‡‡‡ Chapter 16 was amended by Local Law 77 of 2023. This law has an effective date of June 10, 2023.

CHAPTER 16

STRUCTURAL DESIGN

SECTION BC 1601
GENERAL

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

1601.1.1 Referenced standards. Where this code makes reference to the nationally recognized standard ASCE 7, such standard shall be as modified for New York City in accordance with this chapter.

1601.2 Special provisions for prior code buildings. The provisions of Sections 1601.2.1 through 1601.2.4 shall apply to structural work on prior code buildings.

1601.2.1 Use of this code. Notwithstanding the applicant's election to use the 1968 Building Code or prior code, the structural calculations shall be permitted to be performed in accordance with this code provided that the structural safety of the prior code building is not reduced. Notwithstanding the provisions of Section 28-101.4.4 of the *Administrative Code*, the use of Load and Resistance Factor Design (LRFD) engineering calculations shall not be deemed to reduce structural safety provided the properties of the existing materials are determined using accepted engineering principles.

1601.2.2 Live loads. Loads indicated in the applicable prior code shall be permitted for structural calculations using engineering formulas from this code provided that the structural safety of the prior code building is not reduced.

1601.2.3 Seismic loads. The determination as to whether seismic requirements apply to an alteration shall be made in accordance with the 1968 Building Code and interpretations by the department relating to such determinations. Any applicable seismic loads and requirements, including for the bracing of architectural, mechanical, plumbing, fuel gas, fire suppression and electrical systems and equipment, shall be permitted to be determined in accordance with this chapter or the 1968 Building Code and reference standard RS 9-6 of such code.

1601.2.4 Wind loads. All alterations, minor alterations, and ordinary repairs, to the extent of such work, shall be permitted to be performed in accordance with the wind load requirements set forth in the 1968 Building Code, or where the 1968 Building Code so authorizes, the code in effect prior to December 6, 1968.

Exceptions:

1. Equipment, appliances and supports that are exposed to wind shall be designed and installed to resist the wind pressures determined in accordance with Section 1609.
2. Wind loads on glass shall not be permitted to be calculated in accordance with the code in effect prior to December 6, 1968.
3. When the wind surface area of a prior code building or structure is increased by more than 5 percent in any direction or there is a permanent decrease of the lateral force capacity by more than 20 percent in any direction, the entire building or structure shall be designed to resist the design wind load as calculated pursuant to the applicable code, but not less than 5 psf (0.24k N/m^2).

SECTION BC 1602
DEFINITIONS AND NOTATIONS

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

ALLOWABLE STRESS DESIGN.

BALCONY, EXTERIOR.

DEAD LOAD.

DECK.

DESIGN STRENGTH.

DIAPHRAGM.

Diaphragm, blocked.

Diaphragm boundary.

Diaphragm chord.

ESSENTIAL FACILITIES.

FABRIC PARTITION.

FACTORED LOAD.

HELIPAD.

ICE-SENSITIVE STRUCTURE.

IMPACT LOAD.

LIMIT STATE.

LIVE LOAD.

LIVE LOAD (ROOF).

LOAD AND RESISTANCE FACTOR DESIGN (LRFD).

LOAD EFFECTS.

LOAD FACTOR.

LOADS.

NOMINAL LOADS.

OTHER STRUCTURES.

PANEL (PART OF A STRUCTURE).

RESISTANCE FACTOR.

RISK CATEGORY.

STRENGTH, NOMINAL.

STRENGTH, REQUIRED.

STRENGTH DESIGN.

SUSCEPTIBLE BAY.

VEHICLE BARRIER.

NOTATIONS.

D = **Dead load.**

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

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

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

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

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

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

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

R = Rain load.

S = Snow load.

T = Cumulative effects of self-straining load forces and effects.

V_{asd} =Allowable stress design wind speed, miles per hour (mph) (km/hr) where applicable.

V = Basic design wind speeds, miles per hour (mph) (km/hr) determined from Table 1609.3 of this code.

W = Load due to wind pressure.

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

SECTION BC 1603
CONSTRUCTION DOCUMENTS

1603.1 General. Construction documents shall include drawings that show the sizes, sections 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 1603.1.10 shall be clearly indicated on such drawings of parts of the building or structure.

Exception: In lieu of the requirements of Sections 1603.1.1 through 1603.1.10, construction documents for buildings constructed in accordance with the conventional light-frame construction provisions of Section 2308 shall include drawings that indicate the following structural design information:

1. Floor and roof dead and live loads.
2. Ground snow load, P_g.
3. Basic design wind speed, V, miles per hour (mph) (km/hr) and allowable stress design wind speed, V_{asd}, as determined in accordance with Section 1609.3.1 and wind exposure.
4. Seismic design category and site class.
5. Flood design data, if located in flood hazard areas.
6. Design load-bearing values of soils or rock under shallow foundations and/or the design load capacity of deep foundations.

