fuel storage tanks, cooling towers or electrical substation structures) required for operation of Seismic Use Group III structures.

1622.4.3.1 Above-grade storage tanks. For storage tanks mounted above grade in structures, the attachments, supports and the tank shall be designed to meet the force requirements of Section 1621.1.4, with R_p taken equal to R specified in Section 1622. The weight of the storage tank, W_p, shall include the weight of the tank structure and appurtenances and the operating weight of the contents at maximum rated capacity.

When the sloshing period of the stored liquid is within 70 percent to 150 percent of the fundamental period of the supporting structure, the effects of sloshing shall be included in the design of the tank and supporting structure.

1622.4.3.2 At-grade storage tanks. Storage tanks mounted at the base shall be designed to meet the design requirements of Section 1622. In addition, for sites where S_{DS} is greater than 0.60, as determined in Section 1615.1.3, flat-bottom tanks designated with an I_p greater than 1.0 and tanks greater than 20 feet (6096 mm) in diameter and tanks that have a height-to-diameter ratio greater than 1.0 shall be designed to meet the following requirements:

1. The tank shall be designed to resist the effects of sloshing.
2. Piping connections to steel flat-bottom storage tanks shall be designed to either resist or survive without damage the potential uplift of the tank when it is subjected to the design earthquake spectral accelerations determined in Section 1615.1. Unless otherwise calculated, the following displacements shall be assumed for side-wall connections and bottom penetrations:
   - 2.1. Vertical displacement of 2 inches (51 mm) for anchored tanks.
   - 2.2. Vertical displacement of 12 inches (305 mm) for unanchored tanks.
   - 2.3. Horizontal displacement of 8 inches (203 mm) for unanchored tanks with a diameter of 40 feet (12 192 mm) or less.

1622.4.4 Telecommunication towers. Self-supporting and guyed telecommunication towers shall be designed to resist seismic lateral forces determined from a substantiated analysis using standards approved by the building official.

1622.4.5 Stacks and chimneys. Steel stacks, concrete stacks, steel chimneys, concrete chimneys and liners shall be designed to resist seismic lateral forces determined from a substantiated analysis using standards approved by the building official. Interaction of the stack or chimney with the liners, if present, shall be included in the design. A minimum separation shall be provided between the liner and chimney equal to C_d times the calculated differential lateral drift, where C_d is as specified in Table 1622.2.5(1).

1622.4.6 Amusement structures. Permanently fixed amusement structures shall be designed to resist minimum seismic lateral forces determined from a substantiated analysis using standards approved by the building official.

1622.4.7 Special hydraulic structures. As used in this section, the term "special hydraulic structures" means structures that are contained inside liquid-containing structures. These structures are exposed to liquids on both wall surfaces at the same head elevation under normal operating conditions. Special hydraulic structures are subjected to out-of-plane forces only during an earthquake when the structure is subjected to differential hydrodynamic fluid forces. Examples of special hydraulic structures include separation walls, baffle walls, weirs and other similar structures. Special hydraulic structures shall be designed for out-of-phase movement of the fluid. Unbalanced forces from the motion of the liquid shall be applied simultaneously "in front of" and "behind" these elements.

Structures subject to hydrodynamic pressures induced by earthquakes shall be designed for rigid body and sloshing liquid forces and their own inertia force. The height of sloshing shall be determined and compared to the freeboard height of the structure.

Interior elements, such as baffles or roof supports, shall also be designed for the effects of unbalanced forces and sloshing.

1622.4.8 Buried structures. As used in this section, the term "buried structures" means subgrade structures such as tanks, tunnels and pipes. Buried structures that are designated as Seismic Use Group II or III, as determined in Section 1616.2, or are of such a size or length to warrant special seismic design as determined by the registered design professional, shall be identified in the geotechnical report. Buried structures shall be designed to resist seismic lateral forces determined from a substantiated analysis using standards approved by the building official. Flexible couplings shall be provided for buried structures where changes in the support system, configuration or soil condition occur.

1622.4.9 Inverted pendulums. As used in this section, the term "inverted pendulum" means structures that support an elevated lumped mass, excluding water tanks. Inverted pendulum structures shall be designed to resist seismic forces determined by using a substantiated analysis using standards approved by the building official.

SECTION 1623
SEISMICALLY ISOLATED STRUCTURES

1623.1 Design requirements. Every seismically isolated structure and every portion thereof shall be designed and constructed in accordance with the requirements of this section.

The lateral-force-resisting system and the isolation system shall be designed to resist the deformations and stresses produced by the effects of seismic ground motions as provided in this section.

The stability of the vertical-load-carrying elements of the isolation system shall be verified by analysis and text, as specified in this section, for lateral seismic displacement equal to the total maximum displacement.

For a seismically isolated structure, the determination of earthquake load, seismic use group, occupancy importance, I, seismic design category, response modification factor, R, system overstrength factor, Ω_0, deflection amplification factor, C_d, and limitations on height and use of structural systems, shall be the same as specified for nonisolated structures in Sections 1614 through 1616.1, unless modified by this section.

1623.1.1 Seismic use group. Portions of the structure, including the isolated structure system, shall be assigned a seismic use group in accordance with the requirements of Section 1616.2.

1623.1.2 Configuration. Each structure shall be designated as being regular or irregular on the basis of the structural configuration above the isolation system in accordance with the requirements of Section 1616.5.

1623.1.3 Selection of lateral response procedure. The selection of lateral response procedure shall be as specified below.

1623.1.3.1 Equivalent lateral response procedure. The equivalent lateral response procedure of Section 1623.2 is permitted to be used for design of a seismically isolated structure provided that:

1. The structure is located at a site with S_1, as determined in Section 1615.1, less than or equal to 0.60g.
2. The structure is located on soil classified as Site Class A, B, C or D, as determined in Section 1615.1.1.
3. The structure above the isolation interface is not more than four stories or 65 feet (19 812 mm) in height.
4. The effective period at maximum displacement of the isolated structure, T_M, as determined in Section 1623.2.3.1, is less than or equal to 3.0 seconds.
5. The effective period of the isolated structure at the design displacement, T_D, as determined in Section 1623.2.2.2, is greater than three times the elastic, fixed-base period of the structure above the isolation system as determined by Equations 16-39 and 16-40.
6. The structure above the isolation system is of regular configuration, as determined in Section 1616.5.
7. The isolation system meets the following criteria:
   - 7.1. The effective stiffness of the isolation system at the design displacement is greater than one-third of the effective stiffness at 20 percent of the design displacement;
   - 7.2. The isolation system is capable of producing a restoring force as specified in Section 1623.5.1.4.
   - 7.3. The isolation system has force-deflection properties that are independent of the rate of loading.
   - 7.4. The isolation system has force-deflection properties that are independent of vertical load and bilateral load.
   - 7.5. The isolation system does not limit maximum capable earthquake displacement to less than S_{M1}/S_{D1} times the total design displacement, where S_{M1} and S_{D1} are determined in accordance with Section 1615.

1623.1.3.2 Dynamic lateral response procedure. Use of the dynamic lateral response procedure of Section 1623.3 is permitted for the design of any structure. Use of the dynamic lateral response procedure of Section 1623.3 is required for the design of isolated structures not satisfying Section 1623.1.3.1.

1623.1.3.3 Response-spectrum analysis. Response-spectrum analysis, consistent with the requirements of Section 1623.3, is permitted to be used for design of a seismically isolated structure provided that:

1. The structure is located on a soil classified as Site Class A, B, C or D, as determined in Section 1615.1.
2. The isolation system meets the criteria of Item 7 of Section 1623.1.3.1.

1623.1.3.4 Time-history analysis. Time-history analysis consistent with the requirements of Section 1623.3 is permitted to be used for design of any seismically isolated structure and shall be used for design of seismically isolated structures not meeting the criteria of Section 1623.1.3.3.

1623.1.3.5 Site-specific design spectra. Site-specific ground-motion spectra of the design earthquake and the maximum considered earthquake developed in accordance with Section 1623.3.3.1 shall be used for design and analysis of seismically isolated structures if any one of the following conditions apply:

1. The structure is located on a soil classified as Site Class E or F, as determined in Section 1615.1.1.
2. The structure is located at a site with S_1 greater than 0.60g, as determined in Section 1615.1.1.

1623.2 Equivalent lateral response procedure. Where use of this procedure is permitted by Section 1623.1.3.1, the seismically isolated structure or portion thereof shall be designed and constructed to resist minimum earthquake displacements and forces as specified by this section and the applicable requirements of Section 1617.4.

1623.2.1 Deformation characteristics of the isolation system. Minimum lateral earthquake design displacements and forces on seismically isolated structures shall be based on the deformation characteristics of the isolation system.

The deformation characteristics of the isolation system shall explicitly include the effects of the wind-restraint system if such a system is used.

The deformation characteristics of the isolation system shall be based on properly substantiated tests performed in accordance with Section 1623.8.

1623.2.2 Minimum lateral displacements. The isolation system shall be designed and constructed to withstand the minimum lateral earthquake displacements specified in this section.

1623.2.2.1 Design displacement. The isolation system shall be designed and constructed to withstand minimum lateral earthquake displacements, D_D, that act in the direction of each of the main horizontal axes of the structure in accordance with the following:

$$D_D = \left(\frac{g}{4\pi^2}\right)\frac{S_{D1}T_D}{B_D} \qquad \textbf{(Equation 16-79)}$$

where:

B_D = Numerical coefficient related to the effective damping of the isolation system at the design displacement, β_D, as set forth in Table 1623.2.2.1.

g = Acceleration of gravity, in inches/sec^2 (mm/sec^2) if the units of the design displacement, D_D, are inches (mm).

S_{D1} = Design 5-percent damped spectral acceleration at 1-second period as determined in Section 1615.1.

T_D = Effective period, in seconds, of seismically isolated structure at the design displacement in the direction under consideration, as prescribed by Equation 16-80.

TABLE 1623.2.2.1
DAMPING COEFFICIENT, B_D OR B_M

EFFECTIVE DAMPING, β_D or β_M (percentage of critical)[a,b]	B_D OR B_M FACTOR
≤ 2%	0.8
5%	1.0
10%	1.2
20%	1.5
30%	1.7
40%	1.9
≥ 50%	2.0

a. The damping coefficient shall be based on the effective damping of the isolation system determined in accordance with the requirements of Section 1623.8.4.2.
b. The damping coefficient shall be based on linear interpolation for effective damping values other than those given.

1623.2.2.2 Effective period at design displacement. The effective period of the isolated structure, T_D, shall be determined using the deformational characteristics of the isolation system in accordance with the following equation:

$$T_D = 2\pi\sqrt{\frac{W_1}{k_{D\min}\, g}} \qquad \textbf{(Equation 16-80)}$$

where:

g = Acceleration of gravity in inches/sec^2 (mm/sec^2).

k_{Dmin} = Minimum effective stiffness, in kips/inch (kN/mm), of the isolation system at the design displacement in the horizontal direction under consideration as prescribed by Equation 16-93.

W_1 = Total seismic dead load weight, in kips (kN), of the structure above the isolation interface as defined in Sections 1617.4.1 and 1618.4.

1623.2.3 Maximum lateral displacement. The maximum displacement of the isolation system, D_M, in the most critical direction of horizontal response shall be calculated in accordance with the equation:

$$D_M = \frac{\left(\frac{g}{4\pi^2}\right)S_{M1}T_M}{B_M} \qquad \textbf{(Equation 16-81)}$$

where:

g = Acceleration of gravity in inches/sec^2 (mm/sec^2) if the units of the maximum displacement, D_M, are inches (mm).

S_{M1} = Maximum considered 5-percent damped spectral acceleration at 1-second period as determined in Section 1615.1.

T_M = Effective period, in seconds, of seismic-isolated structure at the maximum displacement in the direction under consideration as prescribed by Equation 16-82.

B_M = Numerical coefficient related to the effective damping of the isolation system at the maximum displacement, β_M, as set forth in Table 1623.2.2.1.

1623.2.3.1 Effective period at maximum displacement. The effective period of the isolated structure at maximum displacement, T_M, shall be determined using the deformational characteristics of the isolation system in accordance with the equation:

$$T_M = 2\pi\sqrt{\frac{W_I}{k_{Mmin}\,g}} \qquad \textbf{(Equation 16-82)}$$

where:

k_{Mmin} = Minimum effective stiffness, in kips/inch (kN/mm), of the isolation system at the maximum displacement in the horizontal direction under consideration as prescribed by Equation 16-95.

W_I = Total seismic dead load weight, in kips (kN), of the structure above the isolation interface as defined in Section 1617.4.1.

g = acceleration of gravity in inches/sec^2 (mm/sec^2) if the units of the maximum displacement, D_M, are inches (mm).

1623.2.4 Total lateral displacement. The total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, of elements of the isolation system shall include additional displacement due to actual and accidental torsion calculated considering the spatial distribution of the lateral stiffness of the isolation system and the most disadvantageous location of mass eccentricity.

The total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, of elements of an isolation system with uniform spatial distribution of lateral stiffness shall not be taken as less than that prescribed by the following equations:

$$D_{TD} = D_D\left[1 + y_i\left(\frac{12e}{b^2 + d^2}\right)\right] \qquad \textbf{(Equation 16-83)}$$

$$D_{TM} = D_M\left[1 + y_i\left(\frac{12e}{b^2 + d^2}\right)\right] \qquad \textbf{(Equation 16-84)}$$

where:

b = The shortest plan dimension of the structure, in feet (mm), measured perpendicular to d.

D_D = Design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Equation 16-79.

D_M = Maximum displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed in Equation 16-81.

d = The longest plan dimension of the structure, in feet (mm).

e = The actual eccentricity, in feet (mm), measured in plan between the center of mass of the structure above the isolation interface and the center of rigidity of the isolation system, plus accidental eccentricity, in feet (mm), taken as 5 percent of the longest plan dimension of the structure perpendicular to the direction of force under consideration.

y_i = The distance, in feet (mm), between the center of rigidity of the isolation system and the element of interest measured perpendicular to the direction of seismic loading under consideration.

The total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, are permitted to be taken as less than the value prescribed by Equations 16-83 and 16-84, respectively, but not less than 1.1 times D_D and D_M, as determined by Equations 16-79 and 16-81, respectively, provided the isolation system is shown by calculation to be configured to resist torsion accordingly.

1623.2.5 Minimum lateral forces. The minimum lateral forces used in the design of the isolation system and the structure above the isolation system shall be in accordance with Sections 1623.2.5.1 through 1623.2.5.3.

1623.2.5.1 Isolation system and structural elements at or below the isolation system. The isolation system, the foundation and structural elements below the isolation system shall be designed and constructed to withstand a minimum lateral seismic force, V_b, using the appropriate requirements for a nonisolated structure where:

$$V_b = k_{Dmax} D_D \qquad \textbf{(Equation 16-85)}$$

where:

D_D = Design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Equation 16-79.

k_{Dmax} = Maximum effective stiffness, in kips/inch (kN/mm), of the isolation system at the design displacement in the horizontal direction under consideration as prescribed by Equation 16-92.

V_b = The total lateral seismic design force or shear on elements of the isolation system or elements below the isolation system as prescribed by Equation 16-85.

V_b shall not be taken as less than the maximum force in the isolation system at any displacement up to and including the design displacement.

1623.2.5.2 Structural elements above the isolation system. The structure above the isolation system shall be designed and constructed to withstand a minimum shear force, V_s, using the appropriate provisions for a nonisolated structure where:

$$V_s = \frac{k_{Dmax} D_D}{R_I} \qquad \textbf{(Equation 16-86)}$$

where:

D_D = Design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Equation 16-79.

k_{Dmax} = Maximum effective stiffness, in kips/inch (kN/mm), of the isolation system at the design displacement in the horizontal direction under consideration as prescribed by Equation 16-92.

R_I = Numerical coefficient related to the type of lateral-force-resisting system above the isolation system.

The R_I factor shall be based on the type of lateral-force-resisting system used for the structure above the isolation system and shall be $^3/_8$ of the R-value given in Table 1617.6 with an upper bound value not to exceed 2.0 and a lower bound value not less than 1.0.

1623.2.5.3 Limits on V_s. The value of V_s shall be taken as not less than the following:

1. The lateral seismic force required by Section 1617.5 for a fixed-base structure of the same weight, W, and a period equal to the isolated period, T_D, as determined in Section 1623.2.2.2.
2. The base shear corresponding to the factored design wind load.
3. The lateral seismic force required to exceed the isolation system (the yield level of a softening system, the ultimate capacity of a sacrificial wind-restraint system, or the break-away friction level of a sliding system factored by 1.5).

1623.2.6 Vertical distribution of force. The total force, Vs, as determined in Section 1623.2.5, shall be distributed over the height of the structure above the isolation interface in accordance with the following formula:

$$F_x = \frac{V_s w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad \textbf{(Equation 16-87)}$$

where:

h_i = Height above the base to Level i.

h_x = Height above the base of Level x.

V_s = Total lateral seismic design force or shear on elements above the isolation system as prescribed by Equation 16-86.

w_x = Portion of W that is located at or assigned to Level x.

w_i = Portion of W that is located at or assigned to Level i.

At each level designated as x, the force, F_x, shall be applied over the area of the structure in accordance with the mass distribution at the level. Stresses in each structural element shall be calculated as the effect of force, F_x, applied at the appropriate levels above the base.

1623.2.7 Drift limits. The maximum interstory drift of the structure above the isolation system shall not exceed $0.015h_{sx}$. The drift shall be calculated by Equation 16-46 with the C_d factor of the isolated structure equal to the R_I factor defined in Section 1623.2.5.2.

1623.3 Dynamic lateral response procedure. Where a dynamic lateral response procedure is used for design of a seismically isolated structure, the seismically isolated structure or portion thereof shall be designed and constructed to resist earthquake displacements and forces as specified in this section and the applicable requirements of Section 1617.4.

1623.3.1 Isolation system and structural elements below the isolation system. The total design displacement of the isolation system shall be taken as not less than 90 percent of D_{TD} as specified by Section 1623.2.4.

The total maximum displacement of the isolation system shall be taken as not less than 80 percent of D_{TM} as prescribed by Equation 16-84.

The design lateral shear force on the isolation system and structural elements below the isolation system shall be taken as not less than 90 percent of V_b as prescribed by Equation 16-85.

The limits on displacements specified by this section shall be evaluated using values of D_{TD} and D_{TM} determined in accordance with Section 1623.2.4 except that D_D' is permitted to be used in lieu of D_{TD} and D_M' is permitted to be used in lieu of D_{TM}, where D_D' and D_M' are prescribed by the following formulas:

$$D_D' = \frac{D_D}{\sqrt{1+\left(\frac{T}{T_D}\right)^2}} \qquad \textbf{(Equation 16-88)}$$

$$D_M' = \frac{D_M}{\sqrt{1+\left(\frac{T}{T_M}\right)^2}} \qquad \textbf{(Equation 16-89)}$$

where:

D_D = Design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Equation 16-79.