1603.1.1 Floor live load. The uniformly distributed, concentrated and impact floor live load used in the design shall be indicated for floor areas. Use of live load reduction in accordance with Section 1607.11 shall be indicated for each type of live load used in the design.

1603.1.2 Partition loads. The equivalent uniform partition loads or, in lieu of these, a statement to the effect that the design was predicated on actual partition loads.

1603.1.3 Roof live load. The roof live load used in the design shall be indicated for roof areas (Section 1607.13).

1603.1.4 Roof snow load data. The ground snow load, P_g, shall be indicated. 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_s.
4. Thermal factor, C_t.
5. Slope factor(s), C_s.
6. Drift surcharge load(s), P_d, where the sum of P_d and P_f exceeds 20 psf (0.96 kN/m^2).
7. Width of snow drift(s), w.

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

1. Basic design wind speed, V, miles per hour (km/hr) and/or allowable stress design wind speed, V_{asd}, as determined in accordance with Section 1609.3 and 1609.3.1.
2. Risk category.
3. Wind exposure. Applicable wind direction if more than one wind exposure is utilized.
4. Applicable internal pressure coefficient.
5. Design wind pressures to be used for exterior component and cladding materials not specifically designed by the registered design professional responsible for the design of the structure, psf (kN/m^2).
6. Design base shear.

1603.1.6 Earthquake design data. The following information related to seismic loads shall be shown, regardless of whether seismic loads govern the design of the lateral force-resisting system of the structure:

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

1603.1.7 Geotechnical information. The design load-bearing values of soils or rock under shallow foundations and/or the design load capacity of deep foundations shall be shown on the construction documents.

1603.1.8 Flood hazard areas. Construction documents for buildings and other structures located in flood hazard areas shall comply with provisions of Section G104.2.1.

1603.1.9 Special loads. Special loads that are applicable to the design of the building, structure or portions thereof, including but not limited to the loads of machinery or equipment, and that are greater than the specified floor and roof loads shall be specified by their descriptions and locations.

1603.1.9.1 Photovoltaic panel systems. The dead load of rooftop-mounted photovoltaic panel systems, including rack support systems, shall be indicated on the construction documents.

1603.1.10 Superimposed dead loads. The uniformly distributed superimposed dead loads used in the design shall be indicated for floor and roof areas.

SECTION BC 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 and referenced standards.

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 commissioner.

1604.3 Serviceability. Structural systems and members thereof shall be designed to have adequate stiffness to limit deflections as indicated in Table 1604.3 of this code. Drift limits applicable to earthquake loading shall be in accordance with ASCE 7 Chapter 12, 13, 15 or 16, as applicable.

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

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

For SI: 1 foot = 304.8 mm.

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

b. 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 1607.15.

c. See Section 2403 for glass supports.

d. The deflection limit for the D+(L+L_r) load combination only applies to the deflection due to the creep component of long-term dead load deflection plus the short-term live load deflection. For lumber, structural glued laminated timber, prefabricated wood I-joists and structural composite lumber members that are dry at time of installation and used under dry conditions in accordance with the ANSI/AWC NDS, the creep component of the long-term deflection shall be permitted to be estimated as the immediate dead load deflection resulting from 0.5D. For lumber and glued laminated timber members installed or used at all other moisture conditions or cross laminated timber and wood structural panels that are dry at time of installation and used under dry conditions in accordance with ANSI/AWC NDS, the creep component of the long-term deflection is permitted to be estimated as the immediate dead load deflection resulting from D. The value of 0.5D shall not be used in combination with ANSI/AWC NDS provisions for long-term loading.

e. The preceding deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to ensure adequate drainage shall be investigated for ponding. See Chapter 8 of ASCE 7.

f. The wind load shall be permitted to be taken as 0.42 times the "component and cladding" loads or directly calculated using the 10-year mean return interval wind speed for the purpose of determining deflection limits in Table 1604.3. Where framing members support glass, the deflection limit therein shall not exceed that specified in Section 1604.3.7.

g. For steel structural members, the deflection due to creep component of long-term dead load shall be permitted to be taken as zero.

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

i. l = Length of the member between supports.‡ For cantilever members, l shall be taken as twice the length of the cantilever.

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 360, AISI S100, ASCE 8, SJI 100 or SJI 200, as applicable.

1604.3.4 Masonry. The deflection of masonry structural members shall not exceed that permitted by TMS 402, as modified by Chapter 21 of this code.

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

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

1604.3.7 Framing supporting glass. The deflection of framing members supporting glass subjected to 0.6 times the "component and cladding" wind loads shall not exceed either of the following:

1. $^{1}/_{175}$ of the length of span of the framing member, for framing members having a length not more than 13 feet 6 inches (4115 mm).
2. $^{1}/_{240}$ of the length of span of the framing member + $^{1}/_{4}$ inch (6.4 mm), for framing members having a length greater than 13 feet 6 inches (4114.8 mm).

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 effects of added deformations expected to occur during their service life. Secondary stresses in trusses shall be considered and, where of significant magnitude, their effects shall be provided for in the design.

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

The total lateral force shall be distributed to the various vertical elements of the lateral force-resisting system in proportion to their rigidities, considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements assumed not to be a part of the lateral force-resisting system are permitted to be incorporated into buildings provided that their effect on the action of the system is considered and provided for in the design. A diaphragm is rigid for the purpose of distribution of story shear and torsional moment when the lateral deformation of the diaphragm is less than or equal to two times the average story drift. Where required by ASCE 7, 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 effects caused by the forces specified in this chapter including overturning, uplift and sliding. Where sliding is used to isolate the elements, the effects of friction between sliding elements shall be included as a force.

1604.5 Risk category. Each building and structure shall be assigned a risk category in accordance with Table 1604.5 of this code. Where a referenced standard specifies an occupancy category, the risk category shall not be taken as lower than the occupancy category specified therein. Where a referenced standard specifies that the assignment of a risk category be in accordance with ASCE 7, Table 1.5-1, Table 1604.5 of this code shall be used in lieu of ASCE 7, Table 1.5-1.

TABLE 1604.5
RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

RISK CATEGORY	NATURE OF OCCUPANCY
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: 1. Agricultural facilities. 2. Certain temporary facilities. 3. Minor storage facilities.
II	Buildings and other structures except those listed in Risk Categories I, III and IV.
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: 1. Buildings and other structures whose primary occupancy is Places of Assembly with an occupant load greater than 300. 2. Buildings and other structures containing a Group E occupancy with an occupant load greater than 250. 3. Buildings and other structures containing educational occupancies for students above the 12th grade with an occupant load greater than 500. 4. Group I-2 occupancies with an occupant load of 50 or more resident care recipients but not having surgery or emergency treatment facilities. 5. Group I-3 occupancies. 6. Any other occupancy with an occupant load greater than 5,000[a]. 7. Power-generating stations, water treatment facilities for potable water, waste water treatment facilities and other public utility facilities not included in Risk Category IV. 8. Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive materials that: Exceed maximum allowable quantities per control area as given in Table 307.1(1) or 307.1(2), or per outdoor control area in accordance with the *New York City Fire Code*; and Are sufficient to pose a threat to the public if released.[b]
IV	Buildings and other structures designed as essential facilities, including but not limited to: 1. Group I-2 occupancies having surgery or emergency treatment facilities. 2. Fire, rescue, ambulance and police stations and emergency vehicle garages. 3. Designated earthquake, hurricane or other emergency shelters. 4. Designated emergency preparedness, communications and operations centers and other facilities required for emergency response. 5. Power-generating stations and other public utility facilities required as emergency backup facilities for Risk Category IV structures. 6. Buildings and other structures containing quantities of highly toxic materials that: Exceed maximum allowable quantities per control area as given in Table 307.1(2) or per outdoor control area in accordance with the *New York City Fire Code*; and Are sufficient to pose a threat to the public if released[b] 7. Aviation control towers, air traffic control centers and emergency aircraft hangars. 8. Buildings and other structures having critical national defense functions. 9. Water storage facilities and pump structures required to maintain water pressure for fire suppression.

a. For purposes of occupant load calculation, occupancies required by Table 1004.1.3 to use gross floor area calculations shall be permitted to use net floor areas to determine the total occupant load.

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

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

Exception: Where a storm shelter designed and constructed in accordance with ICC 500 is provided in a building, structure or portion thereof normally occupied for other purposes, the risk category for the normal occupancy of the building shall apply unless the storm shelter is a designated emergency shelter in accordance with Table 1604.5 of this code.

1604.5.2 Importance factors. Importance factors for snow, ice, wind and seismic loads shall be determined in accordance with ASCE 7, Table 1.5-2 based on the risk category assigned in accordance with Table 1604.5 of this code.

1604.6 In-situ load tests. The commissioner is authorized to require an engineering analysis or a load test, or both, of any construction whenever there is reason to question the safety of the construction for the intended occupancy. Engineering analysis and load tests shall be conducted in accordance with Section 1708.

1604.7 Preconstruction load tests. Materials and methods of construction that are not capable of being designed by approved engineering analysis or that do not comply with the applicable referenced standards, or alternative test procedures in accordance with Section 1709, shall be load tested in accordance with Section 1710.