D_M = Maximum displacement in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Equation 16-81.

T = Elastic, fixed-base period of the structure above the isolation system as determined by Section 1617.4.2.

T_D = Effective period, in seconds, of the seismically isolated structure at the design displacement in the direction under consideration as prescribed by Equation 16-80.

T_M = Effective period, in seconds, of the seismically isolated structure at the maximum displacement in the direction under consideration as prescribed by Equation 16-82.

1623.3.2 Structural elements above the isolation system. The design lateral shear force on the structure above the isolation system, if regular in configuration, as determined in Section 1616.5, shall be taken as not less than 80 percent of V_s, as prescribed by Equation 16-86 and the limits specified by Section 1623.2.5.3.

Exceptions:

1. The design lateral shear force on the structure above the isolation system, if regular in configuration, is permitted to be taken as less than 80 percent, but not less than 60 percent of V_s, provided time-history analysis, as specified in Section 1623.3.3.2, is used for design of the structure. The design lateral shear force on the structure above the isolation system, if irregular in configuration, as determined in Section 1616.5, shall be taken as not less than V_s, as prescribed by Equation 16-86 and the limits specified by Section 1623.2.5.3.
2. The design lateral shear force on the structure above the isolation system, if irregular in configuration, is permitted to be taken as less than 100 percent, but not less than 80 percent of V_s, provided time-history analysis, as specified in Section 1623.3.3.2, is used for design of the structure.

1623.3.3 Ground motion. Ground motions used with dynamic lateral response procedures shall be determined by Sections 1623.3.3.1 through 1623.3.3.2.

1623.3.3.1 Design spectra. Site-specific spectra are required for the design of structures located on soil classified as Site Class E or F, as determined in Section 1615.1.1, or located at a site with S_1 greater than 0.60g, as determined in Section 1615.1. Structures that do not require site-specific spectra and for which site-specific spectra have not been calculated shall be designed using the response spectrum shape specified in Section 1615.1.4.

Two design spectra shall be constructed: one for the design earthquake and one for the maximum considered earthquake.

The design earthquake response spectrum shall be taken as not less than the design earthquake response spectrum specified in Section 1615.1.

Exception: If a site-specific spectrum is calculated for the design earthquake, the design spectrum is permitted to be taken as less than 100 percent but not less than 80 percent of the design earthquake response spectrum specified in Section 1615.1.

The maximum considered earthquake design spectrum shall be taken as not less than 1.5 times the design earthquake response spectrum specified in Section 1615.1. This design spectrum shall be used to determine the total maximum displacement and overturning forces for design and testing of the isolation system.

Exception: If a site-specific spectrum is calculated for the maximum considered earthquake, the design spectrum is permitted to be taken as less than 100 percent but not less than 80 percent of 1.5 times the design earthquake response spectrum specified in Section 1615.1.

1623.3.3.2 Time histories. Where time-history analysis is used, pairs of appropriate horizontal ground motion time history components shall be selected and scaled from not less than three recorded events. Appropriate time histories shall be based on recorded events with magnitudes, fault distances and source mechanisms that are consistent with those that control the design earthquake (or maximum considered earthquake). Where three appropriate recorded ground motion time history pairs are not available, appropriate simulated ground motion time-history pairs are permitted to be used to make up the total number required. For each pair of horizontal ground-motion components, the square root sum of the squares of the 5 percent damped spectrum of the scaled, horizontal components shall be constructed. The motions shall be scaled such that the average value of the square-root-sum-of-the-squares spectra does not fall below 1.3 times the 5 percent damped spectrum of the design earthquake (or maximum considered earthquake) by more than 10 percent for periods from $0.5T_D$ seconds to $1.25T_M$ seconds, where T_D and T_M are determined in accordance with Sections 1623.2.2.2 and 1623.2.3.1, respectively.

1623.3.4 Mathematical model. The mathematical models of the isolated structure including the isolation system, the lateral-force-resisting system, and other structural elements shall conform to Section 1618.1.1 and to the requirements of Sections 1623.3.4.1 and 1623.3.4.2.

1623.3.4.1 Isolation system. The isolation system shall be modeled using deformational characteristics developed and verified by test in accordance with the requirements of Section 1623.2.1. The isolation system shall be modeled with sufficient detail to:

1. Account for the spatial distribution of isolator units.
2. Calculate translation, in both horizontal directions, and torsion of the structure above the isolation interface considering the most disadvantageous location of mass eccentricity.
3. Assess overturning/uplift forces on individual isolator units.
4. Account for the effects of vertical load, bilateral load, and/or the rate of loading if the force deflection properties of the isolation system are dependent on one or more of these attributes.

1623.3.4.2 Isolated structure. The isolated structure shall be modeled in accordance with Sections 1623.3.4.2.1 and 1623.3.4.2.2.

1623.3.4.2.1 Displacement. The maximum displacement of each floor and the total design displacement and total maximum displacement across the isolation system shall be calculated using a model of the isolated structure that incorporates the force-deflection characteristics of nonlinear elements of the isolation system and the lateral-force-resisting system. Lateral-force-resisting systems with nonlinear elements include, but are not limited to, irregular structural systems designed for a lateral force less than 100 percent and regular structural systems designed for a lateral force less than 80 percent of V_s, as prescribed by Equation 16-86 and the limits specified by Section 1623.2.5.3, where regularity is determined in accordance with Section 1616.5.

1623.3.4.2.2 Forces and displacements in elements of the lateral-force-resisting system. Design forces and displacements in elements of the lateral-force-resisting system are permitted to be calculated using a linear elastic model of the isolated structure provided that:

1. Stiffness properties assumed for nonlinear isolation-system components are based on the maximum effective stiffness of the isolation system.
2. No elements of the lateral-force-resisting system of the structure above the isolation system are nonlinear.

1623.3.5 Response-spectrum and time-history analysis procedures. Response-spectrum and time-history analyses shall be performed in accordance with Section 1617.4 and the requirements of this section.

1623.3.5.1 Input earthquake. The design earthquake shall be used to calculate the total design displacement of the isolation system and the lateral forces and displacements of the isolated structure. The maximum considered earthquake shall be used to calculate the total maximum displacement of the isolation system.

1623.3.5.2 Response-spectrum analysis. Response-spectrum analysis shall be performed using a modal damping value for the fundamental mode in the direction of interest not greater than the effective damping of the isolation system or 30 percent of critical, whichever is less. Modal damping values for higher modes shall be selected consistent with those appropriate for response spectrum analysis of the structure above the isolation system as a fixed base.

Response-spectrum analysis used to determine the total design displacement and the total maximum displacement shall include simultaneous excitation of the model by 100 percent of the most critical direction of ground motion and 30 percent of the ground motion on the orthogonal axis. The maximum displacement of the isolation system shall be calculated as the vectorial sum of the two orthogonal displacements.

The design shear at any story, V_x, shall not be less than the story shear obtained using Equation 16-87 and a value of V_s taken as that equal to the base shear obtained from the response-spectrum analysis in the direction of interest.

1623.3.5.3 Time-history analysis. Time-history analysis shall be performed with at least three appropriate pairs of horizontal time-history components as defined in Section 1623.3.3.2.

Each pair of time histories shall be applied simultaneously to the model considering the most disadvantageous location of mass eccentricity. The maximum displacement of the isolation system shall be calculated from the vectorial sum of the two orthogonal components at each time step.

The parameter of interest shall be calculated for each time-history analysis. If three time-history analyses are performed, the maximum response of the parameter of interest shall be used for design. If seven or more time-history analyses are performed, the average value of the response parameter of interest shall be used for design.

1623.3.6 Design lateral force. The design lateral force shall be determined by Sections 1623.3.6.1 through 1623.3.6.4.

1623.3.6.1 Isolation system. The isolation system, foundation and structural elements below the isolation system shall be designed using the appropriate requirements for a nonisolated structure and the forces obtained from the dynamic lateral response procedure of this section without reduction.

1623.3.6.2 Structural elements above the isolation system. Structural elements above the isolation system shall be designed using the appropriate requirements for a nonisolated structure and the forces obtained from the dynamic lateral response procedure of this section divided by a factor of R_I. The R_I factor shall be determined from Section 1623.2.5.2 based on the type of lateral-force-resisting system used for the structure above the isolation system.

1623.3.6.3 Scaling of results. When the factored lateral shear force on structural elements, determined using either response spectrum or time-history analysis, is less than the minimum level prescribed by Sections 1623.3.1 and 1623.3.2, response parameters, including member forces and moments, shall be adjusted proportionally upward.

1623.3.6.4 Drift limits. Maximum interstory drift corresponding to the design lateral force including displacement due to vertical deformation of the isolation system shall not exceed the following limits:

1. The maximum interstory drift of the structure above the isolation system calculated by response spectrum analysis shall not exceed $0.015h_{sx}$, where h_{sx} is the story height below Level x.
2. The maximum interstory drift of the structure above the isolation system calculated by time-history analysis considering the force-deflection characteristics of nonlinear elements of the lateral-force-resisting system shall not exceed $0.020h_{sx}$.

Drift shall be calculated using Equation 16-46 with the C_d factor of the isolated structure equal to the R_I factor defined in Section 1623.2.5.2.

The secondary effects of the maximum considered earthquake lateral displacement, Δ, of the structure above the isolation system combined with gravity forces shall be investigated if the interstory drift ratio exceeds $0.010/R_I$.

1623.4 Lateral load on elements of structures and nonstructural components supported by buildings. Parts or portions of an isolated structure, permanent nonstructural components and the attachments to them, and the attachments for permanent equipment supported by a structure shall be designed to resist seismic forces and displacements as prescribed by this section and the applicable requirements of Section 1621.

1623.4.1 Forces and displacements. Components shall be designed to resist forces and displacements in accordance with Sections 1623.4.1.1 through 1623.4.1.3.

1623.4.1.1 Components at or above the isolation interface. Elements of seismically isolated structures and nonstructural components, or portions thereof, that are at or above the isolation interface shall be designed to resist a total lateral seismic force equal to the maximum dynamic response of the element or component under consideration.

Exception: Elements of seismically isolated structures and nonstructural components or portions thereof are permitted to be designed to resist the earthquake effect, *E*, as prescribed by Equations 16-28 and 16-29.

1623.4.1.2 Components crossing the isolation interface. Elements of seismically isolated structures and nonstructural components, or portions thereof, that cross the isolation interface shall be designed to withstand the total maximum displacement.

1623.4.1.3 Components below the isolation interface. Elements of seismically isolated structures and nonstructural components, or portions thereof, that are below the isolation interface shall be designed and constructed in accordance with the requirements of Section 1616.1.

1623.5 Detailed system requirements. The isolation system and the structural system shall comply with the material requirements of this code. In addition, the isolation system shall comply with the detailed system requirements of this section and the structural system shall comply with the detailed system requirements of this section and the applicable portions of Section 1616.1.

1623.5.1 Isolation system. The isolation system shall meet the following requirements.

1623.5.1.1 Environmental conditions. In addition to the requirements for vertical and lateral loads induced by wind and earthquake, the isolation system shall be designed with consideration given to other environmental conditions including aging effects, creep, fatigue, operating temperature, and exposure to moisture or damaging substances.

1623.5.1.2 Wind forces. Isolated structures shall resist design wind loads at all levels above the isolation interface. At the isolation interface, a wind-restraint system shall be provided to limit lateral displacement in the isolation system to a value equal to that required between floors of the structure above the isolation interface.

1623.5.1.3 Fire resistance. Fire-resistance rating for the isolation system shall be consistent with the requirements of columns, walls or other such elements of the structure.

1623.5.1.4 Lateral-restoring force. The isolation system shall be configured to produce a restoring force such that the lateral force at the total design displacement is at least $0.025W$ greater than the lateral force at 50 percent of the total design displacement, where W is as defined in Section 1617.5.1.

Exception: The isolation system need not be configured to produce a restoring force, as required above, provided the isolation system is capable of remaining stable under full vertical load and accommodating a total maximum displacement equal to the greater of either 3.0 times the total design displacement or $36S_{MI}$ inches (915 S_{MI} mm), where S_{MI} is determined in accordance with Section 1615.1.

1623.5.1.5 Displacement restraint. The isolation system is permitted to be configured to include a displacement restraint that limits lateral displacement due to the maximum considered earthquake to less than S_{MI}/S_{DI} (determined in accordance with Section 1615.1) times the total design displacement provided that the seismically isolated structure is designed in accordance with the following criteria when more stringent than the requirements of Section 1623.1:

1. Maximum considered earthquake response is calculated in accordance with the dynamic analysis requirements of Section 1623.3 explicitly considering the nonlinear characteristics of the isolation system and the structure above the isolation system.
2. The ultimate capacity of the isolation system and structural elements below the isolation system shall exceed the strength and displacement demands of the maximum considered earthquake.
3. The structure above the isolation system is checked for stability and ductility demand of the maximum considered earthquake.
4. The displacement restraint does not become effective at a displacement less than 0.75 times the total design displacement unless it is demonstrated by analysis that earlier engagement does not result in unsatisfactory performance.

1623.5.1.6 Vertical-load stability. Each element of the isolation system shall be designed to be stable under the maximum vertical load $(1.2D + 1.0L + |E|)$ and the minimum vertical load ($0.8D - |E|$) at a horizontal displacement equal to the total maximum displacement. The dead load, D, and the live load, L, are specified in Sections 1606 and 1607, respectively. The seismic load, E, is given by Equations 16-28 and 16-29 where S_{DS} in these equations is replaced by S_{MS} (both as determined in Section 1615.1) and the vertical load due to earthquake, Q_E, is based on peak response due to the maximum considered earthquake.

1623.5.1.7 Overturning. The factor of safety against global structural overturning at the isolation interface shall not be less than 1.0 for required load combinations. All gravity and seismic loading conditions shall be investigated. Seismic forces for overturning calculations shall be based on the maximum considered earthquake and W, as defined in Section 1617.5.1, shall be used for the vertical-restoring force.

Local uplift of individual elements is permitted provided the resulting deflections do not cause overstress or instability of the isolator units or other structural elements.

1623.5.1.8 Inspection and replacement. Inspection and replacement requirements for the isolation system shall be in accordance with the following four items:

1. Access for inspection and replacement of all components of the isolation system shall be provided.
2. A registered design professional shall complete a final series of inspections or observations of structure separation areas and components that cross the isolation interface prior to the issuance of the certificate of occupancy for the seismically isolated structure. Such inspections or observations shall indicate that the conditions allow free and unhindered displacement of the structure to maximum design levels and that components that cross the isolation interface as installed are able to accommodate the stipulated displacements.
3. Seismically isolated structures shall have a periodic monitoring, inspection and maintenance program for the isolation system established by the registered design professional responsible for the design of the system.
4. Remodeling, repair or retrofitting at the isolation system interface, including that of components that cross the isolation interface, shall be performed under the direction of a registered design professional.

1623.5.1.9 Quality control. A quality control testing program for isolator units shall be established by the registered design professional responsible for the structural design.

1623.5.2 Structural system. The structural system shall meet the requirements of Sections 1623.5.2.1 through 1623.5.2.3.

1623.5.2.1 Horizontal distribution of force. A horizontal diaphragm or other structural elements shall provide continuity above the isolation interface and shall have adequate strength and ductility to transmit forces (due to nonuniform ground motion) from one part of the structure to another.

1623.5.2.2 Building separations. Minimum separations between the isolated structure and surrounding retaining walls or other fixed obstructions shall not be less than the total maximum displacement.

1623.5.2.3 Nonbuilding structures. These shall be designed and constructed in accordance with the requirements of Section 1622 using design displacements and forces calculated in accordance with Section 1623.2 or 1623.3.

1623.6 Foundations. Foundations shall be designed and constructed in accordance with the requirements of Chapter 18 using design forces calculated in accordance with Section 1623.2 or 1623.3.

1623.7 Design and construction review. A design review of the isolation system and related test programs shall be performed by an independent team of registered design professionals in the appropriate disciplines and others experienced in seismic analysis methods and the theory and application of seismic isolation. Isolation system design review shall include, but not be limited to, the following:

1. Review of site-specific seismic criteria including the development of site-specific spectra and ground motion time histories and other design criteria developed specifically for the project.
2. Review of the preliminary design including the determination of the total design displacement of the isolation system design displacement and the lateral force design level.
3. Overview and observation of prototype testing (Section 1623.8).
4. Review of the final design of the entire structural system and supporting analyses.
5. Review of the isolation system quality control testing program (Section 1623.5.1.9).

1623.8 Required tests of the isolation system. The deformation characteristics and damping values of the isolation system used in the design and analysis of seismically isolated structures shall be based on tests of a selected sample of the components prior to construction as described in this section.

The isolation system components to be tested shall include the wind-restraint system if such a system is used in the design.

The tests specified in this section are for establishing and validating the design properties of the isolation system and shall not be considered as satisfying the manufacturing quality control tests of Section 1623.5.1.9.

1623.8.1 Prototype tests. Prototype tests shall be performed separately on two full-size specimens (or sets of specimens, as appropriate) of each predominant type and size of isolator unit of the isolation system. The test specimens shall include the wind-restraint system as well as individual isolator units if such systems are used in the design. Specimens tested shall not be used for construction.

1623.8.1.1 Record. For each cycle of tests, the force-deflection behavior of the test specimen shall be recorded.

1623.8.1.2 Sequence and cycles. The following sequence of tests shall be performed for the prescribed number of cycles at a vertical load equal to the average dead load plus one-half the effects due to live load on isolator units of a common type and size:

1. Twenty fully reversed cycles of loading at a lateral force corresponding to the wind design force.
2. Three fully reversed cycles of loading at each of the following increments of the total design displacement: $0.25D_D$, $0.5D_D$, $1.0D_D$, and $1.0D_M$, where D_D and D_M are as determined in Sections 1623.2.2.1 and 1623.2.3, respectively, or Section 1623.3, as appropriate.
3. Three fully reversed cycles of loading at the total maximum displacement, $1.0D_{TM}$, where D_{TM} is as determined in Section 1623.2.4 or 1623.3.
4. $15S_{D1}B_D/S_{DS}$, but not less than 10, fully reversed cycles of loading at one total design displacement, $1.0D_{TD}$, where S_{D1} and S_{DS} are as determined in Section 1615.1, B_D is as determined in Table 1623.2.2.1, and D_{TD} is as determined in Section 1623.2.4 or 1623.3, as appropriate.

If an isolator unit is also a vertical-load-carrying element, then Item 2 of the sequence of cyclic tests specified above shall be performed for two additional vertical load cases:

$$1.2D + 0.5L + |E| \qquad \textbf{(Formula 16-21)}$$

$$0.8D - |E| \qquad \textbf{(Formula 16-22)}$$

where dead load, D, and live load, L, are specified in Sections 1606 and 1607, respectively. The seismic load, E, is given by Equations 16-28 and 16-29 and the load increment due to earthquake overturning, Q_E, shall be equal to or greater than the peak earthquake vertical-force response corresponding to the test displacement being evaluated. In these tests, the combined vertical load shall be taken as the typical or average downward force on isolator units of a common type and size.