1604.8 Anchorage. Buildings and other structures, and portions thereof, shall be provided with anchorage in accordance with Sections 1604.8.1 through 1604.8.4, as applicable.

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 and load combinations.

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

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

1604.8.4 Non-structural walls and partitions. Walls not meeting the requirements of Section 1604.8.2 and partitions shall be laterally supported with sufficient mechanical anchorage to resist the loads imposed per this code, including but not limited to live loads, wind loads, and earthquake loads.

1604.9 Wind and seismic detailing. Lateral force-resisting systems shall meet seismic detailing requirements and limitations prescribed in this code and ASCE 7 Chapters 11, 12, 13, 15, 17 and 18 of this code as applicable, even when wind load effects are greater than seismic load effects. Where ASCE 7 references Chapter 14 of ASCE 7 those references shall not apply, except as specifically required in this code.

1604.10 Loads on storm shelters. Loads and load combinations on storm shelters shall be determined in accordance with ICC 500.

SECTION BC 1605
LOAD COMBINATIONS

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

1. The load combinations specified in Section 1605.2 or 1605.3;
2. The load combinations specified in Chapters 18 through 23; and
3. The seismic load effects including overstrength factor in accordance with Sections 2.3.6 and 2.4.5 of ASCE 7 where required by Chapters 12, 13, and 15 of ASCE 7. With the simplified procedure of Section 12.14 of ASCE 7, the seismic load effects including overstrength factor in accordance with Section 12.14.3.2 and Chapter 2 of ASCE 7 shall be used.

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

Where the load combinations with overstrength factor in Sections 2.3.6 and 2.4.5 of ASCE 7 apply, they shall be used as follows:

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

1605.1.1 Stability. Regardless of which load combinations are used to design for strength, where overall structure stability (such as stability against overturning, sliding, or buoyancy) is being verified, use of the load combinations specified in Section 1605.2 or 1605.3 shall be permitted. Where the load combinations specified in Section 1605.2 are used, strength reduction factors applicable to soil resistance shall be provided by a registered design professional. The stability of retaining walls shall be verified in accordance with Section 1807.2.

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

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

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

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

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

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

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

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

where:

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

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

Exceptions:

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

‡‡‡**1605.2.1 Other loads.** Where a structure is located in a V zone or Coastal A zone and F_a is to be considered in design, in addition to the load combinations of Equations 16-1 through 16-7, the structure and portions thereof shall resist the most critical effects of the load combinations of Equations 16-8 and 16-10. Where a structure is located in an A zone and F_a is to be considered in design, in addition to the load combinations of Equations 16-1 through 16-7, structures and portions thereof shall resist the most critical effects of the load combinations of Equation 16-9 and 16-11. Where self-straining loads, T, are considered in design, their structural effects in combination with other loads shall be determined in accordance with Section 2.3.4 of ASCE 7. Where ice loads are to be considered in design, the load combinations of Section 2.3.3 of ASCE 7 shall be used. Refer to the following sections for other load combinations:

Flood Load Combinations:

$1.2(D + F) + 1.0W + 2.0F_a + f_1L + 1.6H + 0.5(L_r \text{ or } S \text{ or } R)$ **(Equation 16-8)**

$1.2(D + F) + 0.5W + 1.0F_a + f_1L + 1.6H + 0.5(L_r \text{ or } S \text{ or } R)$ **(Equation 16-9)**

$0.9D + 1.0W + 2.0F_a + 1.6H$ **(Equation 16-10)**

$0.9D + 0.5W + 1.0F_a + 1.6H$ **(Equation 16-11)**

1605.3 Load combinations using allowable stress design. Load combinations for allowable stress design shall be in accordance with Section 1605.3.1 or 1605.4.

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

$D + F$ **(Equation 16-12)**

$D + H + F + L$ **(Equation 16-13)**

$D + H + F + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-14)**

$D + H + F + 0.75(L) + 0.75(L_r \text{ or } S \text{ or } R)$ **(Equation 16-15)**

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

$D + H + F + 0.75(0.6W) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$ **(Equation 16-17)**

$D + H + F + 0.75(0.7E) + 0.75L + 0.75S$ **(Equation 16-18)**

$0.6D + 0.6W + H$ **(Equation 16-19)**

$0.6(D + F) + 0.7E + H$ **(Equation 16-20)**

Exceptions:

1. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads of 30 psf (1.44 kN/m^2) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.
3. Where the effect of H resists the primary variable load effect, a load factor of 0.6 shall be included with H where H is permanent and H shall be set to zero for all other conditions.
4. In Equation 16-19, the wind load, W, is permitted to be reduced in accordance with Exception 2 of Section 2.4.1 of ASCE 7.
5. In Equation 16-20, 0.6 D is permitted to be increased to 0.9 D for the design of special reinforced masonry shear walls complying with Chapter 21.