1623.8.1.3 Units dependent on loading rates. If the force-deflection properties of the isolator units are dependent on the rate of loading, then each set of tests specified in Section 1623.8.1.2 shall be performed dynamically at a frequency equal to the inverse of the effective period, T_D, which is determined in Section 1623.2.2.2 or 1623.3, as appropriate.

If reduced-scale prototype specimens are used to quantify rate-dependent properties of isolators, the reduced-scale prototype specimens shall be of the same type and material and be manufactured with the same processes and quality as full-scale prototypes and shall be tested at a frequency that represents full-scale prototype loading rates.

The force-deflection properties of an isolator unit shall be considered to be dependent on the rate of loading if there is greater than a ± 15-percent difference in the effective stiffness and the effective damping at the design displacement when tested at a frequency equal to the inverse of the effective period, T_D, of the isolated structure and when tested at any frequency in the range of 0.1 to 2.0 times the inverse of the effective period, T_D, of the isolated structure.

1623.8.1.4 Units dependent on bilateral load. If the force-deflection properties of the isolator units are dependent on bilateral load, the tests specified in Sections 1623.8.1.2 and 1623.8.1.3 shall be augmented to include bilateral load at the following increments of the total design displacement: 0.25 and 1.0; 0.50 and 1.0; 0.75 and 1.0; and 1.0 and 1.0.

If reduced-scale prototype specimens are used to quantify bilateral-load-dependent properties, then such scale specimens shall be of the same type and material and manufactured with the same processes and quality as full-scale prototypes.

The force-deflection properties of an isolator unit shall be considered to be dependent on bilateral load if the bilateral and unilateral force-deflection properties have greater than a ± 15-percent difference in effective stiffness at the design displacement.

1623.8.1.5 Maximum and minimum vertical load. Isolator units that carry vertical load shall be statically tested for the maximum and minimum vertical load at the total maximum displacement. In these tests, the combined vertical load, $1.2D + 1.0L + |E|_{max}$, shall be taken as the maximum vertical force, and the combined vertical load, $0.8D - |E|_{min}$, shall be taken as the minimum vertical force, on any one isolator of a common type and size. The dead load, D, and live load, L, are specified in Section 1605. The seismic load, E, is given by Equations 16-28 and 16-29, where S_{DS} in these formulas is replaced by S_{MS} (both as determined in Section 1615.1), and the load increment caused by earthquake overturning, Q_E, is equal to or greater than the peak earthquake vertical-force response corresponding to the maximum considered earthquake.

1623.8.1.6 Sacrificial wind-restraint systems. If a sacrificial wind-restraint system is to be utilized, the ultimate capacity shall be established by test.

1623.8.1.7 Testing similar units. The prototype tests are not required if an isolator unit is of similar dimensional characteristics and of the same type and material as a prototype isolator unit that has been previously tested using the specified sequence of tests.

1623.8.2 Determination of force-deflection characteristics. The force-deflection characteristics of the isolation system shall be based on the cyclic load tests of isolator prototypes specified in Section 1623.8.1. As required, the effective stiffness of an isolator unit, k_{eff}, shall be calculated for each cycle of loading by the equation:

$$k_{eff} = \frac{|F^+| + |F^-|}{|\Delta^+| + |\Delta^-|} \quad \textbf{(Equation 16-90)}$$

where F^+ and F^- are the positive and negative forces at Δ^+ and Δ^-, respectively.

As required, the effective damping, β_{eff}, of an isolator unit shall be calculated for each cycle of loading by the equation:

$$\beta_{eff} = \frac{2}{\pi}\left[\frac{E_{loop}}{k_{eff}\left(|\Delta^+| + |\Delta^-|\right)^2}\right] \quad \textbf{(Equation 16-91)}$$

where the energy dissipated per cycle of loading, E_{loop}, and the effectiveness stiffness, k_{eff}, shall be based on peak test displacements of Δ^+ and Δ^-.

1623.8.3 Test specimen adequacy. The performance of the test specimens shall be assessed as adequate if the following conditions are satisfied:

1. The force-deflection plots of tests specified in Section 1623.8.1 have a positive incremental force-carrying capacity.
2. For each increment of test displacement specified in Item 2 of Section 1623.8.1.2 and for each vertical load case specified in Section 1623.8.1.2, there is no greater than a ± 15-percent difference between the effective stiffness at each of the three cycles of test and the average value of effective stiffness for each test specimen.
   2.1. For each increment of test displacement specified in Item 2 of Section 1623.8.1.2 and for each vertical load case specified in Section 1623.8.1.2, there is no greater than a 15-percent difference in the average value of effective stiffness of the two test specimens of a common type and size of the isolator unit over the required three cycles of test.
3. For each specimen there is no greater than a ± 20 percent change in the initial effective stiffness of each test specimen over the $15S_{D1}\beta_D/S_{DS}$ (see Item 4 of Section 1623.8.1.2), but not less than 10, cycles of test specified in Item 3 of Section 1623.8.1.2.
4. For each specimen there is no greater than a 20 percent decrease in the initial effective damping over for the $15S_{D1}\beta_D/S_{DS}$, but not less than 10, cycles of test specified in Item 3 of Section 1623.8.1.2.
5. All specimens of vertical-load-carrying elements of the isolation system remain stable up to the total maximum displacement for static load as prescribed in Section 1623.8.1.5.

1623.8.4 Design properties of the isolation system. The isolation system shall have the properties shown in Sections 1623.8.4.1 and 1623.8.4.2.

1623.8.4.1 Maximum and minimum effective stiffness. At the design displacement, the maximum and minimum effectiveness stiffness of the isolated system, k_{Dmax} and k_{Dmin}, shall be based on the cyclic tests of Section 1623.8.1.2 and calculated by the equations:

$$k_{Dmax} = \frac{\sum|F_D^+|_{max} + \sum|F_D^-|_{max}}{2D_D} \quad \textbf{(Equation 16-92)}$$

$$k_{Dmin} = \frac{\sum|F_D^+|_{min} + \sum|F_D^-|_{min}}{2D_D} \quad \textbf{(Equation 16-93)}$$

At the maximum displacement, the maximum and minimum effective stiffness of the isolation system, k_{Mmax} and k_{Mmin}, shall be based on the cyclic tests of Item 2 of Section 1623.8.2 and calculated by the equations:

$$k_{Mmax} = \frac{\sum|F_M^+|_{max} + \sum|F_M^-|_{max}}{2D_M} \quad \textbf{(Equation 16-94)}$$

$$k_{Kmin} = \frac{\sum|F_M^+|_{min} + \sum|F_M^-|_{min}}{2D_M} \quad \textbf{(Equation 16-95)}$$

The maximum effective stiffness of the isolation system, k_{Dmax} (or k_{Mmax}), shall be based on forces from the cycle of prototype testing at a test displacement equal to D_D (or D_M) that produces the largest value of effective stiffness. Minimum effective stiffness of the isolation system, k_{Dmin} (or k_{Mmin}), shall be based on forces from the cycle of prototype testing at a test displacement equal to D_D (or D_M) that produces the smallest value of effective stiffness.

For isolator units that are found by the tests of Sections 1623.8.1.2 and 1623.8.1.3 to have force-deflection characteristics that vary with vertical load, rate of loading or bilateral load, respectively, the values of k_{Dmax} and k_{Mmax} shall be increased and the values of k_{Dmin} and k_{Mmin} shall be decreased, as necessary, to bound the effects of measured variation in effective stiffness.

1623.8.4.2 Effective damping. At the design displacement, the effective damping of the isolation system, β_D, shall be based on the cyclic tests of Item 2 of Section 1623.8.1.2 and calculated by the equation:

$$\beta_D = \frac{1}{2\pi}\left[\frac{\sum E_D}{k_{Dmax} D_D^2}\right] \quad \textbf{(Equation 16-96)}$$

The total energy dissipated per cycle of design displacement response, ΣE_D, shall be taken as the sum of the energy dissipated per cycle in all isolator units measured at a test displacement equal to D_D. The total energy dissipated per cycle of design displacement response, ΣE_D, shall be based on forces and deflections from the cycle of prototype testing at test displacement D_D that produces the smallest value of effective damping.

At the maximum displacement, the effective damping of the isolation system, β_M, shall be based on the cyclic tests of Item 2 of Section 1623.8.1.2 and calculated by the equation:

$$\beta_M = \frac{1}{2\pi}\left[\frac{\sum E_M}{k_{Mmax} D_M^2}\right] \qquad \textbf{(Equation 16-97)}$$

In Equation 16-97, the total energy dissipated per cycle of design displacement response, ΣE_M, shall be taken as the sum of the energy dissipated per cycle in all isolator units measured at a test displacement equal to D_M. The total energy dissipated per cycle of maximum displacement response, ΣE_M, shall be based on forces and deflections from the cycle of prototype testing at test displacement D_M that produces the smallest value of effective damping.

CHAPTER 17

STRUCTURAL TESTS AND SPECIAL INSPECTIONS

SECTIONS 1701-1714
Deleted

SECTION 1715
MATERIAL AND TEST STANDARDS

1715.1 Test standards for joist hangers and connectors.

1715.1.1 Test standards for joist hangers. The vertical load-bearing capacity, torsional moment capacity, and deflection characteristics of joist hangers shall be determined in accordance with ASTM D 1761, using lumber having a specific gravity of 0.49 or greater, but not greater than 0.55, as determined in accordance with AFPA NDS for the joist and hangers.

1715.1.2 Vertical load capacity for joist hangers. The vertical load capacity for the joist hanger shall be determined by testing three joist hanger assemblies as specified in ASTM D 1761. If the ultimate vertical load for any one of the tests varies more than 20 percent from the average ultimate vertical load, at least three additional tests shall be conducted. The allowable vertical load for a normal duration of loading of the joist hanger shall be the lowest value determined from the following:

1. The lowest ultimate vertical load from any test divided by 3 (where three tests are conducted and each ultimate vertical load does not vary more than 20 percent from the average ultimate vertical load).
2. The average ultimate vertical load for all tests divided by 6 (where six or more tests are conducted).
3. The vertical load at which the vertical movement of the joist with respect to the header is 0.125 inch (3.2 mm) in any test.
4. The allowable design load for nails or other fasteners utilized to secure the joist hanger to the wood members.
5. The allowable design load for the wood members forming the connection.

1715.1.3 Torsional moment capacity for joist hangers. The torsional moment capacity for the joist hanger shall be determined by testing at least three joist hanger assemblies as specified in ASTM D 1761. The allowable torsional moment for normal duration of loading of the joist hanger shall be the average torsional moment at which the lateral movement of the top or bottom of the joist with respect to the original position of the joist is 0.125 inch (3.2 mm).

1715.1.4 Design value modifications for joist hangers. Allowable design values for joist hangers that are determined by Item 4 or 5 in Section 1715.1.2 shall be permitted to be modified by the appropriate duration of loading factors as specified in AFPA NDS. Allowable design values determined by Item 1, 2 or 3 in Sections 1715.1.2 and 2305.1 shall not be modified by duration of loading factors.

1715.2 Concrete and clay roof tiles.

1715.2.1 Overturning resistance. Concrete and clay roof tiles shall be tested to determine their resistance to overturning due to wind in accordance with SBCCI SSTD 11 and Chapter 15.

1715.2.2 Wind tunnel testing. When roof tiles do not satisfy the limitations in Chapter 16 for rigid tile, a wind tunnel test shall be used to determine the wind characteristic of the concrete or clay tile roof covering in accordance with SBCCI SSTD 11 and Chapter 15.

CHAPTER 18

SOILS AND FOUNDATIONS

SECTION 1801 GENERAL

1801.1 Scope. The provisions of this chapter shall apply to building and foundation systems in those areas not subject to scour or water pressure by wind and wave action. Buildings and foundations subject to such scour or water pressure loads shall be designed in accordance with Chapter 16.

1801.2 Design. Allowable bearing pressures, allowable stresses and design formulas provided in this chapter shall be used with the allowable stress design load combinations specified in Section 1605.3. The quality and design of materials used structurally in excavations, footings and foundations shall conform to the requirements specified in Chapters 16, 19, 21, 22 and 23 of this code. Excavations and fills shall also comply with Chapter 33.

1801.2.1 Foundation design for seismic overturning. Where the foundation is proportioned using the strength design load combinations of Section 1605.2, the seismic overturning moment need not exceed 75 percent of the value computed from Section 1617.4.5 for the equivalent lateral force method, or Section 1618 for the modal analysis method.

SECTION 1802 FOUNDATION AND SOILS INVESTIGATIONS

1802.1 [Comm 62.1802 (1)] General. Foundation and soils investigations shall be conducted in conformance with Sections 1802.2 through 1802.6.

1802.2 [Comm 62.1802 (2)] Where required. The owner or applicant shall make a foundation and soils investigation available to the building official, upon request, where required in Sections 1802.2.1 through 1802.2.7.

Exception: The building official need not require a foundation or soils investigation where satisfactory data from adjacent areas is available that demonstrates an investigation is not necessary for any of the conditions in Sections 1802.2.1 through 1802.2.6.

1802.2.1 [Comm 62.1802 (3)] Questionable soil. Where the safe-sustaining power of the soil is in doubt, or where a load-bearing value superior to that specified in this code is claimed, an investigation complying with the provisions of IBC Sections 1802.4 through 1802.6 shall be made.

1802.2.2 [Comm 62.1802 (4)] Expansive soils. In areas likely to have expansive soil, soil tests shall be conducted to determine where such soils do exist.

1802.2.3 Groundwater table. A subsurface soil investigation shall be performed to determine whether the existing groundwater table is above or within 5 feet (1524 mm) below the elevation of the lowest floor level where such floor is located below the finished ground level adjacent to the foundation.

Exception: A subsurface soil investigation shall not be required where waterproofing is provided in accordance with Section 1806.

1802.2.4 Pile and pier foundations. Pile and pier foundations shall be designed and installed on the basis of a foundation investigation and report as specified in Sections 1802.4 through 1802.6 and Section 1807.2.1.

1802.2.5 Rock strata. Where subsurface explorations at the project site indicate variations or doubtful characteristics in the structure of the rock upon which foundations are to be constructed, a sufficient number of borings shall be made to a depth of not less than 10 feet (3048 mm) below the level of the foundations to provide assurance of the soundness of the foundation bed and its load-bearing capacity.

1802.2.6 Seismic Design Category C. Where a structure is determined to be in Seismic Design Category C in accordance with Section 1616, an investigation shall be conducted, and shall include an evaluation of the following potential hazards resulting from earthquake motions: slope instability, liquefaction, and surface rupture due to faulting or lateral spreading.

1802.2.7 Seismic Design Category D, E or F. Where the structure is determined to be in Seismic Design Category D, E or F, in accordance with Section 1616, the soils investigation requirements for Seismic Design Category C, given in Section 1802.2.6, shall be met, in addition to the following. The investigation shall include:

1. A determination of lateral pressures on basement and retaining walls due to earthquake motions.
2. An assessment of potential consequences of any liquefaction and soil strength loss, including estimation of differential settlement, lateral movement or reduction in foundation soil-bearing capacity, and shall discuss mitigation measures. Such measures shall be given consideration in the design of the structure and can include, but are not limited to, ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements, or any combination of these measures. The potential for liquefaction and soil strength loss shall be evaluated for site peak ground acceleration magnitudes and source characteristics consistent with the design earthquake ground motions. Peak ground acceleration shall be determined from a site-specific study taking into ac-

count soil amplification effects, as specified in Section 1615.2.

Exception: A site-specific study need not be performed provided that peak ground acceleration equal to $S_{DS}/2.5$ is used, where S_{DS} is determined in accordance with Section 1615.2.1.

1802.3 Soil classification. Where required, soils shall be classified in accordance with Section 1802.3.1 or 1802.3.2.

1802.3.1 General. For the purposes of this chapter, the definition and classification of soil materials for use in Table 1804.2 shall be in accordance with ASTM D 2487.

1802.3.2 Expansive soils. Soils meeting all four of the following provisions shall be considered expansive, except that tests to show compliance with Items 1, 2 and 3 shall not be required if the test prescribed in Item 4 is conducted:

1. Plasticity Index (PI) of 15 or greater, determined in accordance with ASTM D 4318.
2. More than 10 percent of the soil particles pass a No. 200 sieve (75 μm), determined in accordance with ASTM D 422.
3. More than 10 percent of the soil particles are less than 5 micrometers in size, determined in accordance with ASTM D 422.
4. Expansion Index greater than 20, determined in accordance with UBC Standard 18-2 or SBCCI SSTD 7.

1802.4 Investigation. Soil classification shall be based on observation and any necessary tests of the materials disclosed by borings, test pits or other subsurface exploration made in appropriate locations. Additional studies shall be made as necessary to evaluate slope stability, soil strength, position and adequacy of load-bearing soils, the effect of moisture variation on soil-bearing capacity, compressibility, liquefaction and expansiveness.

1802.4.1 Exploratory boring. The scope of the soil investigation including the number and types of borings or soundings, the equipment used to drill and sample, the in-situ testing equipment, and the laboratory testing program shall be determined by a registered design professional.

1802.5 Soil boring and sampling. The soil boring and sampling procedure and apparatus shall be in accordance with generally accepted engineering practice. The registered design professional shall have a fully qualified representative on the site during all boring and sampling operations.

1802.6 Reports. The soil classification and design load-bearing capacity shall be shown on the construction document. Where required by the building official, a written report of the investigation shall be submitted that shall include, but need not be limited to, the following information:

1. A plot showing the location of test borings and/or excavations.
2. A complete record of the soil samples.
3. A record of the soil profile.
4. Elevation of the water table, if encountered.
5. Recommendations for foundation type and design criteria, including but not limited to: bearing capacity of natural or compacted soil; provisions to mitigate the effects of expansive soils; mitigation of the effects of liquefaction, differential settlement, and varying soil strength; and the effects of adjacent loads.
6. Expected total and differential settlement.
7. Pile and pier foundation information in accordance with Section 1807.2.1.
8. Special design and construction provisions for footings or foundations founded on expansive soils, as necessary.
9. Compacted fill material properties and testing in accordance with Section 1803.4.

SECTION 1803
EXCAVATION, GRADING AND FILL

1803.1 Excavations near footings or foundations. Excavations for any purpose shall not remove lateral support from any footing or foundation without first underpinning or protecting the footing or foundation against settlement or lateral translation.

1803.2 Placement of backfill. The excavation outside the foundation shall be backfilled with soil that is free of organic material, construction debris, cobbles and boulders. The backfill shall be placed in lifts and compacted in a manner that does not damage the foundation or the waterproofing or dampproofing material.

1803.3 Site grading. The ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5-percent slope) for a minimum distance of 10 feet (3048 mm) measured perpendicular to the face of the wall or an approved alternate method of diverting water away from the foundation shall be used.

Exception: Where climatic or soil conditions warrant, the slope of the ground away from the building foundation is permitted to be reduced to not less than one unit vertical in 48 units horizontal (2-percent slope).

The procedure used to establish the final ground level adjacent to the foundation shall account for additional settlement of the backfill.

1803.4 Compacted fill material. Where footings will bear on compacted fill material, the compacted fill shall comply with the provisions of an approved report. The report shall contain the following:

1. Specifications for the preparation of the site prior to placement of compacted fill material.
2. Specifications for material to be used as compacted fill.
3. Test method to be used to determine the maximum dry density and optimum moisture content of the material to be used as compacted fill.
4. Maximum allowable thickness of each lift of compacted fill material.

5. Field test method for determining the in-place dry density of the compacted fill.
6. Minimum acceptable in-place dry density expressed as a percentage of the maximum dry density determined in accordance with Item 3.
7. Number and frequency of field tests required to determine compliance with Item 6.

Exception: Compacted fill material less than 12 inches (305 mm) in depth need not comply with an approved report, provided it has been compacted to a minimum of 90 percent Modified Proctor in accordance with ASTM D 1557. The compaction shall be verified by a qualified inspector approved by the building official.

SECTION 1804
ALLOWABLE LOAD-BEARING VALUES OF SOILS

1804.1 Design. The presumptive load-bearing values provided in Table 1804.2 shall be used with the allowable stress design load combinations specified in Section 1605.3.

1804.2 Presumptive load-bearing values. The maximum allowable foundation pressure, lateral pressure or lateral sliding resistance values for supporting soils at or near the surface shall not exceed the values specified in Table 1804.2 unless data to substantiate the use of a higher value are submitted and approved.

Presumptive load-bearing values shall apply to materials with similar physical characteristics and dispositions.

Mud, organic silt, organic clays, peat or unprepared fill shall not be assumed to have a presumptive load bearing capacity unless data to substantiate the use of such a value are submitted.

Exception: A presumptive load-bearing capacity is permitted to be used where the building official deems the load-bearing capacity of mud, organic silt or unprepared fill is adequate for the support of lightweight and temporary structures.

1804.3 Lateral sliding resistance. The resistance of structural walls to lateral sliding shall be calculated by combining the values derived from the lateral bearing and the lateral sliding resistance shown in Table 1804.2 unless data to substantiate the use of higher values are submitted for approval.

For clay, sandy clay, silty clay and clayey silt, in no case shall the lateral sliding resistance exceed one-half the dead load.

1804.3.1 Increases in allowable lateral sliding resistance. The resistance values derived from the table are permitted to be increased by the tabular value for each additional foot (305 mm) of depth to a maximum of 15 times the tabular value.

Isolated poles for uses such as flagpoles or signs and poles used to support buildings that are not adversely affected by a $^1/_2$-inch (12.7 mm) motion at the ground surface due to short-term lateral loads are permitted to be designed using lateral-bearing values equal to two times the tabular values.

SECTION 1805
FOOTINGS AND FOUNDATIONS

1805.1 General. Footings and foundations shall be designed and constructed in accordance with Sections 1805.1 through 1805.9.

Footings and foundations shall be built on undisturbed soil or compacted fill material. Compacted fill material shall be placed in accordance with Section 1803.4.

The top surface of footings shall be level. The bottom surface of footings is permitted to have a slope not exceeding one unit vertical in 10 units horizontal (10-percent slope). Footings

TABLE 1804.2
ALLOWABLE FOUNDATION AND LATERAL PRESSURE

CLASS OF MATERIALS	ALLOWABLE FOUNDATION PRESSURE (psf)[d]	LATERAL BEARING (psf/f below natural grade)[d]	LATERAL SLIDING	
			Coefficient of friction[a]	Resistance (psf)[b]
1. Crystalline bedrock	12,000	1,200	0.70	—
2. Sedimentary and foliated rock	4,000	400	0.35	—
3. Sandy gravel and/or gravel (GW and GP)	3,000	200	0.35	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500[c]	100	—	130

For SI: 1 pound per square foot = 0.0479 kPa, 1 pound per square foot per foot = 0.157 kPa/m.

a. Coefficient to be multiplied by the dead load.

b. Lateral sliding resistance value to be multiplied by the contact area, as limited by Section 1804.3.

c. Where the building official determines that in-place soils with an allowable bearing capacity of less than 1,500 psf are likely to be present at the site, the allowable bearing capacity shall be determined by a soils investigation.

d. An increase of one-third is permitted when considering load combinations, including wind or earthquake loads, as permitted by Section 1605.3.2.

shall be stepped where it is necessary to change the elevation of the top surface of the footing or where the surface of the ground slopes more than one unit vertical in 10 units horizontal (10-percent slope).

1805.2 Depth of footings. The minimum depth of footings below the undisturbed ground surface shall be 12 inches (305 mm). Where applicable, the depth of footings shall also conform to Sections 1805.2.1 through 1805.2.3.

1805.2.1 Frost protection. Except where erected on solid rock or otherwise protected from frost, foundation walls, piers and other permanent supports of buildings and structures larger than 400 square feet (37 m^2) in area or 10 feet (3048 mm) in height shall extend below the frost line of the locality, and spread footings of adequate size shall be provided where necessary to properly distribute the load within the allowable load-bearing value of the soil. Alternatively, such structures shall be supported on piles where solid earth or rock is not available. Footings shall not bear on frozen soils unless such frozen condition is of a permanent character.

1805.2.2 Isolated footings. Footings on granular soil shall be so located that the line drawn between the lower edges of adjoining footings shall not have a slope steeper than 30 degrees (0.52 rad) with the horizontal, unless the material supporting the higher footing is braced or retained or otherwise laterally supported in an approved manner or a greater slope has been properly established by engineering analysis.

1805.2.3 Shifting or moving soils. Where it is known that the shallow subsoils are of a shifting or moving character, footings shall be carried to a sufficient depth to ensure stability.

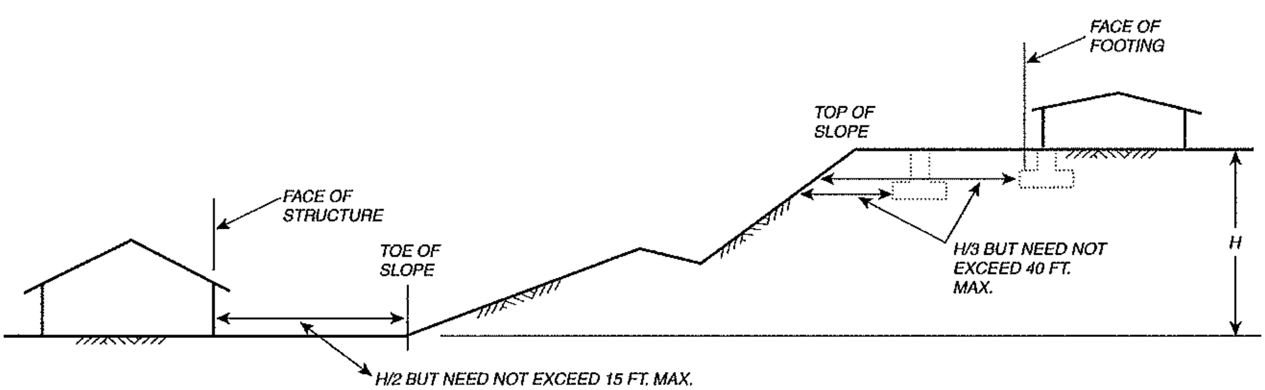
1805.3 Footings on or adjacent to slopes. The placement of buildings and structures on or adjacent to slopes steeper than one unit vertical in three units horizontal (33.3-percent slope) shall conform to Sections 1805.3.1 through 1805.3.5.

1805.3.1 Building clearance from ascending slopes. In general, buildings below slopes shall be set a sufficient distance from the slope to provide protection from slope drainage, erosion and shallow failures. Except as provided for in Section 1805.3.5 and Figure 1805.3.1, the following criteria will be assumed to provide this protection. Where the existing slope is steeper than one unit vertical in one unit horizontal (100-percent slope), the toe of the slope shall be assumed to be at the intersection of a horizontal plane drawn from the top of the foundation and a plane drawn tangent to the slope at an angle of 45 degrees (0.79 rad) to the horizontal. Where a retaining wall is constructed at the toe of the slope, the height of the slope shall be measured from the top of the wall to the top of the slope.

1805.3.2 Footing setback from descending slope surface. Footings on or adjacent to slope surfaces shall be founded in firm material with an embedment and setback from the slope surface sufficient to provide vertical and lateral support for the footing without detrimental settlement. Except as provided for in Section 1805.3.5 and Figure 1805.3.1, the following setback is deemed adequate to meet the criteria. Where the slope is steeper than 1 unit vertical in 1 unit horizontal (100-percent slope), the required setback shall be measured from an imaginary plane 45 degrees (0.79 rad) to the horizontal, projected upward from the toe of the slope.

1805.3.3 Pools. The setback between pools regulated by this code and slopes shall be equal to one half the building footing setback distance required by this section. That portion of the pool wall within a horizontal distance of 7 feet (2134 mm) from the top of the slope shall be capable of supporting the water in the pool without soil support.

1805.3.4 Foundation elevation. On graded sites, the top of any exterior foundation shall extend above the elevation of the street gutter at point of discharge or the inlet of an approved drainage device a minimum of 12 inches (305 mm) plus 2 percent. Alternate elevations are permitted subject to the approval of the building official, provided it can be dem-



For SI: 1 inch = 25.4 mm.

FIGURE 1805.3.1
FOUNDATION CLEARANCES FROM SLOPES

onstrated that required drainage to the point of discharge and away from the structure is provided at all locations on the site.

1805.3.5 [Comm 62.1805] Alternate setback and clearance. Alternate setbacks and clearances are permitted, subject to the approval of the building official.

1805.4 Footings. Footings shall be designed and constructed in accordance with Sections 1805.4.1 through 1805.4.6.

1805.4.1 Design. Footings shall be so designed that the allowable bearing capacity of the soil is not exceeded, and that differential settlement is minimized. The minimum width of footings shall be 12 inches (305 mm).

Footings in areas with expansive soils shall be designed in accordance with the provisions of Section 1805.8.

1805.4.1.1 Design loads. Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in Section 1605.3. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in Section 1607.9 are permitted to be used in designing footings.

1805.4.1.2 Vibratory loads. Where machinery operations or other vibrations are transmitted through the foundation, consideration shall be given in the footing design to prevent detrimental disturbances of the soil.

1805.4.2 Concrete footings. The design, materials and construction of concrete footings shall comply with Sections 1805.4.2.1 through 1805.4.2.6 and the provisions of Chapter 19.

Exception: Where a specific design is not provided, concrete footings supporting walls of light-frame construction are permitted to be designed in accordance with Table 1805.4.2.

1805.4.2.1 Concrete strength. Concrete in footings shall have a specified compressive strength (f'_c) of not less than 2,500 psi (17 237 kPa) at 28 days.

1805.4.2.2 Footing seismic ties. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1616, individual spread footings founded on soil defined in Section 1615.1.1 as Site Class E or F shall be interconnected by ties. Ties shall be capable of carrying, in tension or compression, a force equal to the product of the larger footing load times the seismic coefficient S_{DS} divided by 10 unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade.

1805.4.2.3 Plain concrete footings. In plain concrete footings, the edge thickness shall not be less than 8 inches (203 mm) for footings on soil.

Exception: For occupancies of Group R-3 and buildings less than two stories in height of light- frame construction, the required edge thickness is permitted to be 6 inches (152 mm), provided that the footing does not extend beyond a distance greater than the thickness of the footing on either side of the supported wall.

1805.4.2.4 Placement of concrete. Concrete footings shall not be placed through water unless a tremie or other method approved by the building official is used. Where placed under or in the presence of water, the concrete shall be deposited by approved means to ensure minimum segregation of the mix and negligible turbulence of the water.

1805.4.2.5 Protection of concrete. Concrete footings shall be protected from freezing during depositing and for a period of not less than 5 days thereafter. Water shall not be allowed to flow through the deposited concrete.

1805.4.2.6 Forming of concrete. Concrete footings are permitted to be cast against the earth where, in the opinion of the building official, soil conditions do not require forming. Where forming is required, forming shall be in accordance with Chapter 6 of ACI 318.

TABLE 1805.4.2
FOOTINGS SUPPORTING WALLS OF LIGHT-FRAME CONSTRUCTION[a, b, c, d]

NUMBER OF FLOORS SUPPORTED BY THE FOUNDATION[e]	THICKNESS OF FOUNDATION WALL (inches)		WIDTH OF FOOTING (inches)	THICKNESS OF FOOTING (inches)	DEPTH OF FOOTING BELOW UNDISTURBED GROUND SURFACE (inches)
	Concrete	Unit masonry			
1	6	6	12	6	12
2	8	8	15	7	18
3	10	10	18	8	24

For SI: 1 inch = 25.4 mm.

a. Where frost conditions are found, footings and foundations shall be as required in Section 1805.2.1.

b. The ground under the floor is permitted to be excavated to the elevation of the top of the footing.

c. Interior-stud-bearing walls are permitted to be supported by isolated footings. The footing width and length shall be twice the width shown in this table, and footings shall be spaced not more than 6 feet on center.

d. See Section 1910 for additional requirements for footings of structures assigned to Seismic Design Categories C, D, E and F.

e. Foundations are permitted to support a roof in addition to the stipulated number of floors. Foundations supporting roofs only shall be as required for supporting one floor.

1805.4.3 Masonry-unit footings. The design, materials and construction of masonry-unit footings shall comply with Sections 1805.4.3.1 and 1805.4.3.2, and the provisions of Chapter 21.

Exception: Where a specific design is not provided, masonry-unit footings supporting walls of light-frame construction are permitted to be designed in accordance with Table 1805.4.2.

1805.4.3.1 Dimensions. Masonry-unit footings shall be laid in Type M or S mortar complying with Section 2103.7 and the depth shall not be less than twice the projection beyond the wall, pier or column. The width shall not be less than 8 inches (203 mm) wider than the wall supported thereon.

1805.4.3.2 Offsets. The maximum offset of each course in brick foundation walls stepped up from the footings shall be $1^1/_2$ inches (38 mm) where laid in single courses, and 3 inches (76 mm) where laid in double courses.

1805.4.4 Steel grillage footings. Grillage footings of structural steel shapes shall be separated with approved steel spacers and shall be entirely encased in concrete with at least 6 inches (152 mm) on the bottom and at least 4 inches (102 mm) at all other points. The spaces between the shapes shall be completely filled with concrete or cement grout.

1805.4.5 Timber footings. Timber footings are permitted for buildings of Type V construction and as otherwise approved by the building official. Such footings shall be treated in accordance with AWPA C2 or C3. Treated timbers are not required where placed entirely below permanent water level, or where used as capping for wood piles that project above the water level over submerged or marsh lands. The compressive stresses perpendicular to grain in untreated timber footings supported upon piles shall not exceed 70 percent of the allowable stresses for the species and grade of timber as specified in the AFPA NDS.

1805.4.6 Wood foundations. Wood foundation systems shall be designed and installed in accordance with AFPA Technical Report No. 7. Lumber and plywood shall be treated in accordance with AWPA C22 and shall be identified in accordance with Section 2303.1.8.1.

1805.5 Foundation walls. Concrete and masonry foundation walls shall be designed in accordance with Chapter 19 or 21. Foundation walls that are laterally supported at the top and bottom and within the parameters of Tables 1805.5(1) through 1805.5(4) are permitted to be designed and constructed in accordance with Sections 1805.5.1 through 1805.5.4.

1805.5.1 Foundation wall thickness. The minimum thickness of concrete and masonry foundation walls shall comply with Sections 1805.5.1.1 through 1805.5.1.3.

1805.5.1.1 Thickness based on walls supported. The thickness of foundation walls shall not be less than the thickness of the wall supported, except that foundation walls of at least 8-inch (203 mm) nominal width are permitted to support brick-veneered frame walls and 10-inch-wide (254 mm) cavity walls provided the requirements of Section 1805.5.1.2 are met. Corbeling of masonry shall be in accordance with Section 2104.2. Where an 8-inch (203 mm) wall is corbeled, the top corbel shall be a full course of headers at least 6 inches (152 mm) in length, extending not higher than the bottom of the floor framing.

1805.5.1.2 Thickness based on soil loads, unbalanced backfill height and wall height. The thickness of foundation walls shall comply with the requirements of Table 1805.5(1) for plain masonry and plain concrete walls or Table 1805.5(2), 1805.5(3) or 1805.5(4) for reinforced concrete and masonry walls. When using the tables, masonry shall be laid in running bond and the mortar shall be Type M or S.

Unbalanced backfill height is the difference in height of the exterior and interior finish ground levels. Where an interior concrete slab is provided, the unbalanced backfill height shall be measured from the exterior finish ground level to the top of the interior concrete slab.

1805.5.1.3 Rubble stone. Foundation walls of rough or random rubble stone shall not be less than 16 inches (406 mm) thick. Rubble stone shall not be used for foundations for structures in Seismic Design Category C, D, E or F.

1805.5.2 Foundation wall materials. Foundation walls constructed in accordance with Tables 1805.5(1), 1805.5(2), 1805.5(3) or 1805.5(4) shall comply with the following:

1. Vertical reinforcement shall have a minimum yield strength of 60,000 psi (414 MPa).
2. The specified location of the reinforcement shall equal or exceed the effective depth distance, *d*, noted in Tables 1805.5(2), 1805.5(3) and 1805.5(4) and shall be measured from the face of the soil side of the wall to the center of vertical reinforcement. The reinforcement shall be placed within the tolerances specified in ACI 530.1/ASCE 6/TMS 402, Article 3.4E1 of the specified location.
3. Concrete shall have a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.
4. Grout shall have a specified compressive strength of not less than 2,000 psi (13.8 MPa) at 28 days.
5. Hollow masonry units shall comply with ASTM C 90 and shall be installed with Type M or S mortar.

1805.5.3 Alternative foundation wall reinforcement. In lieu of the reinforcement provisions in Table 1805.5(2), 1805.5(3) or 1805.5(4), alternative reinforcing bar sizes and spacings having an equivalent cross-sectional area of reinforcement per linear foot (mm) of wall is permitted to be used, provided the spacing of reinforcement does not exceed 72 inches (1829 mm) and reinforcing bar sizes do not exceed No. 11.

1805.5.4 Hollow masonry walls. At least 4 inches (102 mm) of solid masonry shall be provided at girder supports at the top of hollow masonry unit foundation walls.