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

‡‡‡ **1605.3.1.2 Other loads.** Where a structure is located in a V zone or Coastal A zone and F_a is to be considered in design, in addition to load combinations of Equations 16-12 through 16-20, structures and portions thereof shall resist the most critical effects of load combinations of Equations 16-21, 16-23 and 16-25. Where a structure is located in an A zone and F_a is to be considered in design, in addition to load combinations of Equations 16-12 through 16-20, structures and portions thereof shall resist the most critical effects of load combinations of Equations 16-22, 16-24 and 16-26. Where self-straining loads, T, are considered in design, their structural effects in combination with other loads shall be determined in accordance with Section 2.4.4 of ASCE 7. Where ice loads are to be considered in design, the load combinations of Section 2.4.3 of ASCE 7 shall be used.

Flood Load Combinations:

$D + H + F + 0.6W + 1.5F_a$ **(Equation 16-21)**

$D + H + F + 0.6W + 0.75F_a$ **(Equation 16-22)**

$D + H + F + 0.75(0.6W) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R) + 1.5F_a$ **(Equation 16-23)**

$D + H + F + 0.75(0.6W) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R) + 0.75F_a$ **(Equation 16-24)**

$0.6D + 0.6W + H + 1.5F_a$ **(Equation 16-25)**

$0.6D + 0.6W + H + 0.75F_a$ **(Equation 16-26)**

1605.4 Structural integrity load combinations—alternate load path method. Where specifically required by Sections 1615 through 1617, elements and components shall be designed to resist the forces calculated using the following combination of factored loads:

$D + f_1L + f_2W$ **(Equation 16-27)**

where:

f_1 = 0.25 for buildings in Risk Category II.

f_1 = 0.5 for buildings in Risk Category III or IV.

f_2 = 0 for buildings in Risk Category II.

f_2 = 0.33 for buildings in Risk Category III or IV.

The live load component f_1L need not be greater than the reduced live load.

1605.5 Structural integrity load combinations—vehicular impact and gas explosions. Where specifically required by Sections 1616.4 and 1616.5, elements and components shall be designed to resist the forces calculated using the following combination of factored loads:

$1.2D + A_k + (0.5L \text{ or } 0.2S)$ **(Equation 16-28)**

$0.9D + A_k + 0.2W$ **(Equation 16-29)**

Where A_k is the load effect of the vehicular impact or gas explosion.

1605.6 Structural integrity load combinations—specific local resistance method. Where the specific local resistance method is used in a key element analysis, the specified local loads shall be used as specified in Section 1617.6.

SECTION BC 1606
DEAD LOADS

1606.1 General. Dead loads are those loads defined in Chapter 2 of this code. Dead loads shall be considered permanent loads.

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

SECTION BC 1607
LIVE LOADS

1607.1 General. Live loads are those loads defined in Chapter 2 of this code.

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
1. Apartments (see residential)	—	—
2. Access floor systems Office use Computer use	 50 100	 2,000 2,000
3. Armories and drill rooms	150[n]	—
4. Assembly areas Fixed seats (fastened to floor) Follow spot, projections and control rooms Lobbies Movable seats Private assembly spaces, including conference rooms Stage floors Platforms (assembly) Other assembly spaces	 60[m] 50 100[m] 100[m] 50 150[m] 100[m] 100[m]	—
5. Balconies and Decks[h]	1.5 times the live load for the occupancy served. Not required to exceed 100 psf	—
6. Catwalks	40	300
7. Cornices	60	—
8. Corridors First floor Other floors	 100 Same as occupancy served except as indicated	—
9. Dining rooms and restaurants	100[m]	—
10. Dwellings (see residential)	—	—
11. Elevator machine room and control room grating (on area of 2 inches by 2 inches)	—	300
12. Equipment rooms, including pump rooms, generator rooms, transformer vaults, and areas for switch gear, ventilating, air conditioning, and similar electrical and mechanical equipment	75	—
13. Finish light floor plate construction (on area of 1 inch by 1 inch)	—	200
14. Fire escapes (exterior) On single-family dwellings only	100 40	—
15. Garages (passenger vehicles only)	40[o]	Note a
Trucks and buses	See Section 1607.7	
16. Handrails, guards and grab bars	See Section 1607.8	

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
17. Helipads	See Section 1607.6	
18. Hospitals		
Corridors above first floor	80	1,000
Operating rooms, laboratories	60	1,000
Patient rooms	40	1,000
19. Hotels (see residential)	—	—
20. Libraries		
Corridors above first floor	80	1,000
Reading rooms	60	1,000
Stack rooms	150[b, n]	1,000
21. Manufacturing		
Heavy	250[n]	3,000
Light	125[n]	2,000
22. Marquees, except one- and two-family dwellings	75	—
23. Office buildings		
Corridors above first floor	80	2,000
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
24. Penal institutions		
Cell blocks	40	—
Corridors	100	—
25. Recreational uses:		—
Bowling alleys, poolrooms and similar uses	75[m]	
Dance halls and ballrooms	100[m]	
Gymnasiums	100[m]	
Ice skating rink	250[n]	
Reviewing stands, grandstands and bleachers	100[c, m]	
Roller skating rink	100[m]	
Stadiums and arenas with fixed seats (fastened to floor)	60[c, m]	
26. Residential		—
One- and two-family dwellings		
Uninhabitable attics without storage[i]	10	
Uninhabitable attics with storage[i, j, k]	20	
Habitable attics and sleeping areas[k]	30	
Canopies, including marquees	20	
All other areas	40	
Hotels and multifamily dwellings		
Private rooms and corridors serving them	40	
Public rooms[m] and corridors serving them	100	

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
27. Roofs		
All roof surfaces subject to maintenance workers		300
Awnings and canopies:		
Fabric construction supported by a skeleton structure	5[m]	
All other construction, except one- and two-family dwellings	20	
Ordinary flat, pitched, and curved roofs (that are not occupiable)	20	
Primary roof members exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages		2,000
All other primary roof members		300
Occupiable roofs:		
Roof gardens	100	
Assembly areas	100[m]	
All other similar areas	Note l	Note l
28. Schools		
Classrooms	40	1,000
Corridors above the first floor	80	1,000
First-floor corridors	100	1,000
29. Scuttles, skylight ribs and accessible ceilings	—	200
30. Sidewalks, vehicular driveways and yards, subject to trucking	300[d, m]	8,000[e] or 20,000[d]
31. Stairs and exits		
One- and two-family dwellings	40	300[f]
All other	100	300[f]
32. Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		—
Heavy	250[n]	
Light	125[n]	
33. Stores		
Retail		
First floor	100	1,000
Upper floors	75	1,000
Wholesale, all floors	125[n]	1,000
34. Vehicle barriers	See Section 1607.9	
35. Walkways and elevated platforms (other than exitways)	60	—
36. Yards and terraces, pedestrians	100[m]	—

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

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of this Table or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of $4^1/_2$ inches by $4^1/_2$ inches; (2) for mechanical parking structures without slab or deck which are used for storing passenger vehicles only, 2,250 pounds per wheel.
b. The loading applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:
   1. The nominal book stack unit height shall not exceed 90 inches;
   2. The nominal shelf depth shall not exceed 12 inches for each face; and
   3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches wide.
c. Design in accordance with Section 1029.1.
d. The concentrated wheel load of 20,000 pounds shall be applied on a 20 inch by 10 inch area.
e. The concentrated wheel load shall be applied on an area of 4.5 inches by 4.5 inches.
f. The minimum concentrated load on stair treads shall be applied on an area of 2 inches by 2 inches. This load need not be assumed to act concurrently with the uniform load.
g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the department (see Section 1608).
h. See Section 1604.8.3 for decks attached to exterior walls.
i. Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirements.
j. Uninhabitable attics with storage are those where the maximum clear height between the joists and rafters is 42 inches or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses.
   The live load need only be applied to those portions of the joists of truss bottom chords where both of the following conditions are met:
   i. The attic area is accessible from an opening not less than 20 inches in width by 30 inches in length that is located where the clear height in the attic is a minimum of 30 inches; and
   ii. The slopes of the joists or truss bottom chords are no greater than two units vertical in 12 units horizontal.
   The remaining portions of the joists or truss bottom chords shall be designed for a uniformly distributed concurrent live load of not less than 10 pounds per square foot.
k. Attic spaces served by stairways other than the pull-down type shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.
l. Areas of occupiable roofs, other than roof gardens and assembly areas, shall be designed for appropriate loads as approved by the department. Unoccupied landscaped areas of roofs shall be designed in accordance with Section 1607.13.3.
m. Live load reduction is not permitted.
n. Live Load reduction is only permitted in accordance with Section 1607.11.1.2 or Item 1 of Section 1607.11.2.
o. Live Load reduction is only permitted in accordance with Section 1607.11.1.3 or Item 2 of Section 1607.11.2.

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 commissioner.