TABLE 1805.5(1)
PLAIN MASONRY AND PLAIN CONCRETE FOUNDATION WALLS[a, b, c]

PLAIN MASONRY				
		MINIMUM NOMINAL WALL THICKNESS (inches)		
		Soil classes and lateral soil load[a] (psf per foot below natural grade)		
WALL HEIGHT (feet)	HEIGHT OF UNBALANCED BACKFILL (feet)	GW, GP, SW and SP soils 30	GM, GC, SM, SM-SC and ML soils 45	SC, MH, ML-CL and Inorganic CL soils 60
7	4 (or less)	8	8	8
	5	8	10	10
	6	10	12	10 (solid[c])
	7	12	10 (solid[c])	10 (solid[c])
8	4 (or less)	8	8	8
	5	8	10	12
	6	10	12	12 (solid[c])
	7	12	12 (solid[c])	Note d
	8	10 (solid[c])	12 (solid[c])	Note d
9	4 (or less)	8	8	8
	5	8	10	12
	6	12	12	12 (solid[c])
	7	12 (solid[c])	12 (solid[c])	Note d
	8	12 (solid[c])	Note d	Note d
	9	Note d	Note d	Note d
PLAIN CONCRETE				
		MINIMUM NOMINAL WALL THICKNESS (inches)		
		Soil classes and lateral soil load[a] (psf per foot below natural grade)		
WALL HEIGHT (feet)	HEIGHT OF UNBALANCED BACKFILL (feet)	GW, GP, SW and SP soils 30	GM, GC, SM, SM-SC and ML soils 45	SC, MH, ML-CL and Inorganic CL soils 60
7	4 (or less)	$7^1/_2$	$7^1/_2$	$7^1/_2$
	5	$7^1/_2$	$7^1/_2$	$7^1/_2$
	6	$7^1/_2$	$7^1/_2$	8
	7	$7^1/_2$	8	10
8	4 (or less)	$7^1/_2$	$7^1/_2$	$7^1/_2$
	5	$7^1/_2$	$7^1/_2$	$7^1/_2$
	6	$7^1/_2$	$7^1/_2$	10
	7	$7^1/_2$	10	10
	8	10	10	12
9	4 (or less)	$7^1/_2$	$7^1/_2$	$7^1/_2$
	5	$7^1/_2$	$7^1/_2$	$7^1/_2$
	6	$7^1/_2$	$7^1/_2$	10
	7	$7^1/_2$	10	10
	8	10	10	12
	9	10	12	Note e

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads, see Section 1610. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.

b. Provisions for this table are based on construction requirements specified in Section 1805.5.2.

c. Solid grouted hollow units or solid masonry units.

d. A design in compliance with Chapter 21 or reinforcement in accordance with Table 1805.5(2) is required.

e. A design in compliance with Chapter 19 is required.

TABLE 1805.5(2)
8-INCH REINFORCED CONCRETE AND MASONRY FOUNDATION WALLS WHERE d ≥ 5 INCHES[a, b, c]

WALL HEIGHT (feet)	HEIGHT OF UNBALANCED BACKFILL (feet)	VERTICAL REINFORCEMENT — Soil classes and lateral soil load[a] (psf per foot below natural grade): GW, GP, SW and SP soils 30	GM, GC, SM, SM-SC and ML soils 45	SC, MH, ML-CL and inorganic CL soils 60
7	4 (or less)	#4 at 48″ o.c.	#4 at 48″ o.c.	#4 at 48″ o.c.
	5	#4 at 48″ o.c.	#4 at 48″ o.c.	#4 at 40″ o.c.
	6	#4 at 48″ o.c.	#5 at 48″ o.c.	#5 at 40″ o.c.
	7	#4 at 40″ o.c.	#5 at 40″ o.c.	#6 at 48″ o.c.
8	4 (or less)	#4 at 48″ o.c.	#4 at 48″ o.c.	#4 at 48″ o.c.
	5	#4 at 48″ o.c.	#4 at 48″ o.c.	#4 at 40″ o.c.
	6	#4 at 48″ o.c.	#5 at 48″ o.c.	#5 at 40″ o.c.
	7	#5 at 48″ o.c.	#6 at 48″ o.c.	#6 at 40″ o.c.
	8	#5 at 40″ o.c.	#6 at 40″ o.c.	#7 at 40″ o.c.
9	4 (or less)	#4 at 48″ o.c.	#4 at 48″ o.c.	#4 at 48″ o.c.
	5	#4 at 48″ o.c.	#4 at 48″ o.c.	#5 at 48″ o.c.
	6	#4 at 48″ o.c.	#5 at 48″ o.c.	#6 at 48″ o.c.
	7	#5 at 48″ o.c.	#6 at 48″ o.c.	#7 at 48″ o.c.
	8	#5 at 40″ o.c.	#7 at 48″ o.c.	#8 at 48″ o.c.
	9	#6 at 40″ o.c.	#8 at 48″ o.c.	#8 at 32″ o.c.

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads, see Section 1610. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.

b. Provisions for this table are based on construction requirements specified in Section 1805.5.2.

c. For alternative reinforcement, see Section 1805.5.3.

TABLE 1805.5(3)
10-INCH REINFORCED CONCRETE AND MASONRY FOUNDATION WALLS WHERE d ≥ 6.75 INCHES[a, b, c]

WALL HEIGHT (feet)	HEIGHT OF UNBALANCED BACKFILL (feet)	VERTICAL REINFORCEMENT — Soil classes and lateral soil load[a] (psf per foot below natural grade): GW, GP, SW and SP soils 30	GM, GC, SM, SM-SC and ML soils 45	SC, MH, ML-CL and inorganic CL soils 60
7	4 (or less)	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 56″ o.c.
	5	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 56″ o.c.
	6	#4 at 56″ o.c.	#4 at 48″ o.c.	#4 at 40″ o.c.
	7	#4 at 56″ o.c.	#5 at 56″ o.c.	#5 at 40″ o.c.
8	4 (or less)	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 56″ o.c.
	5	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 48″ o.c.
	6	#4 at 56″ o.c.	#4 at 48″ o.c.	#5 at 56″ o.c.
	7	#4 at 48″ o.c.	#4 at 32″ o.c.	#6 at 56″ o.c.
	8	#5 at 56″ o.c.	#5 at 40″ o.c.	#7 at 56″ o.c.
9	4 (or less)	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 56″ o.c.
	5	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 48″ o.c.
	6	#4 at 56″ o.c.	#4 at 40″ o.c.	#4 at 32″ o.c.
	7	#4 at 40″ o.c.	#5 at 48″ o.c.	#6 at 48″ o.c.
	8	#4 at 32″ o.c.	#6 at 48″ o.c.	#4 at 16″ o.c.
	9	#5 at 40″ o.c.	#6 at 40″ o.c.	#7 at 40″ o.c.

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads, see Section 1610. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.

b. Provisions for this table are based on construction requirements specified in Section 1805.5.2.

c. For alternative reinforcement, see Section 1805.5.3.

TABLE 1805.5(4)
12-INCH REINFORCED CONCRETE AND MASONRY FOUNDATION WALLS WHERE d ≥ 8.75 INCHES[a, b, c]

WALL HEIGHT (feet)	HEIGHT OF UNBALANCED BACKFILL (feet)	VERTICAL REINFORCEMENT — Soil classes and lateral soil load[a] (psf per foot below natural grade): GW, GP, SW and SP soils 30	GM, GC, SM, SM-SC and ML soils 45	SC, MH, ML-CL and Inorganic CL soils 60
7	4 (or less)	#4 at 72″ o.c.	#4 at 72″ o.c.	#4 at 72″ o.c.
	5	#4 at 72″ o.c.	#4 at 72″ o.c.	#4 at 72″ o.c.
	6	#4 at 72″ o.c.	#4 at 64″ o.c.	#4 at 48″ o.c.
	7	#4 at 72″ o.c.	#4 at 48″ o.c.	#5 at 56″ o.c.
8	4 (or less)	#4 at 72″ o.c.	#4 at 72″ o.c.	#4 at 72″ o.c.
	5	#4 at 72″ o.c.	#4 at 72″ o.c.	#4 at 72″ o.c.
	6	#4 at 72″ o.c.	#4 at 56″ o.c.	#5 at 72″ o.c.
	7	#4 at 64″ o.c.	#5 at 64″ o.c.	#4 at 32″ o.c.
	8	#4 at 48″ o.c.	#4 at 32″ o.c.	#5 at 40″ o.c.
9	4 (or less)	#4 at 72″ o.c.	#4 at 72″ o.c.	#4 at 72″ o.c.
	5	#4 at 72″ o.c.	#4 at 72″ o.c.	#4 at 64″ o.c.
	6	#4 at 72″ o.c.	#4 at 56″ o.c.	#5 at 64″ o.c.
	7	#4 at 56″ o.c.	#4 at 40″ o.c.	#6 at 64″ o.c.
	8	#4 at 64″ o.c.	#6 at 64″ o.c.	#6 at 48″ o.c.
	9	#5 at 56″ o.c.	#7 at 72″ o.c.	#6 at 40″ o.c.

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads, see Section 1610. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.

b. Provisions for this table are based on construction requirements specified in Section 1805.5.2.

c. For alternative reinforcement, see Section 1805.5.3.

1805.5.5 Foundation wall drainage. Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 1806.4.2 and 1806.4.3.

1805.5.6 Pier and curtain wall foundations. Except in Seismic Design Categories D, E and F, pier and curtain wall foundations are permitted to be used to support light-frame construction not more than two stories in height, provided the following requirements are met:

1. All load-bearing walls shall be placed on continuous concrete footings bonded integrally with the exterior wall footings.
2. The minimum actual thickness of a load-bearing masonry wall shall not be less than 4 inches (102 mm) nominal or $3^5/_8$ inches (92 mm) actual thickness, and shall be bonded integrally with piers spaced 6 feet (1829 mm) on center.
3. Piers shall be constructed in accordance with Chapter 21 and the following:
   - 3.1. The unsupported height of the masonry piers shall not exceed 10 times their least dimension.
   - 3.2. Where structural clay tile or hollow concrete masonry units are used for piers supporting beams and girders, the cellular spaces shall be filled solidly with concrete or Type M or S mortar.

     Exception: Unfilled hollow piers are permitted where the unsupported height of the pier is not more than four times its least dimension.
   - 3.3. Hollow piers shall be capped with 4 inches (102 mm) of solid masonry or concrete or the cavities of the top course shall be filled with concrete or grout.
4. The maximum height of a 4-inch (102 mm) load-bearing masonry foundation wall supporting wood-framed walls and floors shall not be more than 4 feet (1219 mm) in height.
5. The unbalanced fill for 4-inch (102 mm) foundations walls shall not exceed 24 inches (610 mm) for solid masonry, nor 12 inches (305 mm) for hollow masonry.

1805.6 Foundation plate or sill bolting. Wood foundation plates or sills shall be bolted or strapped to the foundation or foundation wall as provided in Chapter 23.

1805.7 Designs employing lateral bearing. Designs to resist both axial and lateral loads employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth shall conform to the requirements of Sections 1805.7.1 through 1805.7.3.

1805.7.1 Limitations. The design procedures outlined in this section are subject to the following limitations:

1. The frictional resistance for structural walls and slabs on silts and clays shall be limited to one-half of the normal force imposed on the soil by the weight of the footing or slab.

2. Posts embedded in earth shall not be used to provide lateral support for structural or nonstructural materials such as plaster, masonry or concrete unless bracing is provided that develops the limited deflection required.

Wood poles shall be treated in accordance with AWPA C2 or C4.

1805.7.2 Design criteria. The depth to resist lateral loads shall be determined by the design criteria established in Sections 1805.7.2.1 through 1805.7.2.3, or by other methods approved by the building official.

1805.7.2.1 Nonconstrained. The following formula shall be used in determining the depth of embedment required to resist lateral loads where no constraint is provided at the ground surface, such as rigid floor or rigid ground surface pavement, and where no lateral constraint is provided above the ground surface, such as a structural diaphragm.

$$d = 0.5A[1 + (1 + (4.36h/A))^{1/2}] \qquad \textbf{(Equation 18-1)}$$

where:

A = $2.34P/S_1\,b$.

b = diameter of round post or footing or diagonal dimension of square post or footing, feet (m).

d = depth of embedment in earth in feet (m) but not over 12 feet (3658 mm) for purpose of computing lateral pressure.

h = distance in feet (m) from ground surface to point of application of "P."

P = applied lateral force in pounds (kN).

S_1 = allowable lateral soil-bearing pressure as set forth in Section 1804.3 based on a depth of one-third the depth of embedment in pounds per square foot (kPa).

S_3 = allowable lateral soil-bearing pressure as set forth in Section 1804.3 based on a depth equal to the depth of embedment in pounds per square foot (kPa).

1805.7.2.2 Constrained. The following formula shall be used to determine the depth of embedment required to resist lateral loads where constraint is provided at the ground surface, such as a rigid floor or pavement.

$$d^2 = 4.25(Ph/S_3\,b) \qquad \textbf{(Equation 18-2)}$$

or alternatively

$$d^2 = 4.25\,(M_g/S_3\,b) \qquad \textbf{(Equation 18-3)}$$

where:

M_g = moment in the post at grade, in foot-pounds (kN-m).

1805.7.2.3 Vertical load. The resistance to vertical loads shall be determined by the allowable soil-bearing pressure set forth in Table 1804.2.

1805.7.3 Backfill. The backfill in the annular space around columns not embedded in poured footings shall be by one of the following methods:

1. Backfill shall be of concrete with an ultimate strength of 2,000 pounds per square inch (13.8 MPa) at 28 days. The hole shall not be less than 4 inches (102 mm) larger than the diameter of the column at its bottom or 4 inches (102 mm) larger than the diagonal dimension of a square or rectangular column.
2. Backfill shall be of clean sand. The sand shall be thoroughly compacted by tamping in layers not more than 8 inches (203 mm) in depth.

1805.8 Design for expansive soils. Footings or foundations for buildings and structures founded on expansive soils shall be designed in accordance with Section 1805.8.1 or 1805.8.2.

Footing or foundation design need not comply with Section 1805.8.1 or 1805.8.2 where the soil is removed in accordance with Section 1805.8.3, nor where the building official approves stabilization of the soil in accordance with Section 1805.8.4.

1805.8.1 Foundations. Footings or foundations placed on or within the active zone of expansive soils shall be designed to resist differential volume changes and to prevent structural damage to the supported structure. Deflection and racking of the supported structure shall be limited to that which will not interfere with the usability and serviceability of the structure.

Foundations placed below where volume change occurs or below expansive soil shall comply with the following provisions:

1. Foundations extending into or penetrating expansive soils shall be designed to prevent uplift of the supported structure.
2. Foundations penetrating expansive soils shall be designed to resist forces exerted on the foundation due to soil volume changes or shall be isolated from the expansive soil.

1805.8.2 Slab-on-ground foundations. Slab-on-ground, mat or raft foundations on expansive soils shall be designed and constructed in accordance with WRI/CRSI Design of Slab-on-Ground Foundations or PTI Design and Construction of Post-Tensioned Slabs-On-Ground.

Exception: Slab-on-ground systems that have performed adequately in soil conditions similar to those encountered at the building site are permitted subject to the approval of the building official.

1805.8.3 Removal of expansive soil. Where expansive soil is removed in lieu of designing footings or foundations in accordance with Section 1805.8.1 or 1805.8.2, the soil shall be removed to a depth sufficient to ensure a constant moisture content in the remaining soil. Fill material shall not contain expansive soils and shall comply with Section 1803.4.

Exception: Expansive soil need not be removed to the depth of constant moisture, provided the confining pressure in the expansive soil created by the fill and supported structure exceeds the swell pressure.

1805.8.4 Stabilization. Where the active zone of expansive soils is stabilized in lieu of designing footings or foundations in accordance with Section 1805.8.1 or 1805.8.2, the soil shall be stabilized by chemical, dewatering, presaturation or equivalent techniques.

1805.9 Seismic requirements. See Section 1910 for additional requirements for footings and foundations of structures assigned to Seismic Design Categories C, D, E and F.

For structures assigned to Seismic Design Categories D, E and F, provisions of ACI 318, Sections 21.8.1 to 21.8.3, shall apply when not in conflict with the provisions of Section 1805. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.

Exception: Group R or Group U Occupancies of light-frame construction and two stories or less in height are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.

SECTION 1806 DAMPPROOFING AND WATERPROOFING

1806.1 Where required. Walls or portions thereof that retain earth and enclose interior spaces and floors below grade shall be waterproofed and dampproofed in accordance with this section, with the exception of those spaces containing groups other than residential and institutional where such omission is not detrimental to the building or occupancy.

Ventilation for crawl spaces shall comply with Section 1202.4.

1806.1.1 Story above grade. Where a basement is considered a story above grade and the finished ground level adjacent to the basement wall is below the basement floor elevation for 25 percent or more of the perimeter, the floor and walls shall be dampproofed in accordance with Section 1806.2 and a foundation drain shall be installed in accordance with Section 1806.4.2. The foundation drain shall be installed around the portion of the perimeter where the basement floor is below ground level. The provisions of Sections 1802.2.3, 1806.3 and 1806.4.1 shall not apply in this case.

1806.1.2 Underfloor space. The finished ground level of an underfloor space such as a crawl space shall not be located below the bottom of the footings. Where there is evidence that the ground water table rises to within 6 inches (152 mm) of the ground level at the outside building perimeter or where there is evidence that the surface water does not readily drain from the building site, the ground level of the underfloor space shall be as high as the outside finished ground level, unless an approved drainage system is provided. The provisions of Sections 1802.2.3, 1806.2, 1806.3 and 1806.4 shall not apply in this case.

1806.1.2.1 Flood hazard areas. For buildings and structures in flood hazard areas as established in Section 1612.3, the finished ground level of an underfloor space such as a crawl space shall be equal to or higher than the outside finished ground level.

1806.1.3 Ground-water control. Where the ground-water table is lowered and maintained at an elevation not less than 6 inches (152 mm) below the bottom of the lowest floor, the floor and walls shall be dampproofed in accordance with Section 1806.2. The design of the system to lower the ground-water table shall be based on accepted principles of engineering that shall consider, but not necessarily be limited to, permeability of the soil, rate at which water enters the drainage system, rated capacity of pumps, head against which pumps are to pump; and the rated capacity of the disposal area of the system.

1806.2 Dampproofing required. Where hydrostatic pressure will not occur as determined by Section 1802.2.3, floors and walls for other than wood foundation systems shall be dampproofed in accordance with this section. Wood foundation systems shall be constructed in accordance with AFPA TR7.

1806.2.1 Floors. Dampproofing materials for floors shall be installed between the floor and the base course required by Section 1806.4.1, except where a separate floor is provided above a concrete slab.

Where installed beneath the slab, dampproofing shall consist of not less than 6-mil (0.006 inch; 0.152 mm) polyethylene with joints lapped not less than 6 inches (152 mm), or other approved methods or materials. Where permitted to be installed on top of the slab, dampproofing shall consist of mopped-on bitumen, not less than 4-mil (0.004 inch; 0.102 mm) polyethylene, or other approved methods or materials. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

1806.2.2 Walls. Dampproofing materials for walls shall be installed on the exterior surface of the wall, and shall extend from the top of the footing to above ground level.

Dampproofing shall consist of a bituminous material, 3 pounds per square yard (16 N/m^2) of acrylic modified cement, $^1/_8$-inch (3.2 mm) coat of surface-bonding mortar complying with ASTM C 887, any of the materials permitted for waterproofing by Section 1806.3.2, or other approved methods or materials.

1806.2.2.1 Surface preparation of walls. Prior to application of dampproofing materials on concrete walls, holes and recesses resulting from the removal of form ties shall be sealed with a bituminous material or other approved methods or materials. Unit masonry walls shall be parged on the exterior surface below ground level with not less than $^3/_8$ inch (9.5 mm) of portland cement mortar. The parging shall be coved at the footing.

Exception: Parging of unit masonry walls is not required where a material is approved for direct application to the masonry.

1806.3 Waterproofing required. Where the ground-water investigation required by Section 1802.2.3 indicates that a hydrostatic pressure condition exists, and the design does not include a ground-water control system as described in Section 1806.1.3, walls and floors shall be waterproofed in accordance with this section.

1806.3.1 Floors. Floors required to be waterproofed shall be of concrete, designed and constructed to withstand the hydrostatic pressures to which the floors will be subjected.

Waterproofing shall be accomplished by placing a membrane of rubberized asphalt, butyl rubber, or not less than 6-mil (0.006 inch; 0.152 mm) polyvinyl chloride with joints lapped not less than 6 inches (152 mm) or other approved materials under the slab. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

1806.3.2 Walls. Walls required to be waterproofed shall be of concrete or masonry and shall be designed and constructed to withstand the hydrostatic pressures and other lateral loads to which the walls will be subjected.

Waterproofing shall be applied from the bottom of the wall to not less than 12 inches (305 mm) above the maximum elevation of the ground water table. The remainder of the wall shall be dampproofed in accordance with Section 1806.2.2. Waterproofing shall consist of two-ply hot-mopped felts, not less than 6-mil (0.006 inch; 0.152 mm) polyvinyl chloride, 40-mil (0.040 inch; 1.02 mm) polymer-modified asphalt, 6-mil (0.006 inch; 0.152 mm) polyethylene or other approved methods or materials capable of bridging nonstructural cracks. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

1806.3.2.1 Surface preparation of walls. Prior to the application of waterproofing materials on concrete or masonry walls, the walls shall be prepared in accordance with Section 1806.2.2.1.

1806.3.3 Joints and penetrations. Joints in walls and floors, joints between the wall and floor, and penetrations of the wall and floor shall be made watertight utilizing approved methods and materials.

1806.4 Subsoil drainage system. Where a hydrostatic pressure condition does not exist, dampproofing shall be provided and a base shall be installed under the floor and a drain installed around the foundation perimeter. A subsoil drainage system designed and constructed in accordance with Section 1806.1.3 shall be deemed adequate for lowering the ground-water table.

1806.4.1 Floor base course. Floors of basements, except as provided for in Section 1806.1.1, shall be placed over a floor base course not less than 4 inches (102 mm) in thickness that consists of gravel or crushed stone containing not more than 10 percent of material that passes through a No. 4 (4.75 mm) sieve.

Exception: Where a site is located in well-drained gravel or sand/gravel mixture soils, a floor base course is not required.

1806.4.2 Foundation drain. A drain shall be placed around the perimeter of a foundation that consists of gravel or crushed stone containing not more than 10 percent material that passes through a No. 4 (4.75 mm) sieve. The drain shall extend a minimum of 12 inches (305 mm) beyond the outside edge of the footing. The thickness shall be such that the bottom of the drain is not higher than the bottom of the base under the floor, and that the top of the drain is not less than 6 inches (152 mm) above the top of the footing. The top of the drain shall be covered with an approved filter membrane material. Where a drain tile or perforated pipe is used, the invert of the pipe or tile shall not be higher than the floor elevation. The top of joints or the top of perforations shall be protected with an approved filter membrane material. The pipe or tile shall be placed on not less than 2 inches (51 mm) of gravel or crushed stone complying with Section 1806.4.1, and shall be covered with not less than 6 inches (152 mm) of the same material.

1806.4.3 Drainage discharge. The floor base and foundation perimeter drain shall discharge by gravity or mechanical means into an approved drainage system that complies with the *International Plumbing Code.*

Exception: Where a site is located in well-drained gravel or sand/gravel mixture soils, a dedicated drainage system is not required.

SECTION 1807
PIER AND PILE FOUNDATIONS

1807.1 Definitions. The following words and terms shall, for the purposes of this section, have the meanings shown herein.

NEUTRAL PLANE. A pile's neutral plane is the level at which drag load, accumulated from the top down, added to the long-term static service load, equals the upward acting shaft resistance accumulated from the bottom up, added to the pile's toe resistance. [Comm 62.1807 (1)]

PIER FOUNDATIONS. Pier foundations consist of isolated masonry or cast-in-place concrete structural elements extending into firm materials. Piers are relatively short in comparison to their width, with lengths less than or equal to 12 times the least horizontal dimension of the pier. Piers derive their load-carrying capacity through skin friction, through end bearing, or a combination of both.

Belled piers. Belled piers are cast-in-place concrete piers constructed with a base that is larger than the diameter of the remainder of the pier. The belled base is designed to increase the load-bearing area of the pier in end bearing.

PILE FOUNDATIONS. Pile foundations consist of concrete, wood or steel structural elements either driven into the ground or cast-in-place. Piles are relatively slender in comparison to their length, with lengths exceeding 12 times the least horizontal dimension. Piles derive their load-carrying capacity through skin friction, through end bearing, or a combination of both.

Augered uncased piles. Augered uncased piles are constructed by depositing concrete into an uncased augered hole, either during or after the withdrawal of the auger.

Caisson piles. Caisson piles are cast-in-place concrete piles extending into bedrock. The upper portion of a caisson pile consists of a cased pile that extends to the bedrock. The lower portion of the caisson pile consists of an uncased socket drilled into the bedrock.

Concrete-filled steel pipe and tube piles. Concrete-filled steel pipe and tube piles are constructed by driving a steel pipe or tube section into the soil and filling the pipe or tube section with concrete. The steel pipe or tube section is left in place during and after the deposition of the concrete.

Driven uncased piles. Driven uncased piles are constructed by driving a steel shell into the soil to shore an unexcavated hole that is later filled with concrete. The steel casing is lifted out of the hole during the deposition of the concrete.

Enlarged base piles. Enlarged base piles are cast-in- place concrete piles constructed with a base that is larger than the diameter of the remainder of the pile. The enlarged base is designed to increase the load-bearing area of the pile in end bearing.

1807.2 Piers and piles—general requirements.

1807.2.1 General. Pier and pile foundations shall be designed and installed on the basis of a foundation investigation as defined in Section 1802, unless sufficient data upon which to base the design and installation is available.

Comm 62.1807 (2) Downdrag. Investigations and reports for pier or pile foundations shall include analysis of whether downdrag is anticipated. Where downdrag is anticipated, the report shall include a determination of the position of the pile's neutral plane, an estimate of the soil settlement at the neutral plane, and a determination of the maximum load at the neutral plane.

The investigation and report provisions of Section 1802 shall be expanded to include, but not be limited to, the following:

1. Recommended pier or pile types and installed capacities.
2. Driving criteria.
3. Installation procedures.
4. Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity where required).
5. Pier or pile load test requirements.
6. Durability of pier or pile materials.
7. Designation of bearing stratum or strata.
8. Reductions for group action, where necessary.

1807.2.2 Special types of piles. The use of types of piles not specifically mentioned herein is permitted, subject to the approval of the building official, upon the submission of acceptable test data, calculations and other information relating to the structural properties and load capacity of such piles. The allowable stresses shall not in any case exceed the limitations specified herein.

1807.2.3 Pile caps. Pile caps shall be of reinforced concrete. The soil immediately below the pile cap shall not be considered as carrying any vertical load. The tops of piles shall be embedded not less than 3 inches (76 mm) into pile caps and the caps shall extend at least 4 inches (102 mm) beyond the edges of piles. The tops of piles shall be cut back to sound material before capping.

1807.2.4 Stability. Piers or piles shall be braced to provide lateral stability in all directions. Three or more piles connected by a rigid cap shall be considered braced, provided that the piles are located in radial directions from the centroid of the group not less than 60 degrees (1 rad) apart. A two-pile group in a rigid cap shall be considered to be braced along the axis connecting the two piles. Methods used to brace piers or piles shall be subject to the approval of the building official.

Piles supporting walls shall be driven alternately in lines spaced at least 1 foot (305 mm) apart and located symmetrically under the center of gravity of the wall load carried, unless effective measures are taken to provide for eccentricity and lateral forces, or the wall piles are adequately braced to provide for lateral stability. A single row of piles without lateral bracing is permitted for one- and two-family dwellings and lightweight construction not exceeding two stories or 35 feet (10 668 mm) in height, provided the centers of the piles are located within the width of the foundation wall.

1807.2.5 Structural integrity. Piers or piles shall be installed in such a manner and sequence as to prevent distortion or damage to piles being installed or already in place to the extent that such distortion or damage affects the structural integrity of the piles.

1807.2.6 Spacing. The center-to-center spacing of piers or piles shall be as recommended in the soils report.

1807.2.7 Splices. Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the pier or pile during installation and subsequent thereto and shall be of adequate strength to transmit the vertical and lateral loads and moments occurring at the location of the splice during driving and under service loading. Splices shall develop not less than 50 percent of the least capacity of the pier or pile in bending. In addition, splices occurring in the upper 10 feet (3048 mm) of the embedded portion of the pier or pile shall be capable of resisting at allowable working stresses the moment and shear that would result from an assumed eccentricity of the pier or pile load of 3 inches (76 mm), or the pier or pile shall be braced in accordance with Section 1807.2.4 to other piers or piles that do not have splices in the upper 10 feet (3048 mm) of embedment.

1807.2.8 Allowable pier or pile loads.

1807.2.8.1 [Comm 62.1807 (3)] Determination of allowable loads.

(a) The allowable axial and lateral loads on piers or piles shall be determined by an approved formula, load tests or static analysis.

(b) The factor of safety to be used for pier or pile design shall depend on the extent of field testing performed to verify capacity.

(c) If the ultimate capacity is assessed solely by static analysis, a minimum factor of safety of 3.0 shall be applied to the ultimate capacity to determine allowable load capacity.

(d) If only static analysis and dynamic field testing are performed, a minimum factor of safety of 2.5 shall be applied to the ultimate capacity to determine load capacity.

(e) If one or more static load tests are performed, in addition to the analysis and tests described above, a minimum factor of safety of 2.0 shall be applied to the ultimate allowable capacity.

(f) A minimum factor of safety of 2.0 shall be used for occupiable structures provided that all of the conditions in pars. (a) to (e) are met. A minimum factor of safety of 1.5 may be used for nonoccupiable structures, provided that the deep foundations are required only to control settlement, and it can be demonstrated that deep foundations are not required to prevent a bearing capacity failure.

1807.2.8.2 Driving criteria. The allowable compressive load on any pile where determined by the application of an approved driving formula shall not exceed 40 tons (356 kN). For allowable loads above 40 tons (356 kN), the wave equation method of analysis shall be used to estimate pile driveability of both driving stresses and net displacement per blow at the ultimate load. Allowable loads shall be verified by load tests in accordance with Section 1807.2.8.3. The formula or wave equation load shall be determined for gravity-drop or power-actuated hammers and the hammer energy used shall be the maximum consistent with the size, strength and weight of the driven piles. The use of a follower is permitted only with the approval of the building official. The introduction of fresh hammer cushion or pile cushion material just prior to final penetration is not permitted.

1807.2.8.3 Load tests. Where greater compressive loads per pier or pile than permitted by Section 1807.2.10 are desired or where the design load for any pier or pile foundation is in doubt, control test piers or piles shall be tested in accordance with ASTM D 1143 or ASTM D 4945. At least one pier or pile shall be test loaded in each area of uniform subsoil conditions. Where required by the building official, additional piers or piles shall be load tested where necessary to establish the safe design capacity. The resulting allowable loads shall not be more than one-half of that test load, which produces a permanent net settlement of not more than 0.01 inch per ton (0.0285 mm/kN) of test load, and not more than $^3/_4$ inch (19.1 mm). In subsequent driving of the balance of foundation piles, all piles shall be deemed to have a supporting capacity equal to the control pile where such piles are of the same type, size and relative length as the test pile; are installed using the same or comparable methods and equipment as the test pile; are installed in similar subsoil conditions as the test pile; and where the rate of penetration (e.g., net displacement per blow) of such piles is equal to or less than that of the test pile through a comparable driving distance.

Comm 62.1807 (4) Alternative determination of allowable load. The ultimate capacity of the pile shall be defined as the load at which the average pile head deflection is defined by the following equation:

$$\delta = (Pl/AE) + 0.15'' + (B/120)$$

where:

δ = average pile head deflection, inches (mm)

P = applied load, pounds (N)

l = pile length, inches (mm)

A = transformed pile area of pile (to steel)

E = modulus of elasticity (of steel)

B = outside diameter (or width) of pile, inches (mm)

The calculation shall be predicated on an assumed end-bearing condition.

1807.2.8.4 Allowable frictional resistance. The assumed frictional resistance developed by any pier or uncased cast-in-place pile shall not exceed one-sixth of the bearing value of the soil material at minimum depth as set forth in Table 1804.2, up to a maximum of 500 pounds per square foot (24 kPa), unless a greater value is allowed by the building official after a soil investigation as specified in Section 1802 is submitted. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless recommended by a soil investigation as specified in Section 1802.

1807.2.8.5 Uplift capacity. Where required by the design, the uplift capacity of a single pier or pile shall be determined in accordance with ASTM D 3689, or an approved method of analysis based on a minimum safety factor of three. The maximum allowable uplift load shall be one-half that load which produces an upward movement of the pier or pile but equal to the gross elastic extension of the pier or pile plus 0.1 inch (2.5 mm). For pile groups subjected to uplift, the allowable working uplift load for the group shall be the lesser of:

1. The proposed individual pile uplift working load times the number of piles in the group.

2. Two-thirds of the effective weight of the pile group and the soil contained within a block defined by the perimeter of the group and the length of the pile.

1807.2.8.6 Load-bearing capacity. Piers, individual piles and groups of piles shall develop ultimate load capacities of at least twice the design working loads in the designated load-bearing layers. Analysis shall show that no soil layer underlying the designated load-bearing layers causes the load-bearing capacity safety factor to be less than two.

1807.2.8.7 Bent piers or piles. The load-bearing capacity of piers or piles discovered to have a sharp or sweeping bend shall be determined by an approved method of analysis or by load testing a representative pier or pile.

1807.2.8.8 Overloads on piers or piles. The maximum compressive load on any pier or pile due to mislocation shall not exceed 110 percent of the allowable design load.

1807.2.9 Lateral support.

1807.2.9.1 General. Any soil other than fluid soil shall be deemed to afford sufficient lateral support to the pier or pile to prevent buckling and to permit the design of the pier or pile in accordance with accepted engineering practice and the applicable provisions of this code.

1807.2.9.2 Unbraced piles. Piles standing unbraced in air, water or in fluid soils shall be designed as columns in accordance with the provisions of this code. Such piles driven into firm ground can be considered fixed and laterally supported at 5 feet (1524 mm) below the ground surface and in soft material at 10 feet (3048 mm) below the ground surface unless otherwise prescribed by the building official after a foundation investigation by an approved agency.

1807.2.9.3 Allowable lateral load. Where required by the design, the lateral load capacity of a pier, a single pile or a pile group shall be determined by an approved method of analysis or by lateral load tests to at least twice the proposed design working load. The resulting allowable load shall not be more than one-half of that test load that produces a gross lateral movement of 1 inch (25 mm) at the ground surface.

1807.2.10 Use of higher allowable pier or pile stresses. Allowable stresses greater than those specified for piers or for each pile type in Sections 1808 and 1809 are permitted where supporting data justifying such higher stresses is filed with the building official. Such substantiating data shall include:

1. A soils investigation in accordance with Section 1802.
2. Pier or pile load tests in accordance with Section 1807.2.8.3, regardless of the load supported by the pier or pile.

The design and installation of the pier or pile foundation shall be under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pier or pile foundations who shall certify to the building official that the piers or piles as installed satisfy the design criteria.

1807.2.11 [Comm 62.1807 (5)] Piles in subsiding areas. Where piles are driven through subsiding fills or other subsiding strata and derive support from underlying firmer materials, consideration shall be given to the downward drag load that may be imposed on the piles by the subsiding upper strata.

Where the influence of subsiding fills is considered as imposing loads on the pile, the allowable stresses specified in this chapter are permitted to be increased where satisfactory substantiating data are submitted.

The position of the pile's neutral plane shall be determined, and the settlement of the soil at the level of the neutral plane shall be estimated. The maximum load in the pile, which occurs at the neutral plane, shall be determined.

1807.2.12 Settlement analysis. The settlement of piers, individual piles or groups of piles shall be estimated based on approved methods of analysis. The predicted settlement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any stresses to exceed allowable values.

1807.2.13 Pre-excavation. The use of jetting, augering or other methods of pre-excavation shall be subject to the approval of the building official. Where permitted, pre-excavation shall be carried out in the same manner as used for piers or piles subject to load tests and in such a manner that will not impair the carrying capacity of the piers or piles already in place or damage adjacent structures. Pile tips shall be driven below the pre-excavated depth until the required resistance or penetration is obtained.

1807.2.14 Installation sequence. Piles shall be installed in such sequence as to avoid compacting the surrounding soil to the extent that other piles cannot be installed properly, and to prevent ground movements that are capable of damaging adjacent structures.

1807.2.15 Use of vibratory drivers. Vibratory drivers shall only be used to install piles where the pile load capacity is verified by load tests in accordance with Section 1807.2.8.3. The installation of production piles shall be controlled according to power consumption, rate of penetration or other approved means that ensure pile capacities equal or exceed those of the test piles.

1807.2.16 Pile driveability. Pile cross sections shall be of sufficient size and strength to withstand driving stresses without damage to the pile, and to provide sufficient stiffness to transmit the required driving forces.

1807.2.17 Protection of pile materials. Where boring records or site conditions indicate possible deleterious action on pier or pile materials because of soil constituents, changing water levels or other factors, the pier or pile materials shall be adequately protected by materials, methods or processes approved by the building official. Protective materials shall be applied to the piles so as not to be rendered ineffective by driving. The effectiveness of such methods or processes for the particular purpose shall have been thoroughly established by satisfactory service records or other evidence that demonstrates the effectiveness of such protective measures.

1807.2.18 Use of existing piers or piles. Piers or piles left in place where a structure has been demolished shall not be used for the support of new construction unless satisfactory evidence is submitted to the building official, which indicates that the piers or piles are sound and meet the requirements of this code. Such piers or piles shall be load tested or redriven to verify their capacities. The design load applied to such piers or piles shall be the lowest allowable load as determined by tests or redriving data.