1607.2.1 Stage areas using scenery or scenic elements. Scenery battens and suspension systems shall be designed for a load of 30 pounds per linear foot (437.7 N/m) of batten length. Loft block and head block beams shall be designed to support vertical and horizontal loads corresponding to a 4-inch (101.6 mm) spacing of battens for the entire depth of the gridiron. Direction and magnitude of total forces shall be determined from the geometry of the rigging system including load concentrations from spot line rigging. Locking rails shall be designed for a uniform uplift of 500 psf (3447 kN/m^2) with a 1,000 pound (454 kg) concentration. Impact factor for batten design shall be 75 percent and for loft and head block beams shall be 25 percent. A plan drawn to a scale not less than $^1/_4$ inch (6.4 mm) equals 1 foot (304.8 mm) shall be displayed in the stage area indicating the framing plan of the rigging loft and the design loads for all members used to support scenery or rigging. Gridirons over stages shall be designed to support a uniformly distributed live load of 50 psf (2.40 kN/m^2) in addition to the rigging loads indicated.

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

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

1607.5 Partition loads. In office buildings and in other buildings where partition locations are subject to change, provisions for partition weight shall be made, whether or not partitions are shown on the construction documents, unless the specified live load is 80 psf (3.83 kN/m^2) or greater. Weights of all partitions shall be considered, using either actual weights at locations shown on the plans or the equivalent uniform load given in Section 1607.5.2. Partition loads shall be taken as superimposed dead loads.

1607.5.1 Actual loads. Where actual partition weights are used, the uniform design live load may be omitted from the strip of floor area under each partition.

1607.5.2 Equivalent uniform load. The equivalent uniform partition loads in Table 1607.5 may be used in lieu of actual partition weights except for bearing partitions or partitions in toilet room areas (other than in one- and two-family dwellings), at stairs and elevators, and similar areas where partitions are concentrated. In such cases, actual partition weights shall be used in design.

TABLE 1607.5
EQUIVALENT UNIFORM PARTITION LOADS

PARTITION WEIGHT (plf)	EQUIVALENT UNIFORM LOAD (psf) (to be added to floor dead and live loads)
50 or less	0
51 to 100	6
101 to 200	12
201 to 350	20
Greater than 350	20 plus a concentrated live load of the weight in excess of 350 plf.

For SI: 1 pound per linear foot = 0.01459 kN/m^2,
1 pound per square foot = 0.0479 kN/m^2.

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

1. A uniform live load, L, as specified in Items 1.1 and 1.2. This load shall not be reduced.
   1.1. 40 psf (1.92 kN/m^2) where the design basis helicopter has a maximum take-off weight of 3,000 pounds (13.35 kN) or less.
   1.2. 60 psf (2.87 kN/m^2) where the design basis helicopter has a maximum take-off weight greater than 3,000 pounds (13.35 kN).
2. A single concentrated live load, L, of 3,000 pounds (13.35 kN) applied over an area of 4.5 inches by 4.5 inches (114.3 mm by 114.3 mm) and located so as to produce the maximum load effects on the structural elements under consideration. The concentrated load is not required to act concurrently with other uniform or concentrated live loads.
3. Two single concentrated live loads, L, 8 feet (2438.4 mm) apart applied on the landing pad (representing the helicopter's two main landing gear, whether skid type or wheeled type), each having a magnitude of 0.75 times the maximum take-off weight of the helicopter, and located so as to produce the maximum load effects on the structural elements under consideration. The concentrated loads shall be applied over an area of 8 inches by 8 inches (203.2 mm by 203.2 mm) and are not required to act concurrently with other uniform or concentrated live loads.

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

‡‡‡ **1607.7 Heavy vehicle loads.** Floors and other surfaces that are intended to support vehicle loads greater than a 10,000-pound (4535.9 kg) gross vehicle weight rating shall comply with Sections 1607.7.1 through 1607.7.5.

1607.7.1 Loads. Where any structure does not restrict access for vehicles that exceed a 10,000-pound (4535.9 kg) gross vehicle weight rating, those portions of the structure subject to such loads shall be designed using the vehicular live loads, including consideration of impact and fatigue, in accordance with the *NYSDOT Bridge Manual* and the *NYSDOT LRFD Bridge Design Specifications*.

Alternatively, it shall be acceptable to utilize the design live loads for sidewalks as shown in Table 1607.1.

1607.7.2 Fire truck and emergency vehicles. Where a structure or portions of a structure are designated as fire access roads as defined in Chapter 5 of the *New York City Fire Code*, the structure shall be designed for the live loading specified in Section 1607.7.1 of this code or those identified in Chapter 5 of the *New York City Fire Code*, whichever produces the greater load effects.

1607.7.3 Heavy vehicle garages. Garages designed to accommodate vehicles that exceed a 10,000-pound (4535.9 kg) gross vehicle weight rating, shall be designed using the live loading specified by Section 1607.7.1. Vertical impact shall be taken as 10 percent of the vertical load.