1807.2.19 Heaved piles. Piles that have heaved during the driving of adjacent piles shall be redriven as necessary to develop the required capacity and penetration, or the capacity of the pile shall be verified by load tests in accordance with Section 1807.2.8.3.

1807.2.20 Identification. Pier or pile materials shall be identified for conformity to the specified grade with this identity maintained continuously from the point of manufacture to the point of installation or shall be tested by an approved agency to determine conformity to the specified grade. The approved agency shall furnish an affidavit of compliance to the building official.

1807.2.21 Pier or pile location plan. A plan showing the location and designation of piers or piles by an identification system shall be filed with the building official prior to installation of such piers or piles. Detailed records for piers or individual piles shall bear an identification corresponding to that shown on the plan.

1807.2.22 Special inspection. Special inspections in accordance with Sections 1704.8 and 1704.9 shall be provided for piles and piers, respectively.

1807.2.23 Seismic design of piers or piles.

1807.2.23.1 Seismic Design Category C. Where a structure is assigned to Seismic Design Category C in accordance with Section 1616, the following shall apply. Individual pile caps, piers or piles shall be interconnected by ties. Ties shall be capable of carrying, in tension and compression, a force equal to the product of the larger pile cap or column load times the seismic coefficient S_{DS} divided by 10 unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs-on-grade or reinforced concrete slabs-on-grade or confinement by competent rock, hard cohesive soils, or very dense granular soils.

Exception: Piers supporting foundation walls, isolated interior posts detailed so the pier is not subject to lateral loads, lightly loaded exterior decks and patios, of Group R-3 and Group U occupancies not exceeding two stories of light-frame construction, are not subject to interconnection if it can be shown the soils are of adequate stiffness, subject to the approval of the building official.

1807.2.23.1.1 Connection to pile cap. Concrete piles and concrete-filled steel pipe piles shall be connected to the pile cap by embedding the pile reinforcement or field-placed dowels anchored in the concrete pile in the pile cap for a distance equal to the development length. For deformed bars, the development length is the full development length for compression or tension without reduction in length for excess area. Alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the pile will be permitted provided the design is such that any hinging occurs in the confined region. Ends of hoops or ties shall be terminated with 135-degree (2.35 rad) hooks into the confined concrete core. The minimum transverse steel ratio for confinement shall not be less than one-half of that required for columns.

For resistance to uplift forces, anchorage of steel pipe, concrete-filled steel pipe or H-piles to the pile cap shall be made by means other than concrete bond to the bare steel section.

Splices of pile segments shall develop the full strength of the pile.

1807.2.23.1.2 Design details. Pier or pile moments, shears and lateral deflections used for design shall be established considering the nonlinear interaction of the shaft and soil, as recommended by a registered design professional. Where the ratio of the depth of embedment of the pile-to-pile diameter or width is less than or equal to six, the pile may be assumed to be rigid.

Pier or pile group effects from soil on lateral pile capacity shall be considered where pile center-to-center spacing in the direction of lateral force is less than 8 pile diameters. Pile group effects on vertical capacity shall be considered where pile center-to-center spacing is less than 3 pile diameters.

Where a minimum length for reinforcement or the extent of closely spaced confinement reinforcement is specified at the top of the pier or pile, provisions shall be made so that those specified lengths or extents are maintained after pier or pile cut-off.

1807.2.23.2 Seismic Design Category D, E or F. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1616, the requirements for Seismic Design Category C given in Section 1807.2.23.1 shall be met, in addition to the following. Provisions of ACI 318, Section 21.8.4, shall apply when not in conflict with the provisions of Sections 1807 through 1811. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.

Exception: Group R or Group U Occupancies of light-frame construction and two stories or less in height are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.

1807.2.23.2.1 Design details. Piers or piles shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-pile-structure interaction coupled with pier or pile deformations induced by lateral pier or pile resistance to structure seismic forces. Concrete piers or piles on Site Class E or F sites, as determined in Section 1615.1.1, shall be designed and detailed in accordance with requirements for concrete special moment frames (see Table 1617.6 for reference) within 7 pile diameters of the pile cap and the interfaces of soft to medium stiff clay or liquefiable strata. For precast prestressed concrete piles, detailing provisions as given in Sections 1808.2.3.2.1 and 1808.2.3.2.2 shall apply.

Grade beams shall be designed as beams in accordance with ACI 318, Chapter 21. When grade beams have the capacity to resist the forces from the load combinations in Section 1617.1.2, they need not conform to ACI 318, Chapter 21.

1807.2.23.2.2 Connection to pile cap. Design of anchorage of piles into the pile cap shall consider the combined effect of uplift forces and fixity to the pile cap. Anchorage shall develop a minimum of 25 percent of the strength of the pile in tension. For piles required to resist uplift forces or provide rotational restraint, anchorage into the pile cap shall be capable of developing, at a minimum, the lesser of the following:

1. The tensile strength of the longitudinal reinforcement in a concrete pile or the tensile strength of a steel pile.
2. 1.3 times the pile uplift capacity in the soil.

1807.2.23.2.3 Flexural strength. Where the vertical lateral-force-resisting elements are columns, the grade beam or pile cap flexural strengths shall exceed the column flexural strength.

Batter piles and their connection shall be capable of resisting forces from the load combinations of Section 1605.4.

SECTION 1808
DRIVEN PILE FOUNDATIONS

1808.1 Timber piles. Timber piles shall be designed in accordance with the AFPA NDS.

1808.1.1 Materials. Round timber piles shall conform to ASTM D 25. Sawn timber piles shall conform to DOC PS-20.

1808.1.2 Preservative treatment. Timber piles used to support permanent structures shall be treated in accordance with this section unless it is established that the tops of the untreated timber piles will be below the lowest ground water level assumed to exist during the life of the structure. Preservative and minimum final retention shall be in accordance with AWPA C3 for round timber piles, and AWPA C24 for sawn timber piles. Preservative-treated timber piles shall be subject to a quality control program administered by an approved agency. Pile cutoffs shall be treated in accordance with AWPA M4.

1808.1.3 [Comm 62.1808] End-supported piles. Any sudden decrease in driving resistance of an end-supported timber pile shall be investigated with regard to the possibility of damage. If the sudden decrease in driving resistance cannot be correlated to load-bearing data, the pile shall be removed for inspection or rejected, or shall be assigned a reduced capacity commensurate with the loss of end-bearing in lieu of removing or rejecting the pile.

1808.2 Precast concrete piles.

1808.2.1 General. The materials, reinforcement and installation of precast concrete piles shall conform to Sections 1808.2.1.1 through 1808.2.1.4.

1808.2.1.1 Design and manufacture. Piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by handling, driving and service loads.

1808.2.1.2 Minimum dimension. The minimum lateral dimension shall be 8 inches (203 mm). Corners of square piles shall be chamfered.

1808.2.1.3 Reinforcement. Longitudinal steel shall be arranged in a symmetrical pattern and shall be laterally tied with steel ties or wire spiral spaced not more than 4 inches (102 mm) apart, center-to-center, for a distance of 2 feet (610 mm) from the ends of the pile; and not more than 6 inches (152 mm) elsewhere except that at the ends of each pile, the first five ties or spirals shall be spaced 1 inch (25.4 mm) center to center. The gage of ties and spirals shall be as follows:

For piles having a diameter of 16 inches (406 mm) or less, wire shall not be smaller than 0.22 inch (5.6 mm) (No. 5 gage).

For piles having a diameter of more than 16 inches (406 mm) and less than 20 inches (508 mm), wire shall not be smaller than 0.238 inch (6 mm) (No. 4 gage).

For piles having a diameter of 20 inches (508 mm) and larger, wire shall not be smaller than $^{1}/_{4}$ inch (6.4 mm) round or 0.259 inch (6.6 mm) (No. 3 gage).

1808.2.1.4 Installation. Piles shall be handled and driven so as not to cause injury or overstressing, which affects durability or strength.

1808.2.2 Precast nonprestressed piles. Precast nonprestressed concrete piles shall conform to Sections 1808.2.2.1 through 1808.2.2.5.

1808.2.2.1 Materials. Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 3,000 psi (20.68 MPa).

1808.2.2.2 Minimum reinforcement. The minimum amount of longitudinal reinforcement shall be 0.8 percent of the concrete section and shall consist of at least four bars.

1808.2.2.2.1 Seismic reinforcement in Seismic Design Category C. Where a structure is assigned to Seismic Design Category C in accordance with Section 1616, longitudinal reinforcement shall be provided for precast concrete piles with a minimum steel ratio of 0.01. The longtitudinal reinforcing shall be confined with closed ties or equivalent spirals of a minimum of a $^1/_4$ inch (6.4 mm) diameter. Ties or equivalent spirals shall be provided at a maximum 8-bar-diameter spacing with a maximum spacing of 6 inches (152 mm). Reinforcement including ties shall be full length.

1808.2.2.2.2 Seismic reinforcement in Seismic Design Category D, E or F. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1616, the requirements for Seismic Design Category C in Section 1808.2.2.2.1 shall be met. Ties or equivalent spirals shall be provided at a maximum 6-longitudinal-bar-diameter spacing not to exceed a maximum of 4 inches (102 mm) on center. In addition, ties in precast concrete piles shall be provided for at least the top half of the pile.

1808.2.2.3 Allowable stresses. The allowable compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f'_c) applied to the gross cross-sectional area of the pile. The allowable compressive stress in the reinforcing steel shall not exceed 40 percent of the yield strength of the steel (f_y) or a maximum of 30,000 psi (207 MPa). The allowable tensile stress in the reinforcing steel shall not exceed 50 percent of the yield strength of the steel (f_y) or a maximum of 24,000 psi (165 MPa).

1808.2.2.4 Installation. A precast concrete pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the 28-day specified compressive strength (f'_c), but not less than the strength sufficient to withstand handling and driving forces.

1808.2.2.5 Concrete cover. Reinforcement for piles that are not manufactured under plant conditions shall have a concrete cover of not less than 2 inches (51 mm).

Reinforcement for piles manufactured under plant control conditions shall have a concrete cover of not less than $1^1/_4$ inches (32 mm) for No. 5 bars and smaller, and not less than $1^1/_2$ inches (38 mm) for No. 6 through No. 11 bars except that longitudinal bars spaced less than $1^1/_2$ inches (38 mm) clear distance apart shall be considered bundled bars for which the minimum concrete cover shall be equal to that for the equivalent diameter of the bundled bars.

Reinforcement for piles exposed to sea water shall have a concrete cover of not less than 3 inches (76 mm).

1808.2.3 Precast prestressed piles. Precast prestressed concrete piles shall conform to the requirements of Sections 1808.2.3.1 through 1808.2.3.5.

1808.2.3.1 Materials. Prestressing steel shall conform to ASTM A 416. Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 5,000 psi (34.48 MPa).

1808.2.3.2 Design. Precast prestressed piles shall be designed to resist stresses induced by handling and driving as well as by loads. The effective prestress in the pile shall not be less than 400 psi (2.76 MPa) for piles up to 30 feet (9144 mm) in length, 550 psi (3.79 MPa) for piles up to 50 feet (15 240 mm) in length, and 700 psi (4.83 MPa) for piles greater than 50 feet (15 240 mm) in length.

Effective prestress shall be based on an assumed loss of 30,000 psi (207 MPa) in the prestressing steel. The tensile stress in the prestressing steel shall not exceed the values specified in ACI 318.

1808.2.3.2.1 Design in Seismic Design Category C. Where a structure is assigned to Seismic Design Category C in accordance with Section 1616, the following shall apply. The minimum volumetric ratio of spiral reinforcement in the ductile region shall be equal to 0.007. The spiral reinforcement shall not be less than the amount required by the following formula:

$$\rho_s = 0.12 f'_c / f_{yh} \qquad \textbf{(Equation 18-4)}$$

where:

f'_c = $\leq$ 6,000 psi (41.4 MPa).

f_{yh} = Yield strength of spiral reinforcement $\leq$ 85,000 psi (586 MPa).

ρ_s = Spiral reinforcement index (vol. spiral/ vol. core).

The pile cap connection by means of dowels as indicated in Section 1807.2.23.1 is permitted. Pile cap connection by means of developing pile reinforcing strand is permitted provided that the pile reinforcing strand results in a ductile connection.

1808.2.3.2.2 Design in Seismic Design Category D, E or F. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1616, the requirements for Seismic Design Category C in Section 1808.2.3.2.1 shall be met, in addition to the following:

1. Requirements in ACI 318, Chapter 21, need not apply.
2. Where the total pile length in the soil is 35 feet (10 668 mm) or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 35 feet (10 668 mm), the ductile pile region shall be taken as the greater of 35 feet (10 668 mm) or the distance from the un-

derside of the pile cap to the point of zero curvature plus three times the least pile dimension.

3. In the ductile region, the center-to-center spacing of the spirals or hoop reinforcement shall not exceed one-fifth of the least pile dimension, six times the diameter of the longitudinal strand, or 8 inches (203 mm), whichever is smaller.
4. Spiral reinforcement shall be spliced by lapping one full turn by welding or by the use of a mechanical connector. Where spiral reinforcement is lap spliced, the ends of the spiral shall terminate in a seismic hook in accordance with ACI 318, except that the bend shall not be less than 135 degrees (2.35 rad). Welded splices and mechanical connectors shall comply with Section 12.14.3 of ACI 318.
5. Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with the following:

$$\rho_s = 0.25(f'_c/f_{yh})(A_g/A_{ch} - 1.0)[0.5 + 1.4P/(f'_c A_g)]$$

(Equation 18-5)

but not less than:

$$\rho_s = 0.12(f'_c/f_{yh})[0.5 + 1.4P/(f'_c A_g)]$$

(Equation 18-6)

and need not exceed:

$\rho_s = 0.021$ **(Equation 18-7)**

where:

A_g = Pile cross-sectional area, square inches (mm^2).

A_{ch} = Core area defined by spiral outside diameter, square inches (mm^2).

f'_c = $\leq$ 6,000 psi (41.4 MPa).

f_{yh} = Yield strength of spiral reinforcement $\leq$ 85,000 psi (586 MPa).

P = Axial load on pile, pounds (kN), as determined from Formulas 16-5 and 16-6.

ρ_s = Volumetric ratio (vol. spiral/ vol. core).

This required amount of spiral reinforcement is permitted to be obtained by providing an inner and outer spiral.

6. When transverse reinforcement consists of rectangular hoops and cross-ties, the total cross-sectional area of lateral transverse reinforcement in the ductile region with spacings, and perpendicular to dimension, h_c, shall conform to:

$$A_{sh} = 0.3sh_c(f'_c/f_{yh})(A_g/A_{ch} - 1.0)[0.5 + 1.4P/(f'_c A_g)]$$

(Equation 18-8)

but not less than:

$$A_{sh} = 0.12sh_c(f'_c/f_{yh})[0.5 + 1.4P/(f'_c A_g)]$$

(Equation 18-9)

where:

f_{yh} = $\leq$ 70,000 psi (483 MPa).

h_c = Cross-sectional dimension of pile core measured center to center of hoop reinforcement, inch (mm).

s = Spacing of transverse reinforcement measured along length of pile, inch (mm).

A_{sh} = Cross sectional area of tranverse reinforcement, square inches (mm^2)

The hoops and cross-ties shall be equivalent to deformed bars not less than No. 3 in size. Rectangular hoop ends shall terminate at a corner with seismic hooks.

1808.2.3.3 Allowable stresses. The maximum allowable design compressive stress, f_c , in concrete shall be determined as follows:

$f_c = 0.33f'_c - 0.27f_{pc}$ **(Equation 18-10)**

where:

f'_c = The 28-day specified compressive strength of the concrete.

f_{pc} = The effective prestress stress on the gross section.

1808.2.3.4 Installation. A prestressed pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the 28-day specified compressive strength (f'_c), but not less than the strength sufficient to withstand handling and driving forces.

1808.2.3.5 Concrete cover. Prestressing steel and pile reinforcement shall have a concrete cover of not less than $1^1/_4$ inches (32 mm) for square piles of 12 inches (305 mm) or smaller size and $1^1/_2$ inches (38 mm) for larger piles, except that for piles exposed to sea water, the minimum protective concrete cover shall not be less than $2^1/_2$ inches (64 mm).

1808.3 Structural steel piles. Structural steel piles shall conform to the requirements of Sections 1808.3.1 through 1808.3.5.

1808.3.1 Materials. Structural steel piles, steel pipe and fully welded steel piles fabricated from plates shall conform to ASTM A 36, A 252, A 283, A 572, A 588 or A 913.

1808.3.2 Allowable stresses. The allowable axial stresses shall not exceed 35 percent of the minimum specified yield strength (F_y).

Exception: Where justified in accordance with Section 1807.2.10, the allowable axial stress is permitted to be increased above $0.35F_y$, but shall not exceed $0.5F_y$.

1808.3.3 Dimensions of H-piles. Sections of H-piles shall comply with the following:

1. The flange projections shall not exceed 14 times the minimum thickness of metal in either the flange or the web and the flange widths shall not be less than 80 percent of the depth of the section.
2. The nominal depth in the direction of the web shall not be less than 8 inches (203 mm).
3. Flanges and web shall have a minimum nominal thickness of $^3/_8$ inch (9.5 mm).

1808.3.4 Dimensions of steel pipe piles. Steel pipe piles driven open ended shall have a nominal outside diameter of not less than 8 inches (203 mm). The pipe shall have a minimum of 0.34 square inch (219 mm^2) of steel in cross section to resist each 1,000 foot-pounds (1356 N·m) of pile hammer energy or the equivalent strength for steels having a yield strength greater than 35,000 psi (241 MPa). Where pipe wall thickness less than 0.188 inch (4.8 mm) is driven open ended, a suitable cutting shoe shall be provided.

1808.3.5 Design in Seismic Design Category D, E or F. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1616, I-shaped sections shall have an unsupported flange-to-thickness ratio not to exceed:

$52\sqrt{F_y}$ (For SI: $0.317\sqrt{E/F_y}$). **(Equation 18-11)**

Circular sections shall have an outside wall diameter to wall thickness ratio not to exceed:

$1{,}300\sqrt{F_y}$ (For SI: $7.63\sqrt{E/F_y}$). **(Equation 18-12)**

SECTION 1809 CAST-IN-PLACE CONCRETE PILE FOUNDATIONS

1809.1 General. The materials, reinforcement and installation of cast-in-place concrete piles shall conform to Sections 1809.1.1 through 1809.1.3.

1809.1.1 Materials. Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pile, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.

1809.1.2 Reinforcement. Except for steel dowels embedded 5 feet (1524 mm) or less in the pile and as provided in Section 1809.3.4, reinforcement where required shall be assembled and tied together and shall be placed in the pile as a unit before the reinforced portion of the pile is filled with concrete except in augered uncased cast-in-place piles. Tied reinforcement in augered uncased cast-in-place piles shall be placed after piles are concreted, while the concrete is still in a semifluid state.

1809.1.2.1 Reinforcement in Seismic Design Category C. Where a structure is assigned to Seismic Design Category C in accordance with Section 1616, the following shall apply. A minimum reinforcement ratio of 0.0025 shall be provided for uncased cast-in-place concrete drilled piles, drilled piers, or caissons in the top one-third of the pile length or a minimum length of 10 feet (3048 mm) below the ground. The minimum reinforcement ratio, but no less than that ratio required by rational analysis, shall be continued throughout the flexural length of the pile. There shall be a minimum of four bars with closed ties (or equivalent spirals) of a minimum $^1/_4$ inch (6.4 mm) diameter provided at 16-longitudinal-bar-diameter maximum spacing. A maximum spacing of 6 inches (152 mm) or 8-longitudinal-bar-diameters, whichever is less, shall be provided within a distance equal to two times the least pile dimension of the bottom of the pile cap.

1809.1.2.2 Reinforcement in Seismic Design Category D, E or F. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1616, the requirements for Seismic Design Category C given above shall be met, in addition to the following. A minimum reinforcement ratio of 0.005 shall be provided for uncased cast-in-place concrete piles, drilled piers or caissons in the top one-half of the pile length or a minimum length of 10 feet (3048 mm) below ground. The minimum reinforcement ratio, but no less than that ratio required by rational analysis, shall be continued throughout the flexural length of the pile. There shall be a minimum of four bars with closed ties or equivalent spirals provided at 6-longitudinal-bar-diameter maximum spacing with a maximum spacing of 4 inches (102 mm) within seven times the least pile dimension of the bottom of the pile cap in Site Class E or F and at the interfaces of soft to medium stiff clay or liquefiable strata, and three times the least pile dimension at all other sites. Tie spacing throughout the remainder of the concrete section shall not exceed 12-longitudinal-bar-diameters, one-half the least dimension of the section, nor 12 inches (305 mm). Ties shall be a minimum of No. 3 bars for piles with a least dimension up to 20 inches (508 mm), and No. 4 bars for larger piles.

1809.1.3 Concrete placement. Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pile, the concrete shall not be chuted directly into the pile but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pile.

1809.2 Enlarged base piles. Enlarged base piles shall conform to the requirements of Sections 1809.2.1 through 1809.2.5.

1809.2.1 Materials. The maximum size for coarse aggregate for concrete shall be $^3/_4$ inch (19.1 mm). Concrete to be compacted shall have a zero slump.

1809.2.2 Allowable stresses. The maximum allowable design compressive stress for concrete not placed in a permanent steel casing shall be 25 percent of the 28-day specified compressive strength (f'_c). Where the concrete is placed in a permanent steel casing, the maximum allowable concrete stress shall be 33 percent of the 28-day specified compressive strength (f'_c).

1809.2.3 Installation. Enlarged bases formed either by compacting concrete or driving a precast base shall be formed in or driven into granular soils. Piles shall be constructed in the same manner as successful prototype test piles driven for the project. Pile shafts extending through peat or other organic soil shall be encased in a permanent steel casing. Where a cased shaft is used, the shaft shall be adequately reinforced to resist column action or the annular space around the pile shaft shall be filled sufficiently to reestablish the lateral support of the soil. Where pile heave occurs, the pile shall be replaced unless it is demonstrated that the pile is undamaged and capable of carrying twice its design load.

1809.2.4 Load-bearing capacity. Pile load-bearing capacity shall be verified by load tests in accordance with Section 1807.2.8.3.

1809.2.5 Concrete cover. The minimum concrete cover shall be $2^1/_2$ inches (64 mm) for uncased shafts and 1 inch (25 mm) for cased shafts.

1809.3 Drilled or augered uncased piles. Drilled or augered uncased piles shall conform to Sections 1809.3.1 through 1809.3.5.

1809.3.1 Allowable stresses. The allowable design stress in the concrete of drilled uncased piles shall not exceed 33 percent of the 28-day specified compressive strength (f'_c). The allowable design stress in the concrete of augered cast-in-place piles shall not exceed 25 percent of the 28-day specified compressive strength (f'_c). The allowable compressive stress of reinforcement shall not exceed 34 percent of the yield strength of the steel or 25,500 psi (175.8 MPa).

1809.3.2 [Comm 62.1809 (1)] Dimensions. The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 12 inches (305 mm).

Exception: The length of the pile is permitted to exceed 30 times the diameter, provided that the design and installation of the pile foundation is under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pile foundations.

1809.3.3 Installation. Where pile shafts are formed through unstable soils and concrete is placed in an open-drilled hole, a steel liner shall be inserted in the hole prior to placing the concrete. Where the steel liner is withdrawn during concreting, the level of concrete shall be maintained above the bottom of the liner at a sufficient height to offset any hydrostatic or lateral soil pressure.

Where concrete is placed by pumping through a hollow-stem auger, the auger shall be permitted to rotate in a clockwise direction during withdrawal. The augur shall be withdrawn in a continuous manner in increments of about 12 inches (305 mm) each. Concreting pumping pressures shall be measured and shall be maintained high enough at all times to offset hydrostatic and lateral earth pressures. Concrete volumes shall be measured to ensure that the volume of concrete placed in each pile is equal to or greater than the theoretical volume of the hole created by the auger. Where the installation process of any pile is interrupted or a loss of concreting pressure occurs, the pile shall be redrilled to 5 feet (1524 mm) below the elevation of the tip of the auger when the installation was interrupted or concrete pressure was lost and reformed. Augered cast-in-place piles shall not be installed within 6 pile diameters center-to-center of a pile filled with concrete less than 12 hours old, unless approved by the building official. If the concrete level in any completed pile drops during installation of an adjacent pile, the pile shall be replaced.

1809.3.4 Reinforcement. For piles installed with a hollow-stem auger, where full length longitudinal steel reinforcement is placed without lateral ties, the reinforcement shall be placed through ducts in the auger prior to filling the pile with concrete. All pile reinforcement shall have a concrete cover of not less than $2^1/_2$ inches (64 mm).

Exception: Where physical constraints do not allow the placement of the longitudinal reinforcement prior to filling the pile with concrete or where partial length longitudinal reinforcement is placed without lateral ties, the reinforcement is allowed to be placed after the piles are completely concreted but while concrete is still in a semifluid state.

1809.3.5 Reinforcement in Seismic Design Category C, D, E or F. Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1616, the corresponding requirements of Sections 1809.1.2.1 and 1809.1.2.2 shall be met.

1809.4 Driven uncased piles. Driven uncased piles shall conform to Sections 1809.4.1 through 1809.4.4.

1809.4.1 Allowable stresses. The allowable design stress in the concrete shall not exceed 25 percent of the 28-day specified compressive strength (f'_c) applied to a cross-sectional area not greater than the inside area of the drive casing or mandrel.

1809.4.2 [Comm 62.1809 (2)] Dimensions. The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 12 inches (305 mm).

Exception: The length of the pile is permitted to exceed 30 times the diameter, provided that the design and installation of the pile foundation is under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pile foundations.

1809.4.3 Installation. Piles shall not be driven within 6 pile diameters center-to-center in granular soils or within one-half the pile length in cohesive soils of a pile filled with concrete less than 48 hours old unless approved by the building official. If the concrete surface in any completed pile rises or drops, the pile shall be replaced. Piles shall not be installed in soils that could cause pile heave.

1809.4.4 Concrete cover. Pile reinforcement shall have a concrete cover of not less than $2^1/_2$ inches (64 mm), measured from the inside face of the drive casing or mandrel.

1809.5 Steel-cased piles. Steel-cased piles shall comply with the requirements of Sections 1809.5.1 through 1809.5.4.

1809.5.1 Materials. Pile shells or casings shall be of steel and shall be sufficiently strong to resist collapse and sufficiently watertight to exclude any foreign materials during the placing of concrete. Steel shells shall have a sealed tip with a diameter of not less than 8 inches (203 mm).

1809.5.2 Allowable stresses. The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f'_c). The allowable concrete compressive stress shall be 0.40 (f'_c) for that portion of the pile meeting the conditions specified in Sections 1809.5.2.1 through 1809.5.2.4.

1809.5.2.1 Shell thickness. The thickness of the steel shell shall not be less than manufacturer's standard gage No. 14 gage (0.068 inch) (1.75 mm) minimum.

1809.5.2.2 Shell type. The shell shall be seamless or shall be provided with seams of strength equal to the basic material and be of a configuration that will provide confinement to the cast-in-place concrete.

1809.5.2.3 Strength. The ratio of steel yield strength (f_y) to 28-day specified compressive strength (f'_c) shall not be less than six.

1809.5.2.4 Diameter. The nominal pile diameter shall not be greater than 16 inches (406 mm).

1809.5.3 Installation. Piles shall have steel shells that are mandrel-driven their full length in contact with the surrounding soil, left permanently in place and filled with concrete.

Steel shells shall be driven in such order and with such spacing as to ensure against distortion of or injury to piles already in place. A pile shall not be driven within $4^1/_2$ average pile diameters of a pile filled with concrete less than 24 hours old unless approved by the building official. Concrete shall not be placed in steel shells within heave range of driving.

1809.5.4 Reinforcement. Reinforcement shall not be placed within 1 inch (25 mm) of the steel shell. Reinforcing shall be required for unsupported pile lengths or where the pile is designed to resist uplift or unbalanced lateral loads.

1809.5.4.1 Seismic reinforcement. Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1616, the reinforcement requirements for drilled or augered uncased piles in Section 1809.3.5 shall be met.

Exception: A spiral-welded metalcasing of a thickness not less than manufacturer's standard gage No. 14 gage (0.068 inch) is permitted to provide concrete confinement in lieu of the closed ties or equivalent spirals required in an uncased concrete pile. Where used as such, the metal casing shall be protected against possible deleterious action due to soil constituents, changing water levels, or other factors indicated by boring records of site conditions.

1809.6 Concrete-filled steel pipe and tube piles. Concrete-filled steel pipe and tube piles shall conform to the requirements of Sections 1809.6.1 through 1809.6.5.

1809.6.1 Materials. Steel pipe and tube sections used for piles shall conform to ASTM A 252 or A 283. Concrete shall conform to Section 1809.1.1. The maximum coarse aggregate size shall be $^3/_4$ inch (19.1 mm).

1809.6.2 Allowable stresses. The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f'_c). The allowable design compressive stress in the steel shall not exceed 35 percent of the minimum specified yield strength of the steel (F_y), provided F_y shall not be assumed greater than 36,000 psi (248 MPa) for computational purposes.

Exception: Where justified in accordance with Section 1807.2.10, the allowable stresses are permitted to be increased to 0.50 F_y.

1809.6.3 Minimum dimensions. Piles shall have a nominal outside diameter of not less than 8 inches (203 mm) and a minimum wall thickness in accordance with Section 1808.3.4. For mandrel-driven pipe piles, the minimum wall thickness shall be $^1/_{10}$ inch (2.5 mm).

1809.6.4 Reinforcement. Reinforcement steel shall conform to Section 1809.1.2. Reinforcement shall not be placed within 1 inch (25 mm) of the steel casing.

1809.6.4.1 Seismic reinforcement. Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1616, the following shall apply. Minimum reinforcement no less than 0.01 times the cross-sectional area of the pile concrete shall be provided in the top of the pile with a length equal to two times the required cap embedment anchorage into the pile cap, but not less than the tension development length of the reinforcement. The wall thickness of the steel pipe shall not be less than $^3/_{16}$ inch (5 mm).

1809.6.5 Placing concrete. The placement of concrete shall conform to Section 1809.1.3.

1809.7 Caisson piles. Caisson piles shall conform to the requirements of Sections 1809.7.1 through 1809.7.6.

1809.7.1 Construction. Caisson piles shall consist of a shaft section of concrete-filled pipe extending to bedrock with an uncased socket drilled into the bedrock and filled with concrete. The caisson pile shall have a full-length structural steel core or a stub core installed in the rock socket and extending into the pipe portion a distance equal to the socket depth.

1809.7.2 Materials. Pipe and steel cores shall conform to the material requirements in Section 1808.3. Pipes shall have a minimum wall thickness of $^3/_8$ inch (9.5 mm) and shall be fitted with a suitable steel-driving shoe welded to the bottom of the pipe. Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 4,000 psi (27.58 MPa). The concrete mix shall be designed and proportioned so as to produce a cohesive workable mix with a slump of 4 inches (102 mm) to 6 inches (152 mm).

1809.7.3 Design. The depth of the rock socket shall be sufficient to develop the full load-bearing capacity of the caisson pile with a minimum safety factor of two, but the depth shall not be less than the outside diameter of the pipe. The design of the rock socket is permitted to be predicated on the sum of the allowable load-bearing pressure on the bottom of the socket plus bond along the sides of the socket. The minimum outside diameter of the caisson pile shall be 18 inches (457 mm), and the diameter of the rock socket shall be approximately equal to the inside diameter of the pile.

1809.7.4 Structural core. The gross cross-sectional area of the structural steel core shall not exceed 25 percent of the gross area of the caisson. The minimum clearance between the structural core and the pipe shall be 2 inches (51 mm). Where cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.

1809.7.5 Allowable stresses. The allowable design compressive stresses shall not exceed the following: concrete, $0.33f'_c$; steel pipe, $0.35\ F_y$; and structural steel core, $0.50\ F_y$.

1809.7.6 Installation. The rock socket and pile shall be thoroughly cleaned of foreign materials before filling with concrete. Steel cores shall be bedded in cement grout at the base of the rock socket. Concrete shall not be placed through water except where a tremie or other approved method is used.

SECTION 1810
COMPOSITE PILES

1810.1 General. Composite piles shall conform to the requirements of Sections 1810.2 through 1810.5.

1810.2 Design. Composite piles consisting of two or more approved pile types shall be designed to meet the conditions of installation.

1810.3 Limitation of load. The maximum allowable load shall be limited by the capacity of the weakest section incorporated in the pile.

1810.4 Splices. Splices between concrete and steel or wood sections shall be designed to prevent separation both before and after the concrete portion has set, and to ensure the alignment and transmission of the total pile load. Splices shall be designed to resist uplift caused by upheaval during driving of adjacent piles, and shall develop the full compressive strength and not less than 50 percent of the tension and bending strength of the weaker section.

1810.5 Seismic reinforcement. Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1616, the following shall apply. Where concrete and steel are used as part of the pile assembly, the concrete reinforcement shall comply with that given in Sections 1809.1.2.1 and 1809.1.2.2 or the steel section shall comply with Section 1809.6.4.1 or 1808.3.5.

SECTION 1811
PIER FOUNDATIONS

1811.1 General. Isolated and multiple piers used as foundations shall conform to the requirements of Sections 1811.2 through 1811.10, as well as the applicable provisions of Section 1807.2.

1811.2 Lateral dimensions and height. The minimum dimension of isolated piers used as foundations shall be 2 feet (610 mm), and the height shall not exceed 12 times the least horizontal dimension.

Exceptions:

1. The height limitations shall not apply to buildings of Group R-3 and Group U occupancies not exceeding two stories of light-frame construction, subject to the approval of the building official.
2. The lateral dimension and height limitations shall not apply where the piers are constructed of reinforced concrete or structural steel, or are entirely encased in a steel shell at least $^1/_4$ inch (6.4 mm) thick.
3. Where the surrounding foundation materials furnish adequate lateral support, heights greater than herein specified are permitted, subject to the approval of the building official.

1811.3 Materials. Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pier, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.

1811.4 Reinforcement. Except for steel dowels embedded 5 feet (1524 mm) or less in the pier, reinforcement where required shall be assembled and tied together and shall be placed in the pier hole as a unit before the reinforced portion of the pier is filled with concrete.

Exception: Reinforcement is permitted to be wet set and the $2^1/_2$ inch (64 mm) concrete cover requirement be reduced to 2 inches (51 mm) for Group R-3 and Group U occupancies not exceeding two stories of light-frame construction, provided the construction method can be demonstrated to the satisfaction of the building official.

Reinforcement shall conform to the requirements of Sections 1809.1.2.1 and 1809.1.2.2

Exceptions:

1. Isolated piers supporting posts of Group R-3 and Group U occupancies not exceeding two stories of light-frame construction is permitted to be reinforced as required by rational analysis but not less than a minimum of one No. 4 bar, without ties or spirals, when detailed so the pier is not subject to lateral loads and the soil is determined to be of adequate stiffness.
2. Isolated piers supporting posts and bracing from decks and patios appurtenant to Group R-3 and Group U occupancies not exceeding two stories of light-frame construction may be reinforced as required by rational analysis but not less than one No. 4 bar, without ties or spirals, when the lateral load, E, to the top of the pier does not exceed 200 pounds (890 N) and the soil is determined to be of adequate stiffness.
3. Piers supporting the concrete foundation wall of Group R-3 and Group U occupancies not exceeding two stories of light-frame construction is permitted to be reinforced as required by rational analysis but not less than two No. 4 bars, without ties or spirals, when it can be shown the concrete pier will not rupture when designed for the maximum seismic load, E_m, and the soil is determined to be of adequate stiffness.
4. Closed ties or spirals where required by Section 1809.1.2.2, are permitted to be limited to the top 3 feet (914 mm) of the piers 10 feet (3048 mm) or less in depth supporting Group R-3 and Group U occupancies of Seismic Design Category D, not exceeding two stories of light-frame construction.

1811.5 Concrete placement. Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pier, the concrete shall not be chuted directly into the pier but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pier.

1811.6 Belled bottoms. Where pier foundations are belled at the bottom, the edge thickness of the bell shall not be less than that required for the edge of footings. Where the sides of the bell slope at an angle less than 60 degrees (1 rad) from the horizontal, the effects of vertical shear shall be considered.

1811.7 Masonry. Where the unsupported height of foundation piers exceeds six times the least dimension, the allowable working stress on piers of unit masonry shall be reduced in accordance with ACI 530/ASCE 5/TMS 402.

1811.8 Concrete. Where adequate lateral support is not provided, and the unsupported height to least lateral dimension does not exceed three, piers of plain concrete shall be designed and constructed as pilasters in accordance with ACI 318. Where the unsupported height to least lateral dimension exceeds three, piers shall be constructed of reinforced concrete, and shall conform to the requirements for columns in ACI 318.

Exception: Where adequate lateral support is furnished by the surrounding materials as defined in Section 1807.2.9, piers are permitted to be constructed of plain or reinforced concrete. The requirements of ACI 318 for bearing on concrete shall apply.

1811.9 Steel shell. Where concrete piers are entirely encased with a circular steel shell, and the area of the shell steel is considered reinforcing steel, the steel shall be protected under the conditions specified in Section 1807.2.17. Horizontal joints in the shell shall be spliced to comply with Section 1807.2.7.

1811.10 Dewatering. Where piers are carried to depths below water level, the piers shall be constructed by a method that will ensure accurate preparation and inspection of the bottom, and the depositing or construction of sound concrete or other masonry in the dry.