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

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

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

1607.7.5 Posting. The maximum weight of vehicles allowed into or on a garage or other structure shall be posted by the owner or the owner's authorized agent in accordance with Article 118 of Chapter 1 of Title 28 of the *Administrative Code*.

1607.8 Loads on handrails, guards, grab bars, and seats. Handrails and guards shall be designed and constructed for the structural loading conditions set forth in Section 1607.8.1. Grab bars, shower seats and accessible benches shall be designed and constructed for the structural loading conditions set forth in Section 1607.8.2.

1607.8.1 Handrails and guards. Handrails and guards shall be designed to resist a linear load of 50 pounds per linear foot (plf) (0.73 kN/m) in accordance with Section 4.5.1.1 of ASCE 7. Glass handrail assemblies and guards shall also comply with Section 2407.

Exceptions:

1. For one- and two-family dwellings, only the single concentrated load required by Section 1607.8.1.1 shall be applied.
2. In Group I-3, F, H and S occupancies, for areas that are not accessible to the general public and that have an occupant load no greater than 50, the minimum load shall be 20 pounds per foot (0.29 kN/m).

1607.8.1.1 Concentrated load. Handrails and guards shall be designed to resist a concentrated load of 200 pounds (0.89 kN) in accordance with Section 4.5.1.1 of ASCE 7.

1607.8.1.2 Intermediate rails. Intermediate rails (all those except the handrail), balusters and panel fillers shall be designed to resist a concentrated load of 50 pounds (0.22 kN) in accordance with Section 4.5.1.1 of ASCE 7, a vertically downward load of 50 pounds per foot (0.73 kN/m), and a concentrated upward load of 50 pounds (0.22 kN) applied at the most critical location. Reactions due to this loading are not required to be applied simultaneously with one another, and are not required to be superimposed with those of Section 1607.8.1 or 1607.8.1.1. The railings, balusters and components shall be designed separately for the effect of wind when the total wind load on the panel or component exceeds 50 pounds (0.22 kN). The wind load need not be combined with any other live load.

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

1607.9 Vehicle barriers. Vehicle barriers for passenger vehicles shall be designed to resist a concentrated load of 6,000 pounds (26.70 kN) in accordance with Section 4.5.3 of ASCE 7.

Garages accommodating trucks and buses shall be designed in accordance with an approved method that contains provisions for traffic railings.

1607.9.1 Columns in parking areas. Unless specially protected, columns in parking areas subject to impact of moving vehicles shall be designed to resist the lateral load due to impact. For passenger vehicles, this lateral load shall be taken as a minimum service live load of 6,000 pounds (26.70 kN) applied at least 1 foot 6 inches (457.2 mm) above the roadway, and acting simultaneously with other design loads. In addition, columns in parking areas shall meet the requirements of Section 1616 for structural integrity. Vehicle barrier systems in garages accommodating trucks and buses shall be designed in accordance with *NYSDOT Bridge Manual* and the *NYSDOT LRFD Bridge Design Specifications*.

1607.10 Impact loads. The live loads specified in Sections 1607.3 through 1607.9 shall be assumed to include adequate allowance for impact conditions. Provisions shall be made in the structural design for uses and loads that involve unusual vibration and impact forces.

1607.10.1 Elevators. Members, elements and components subject to dynamic loads from elevators shall be designed for impact loads and deflection limits prescribed by ASME A17.1, as modified by Appendix K of this code.

1607.10.2 Machinery. For the purpose of design, the weight of machinery and moving loads shall be increased as follows to allow for impact: (i) light machinery, shaft- or motor-driven, 20 percent; and (ii) reciprocating machinery or power-driven units, 50 percent. Percentages shall be increased where specified by the manufacturer.

1607.10.3 Elements supporting building maintenance equipment. In addition to any other applicable live loads, structural elements that support hoists for façade access and building maintenance equipment shall be designed for a live load of 2.5 times the rated load of the hoist or the stall load of the hoist, whichever is greater.

Equipment installed temporarily to facilitate construction, demolition, alteration, or inspection shall be subject to the requirements of Chapter 33.

1607.10.4 Fall arrest and lifeline anchorages. In addition to any other applicable live loads, lifeline anchorages and structural elements that support lifeline anchorages shall be designed for a live load of at least 3,100 pounds (13.8 kN) for each attached lifeline, in every direction that a fall arrest load may be applied.

1607.10.5 Railroad equipment. Minimum loads (including vertical, lateral, longitudinal, and impact) and the distribution thereof shall meet the applicable requirements of Chapter 15 of the AREMA *Manual for Railway Engineering*.

1607.10.6 Assembly structures. Seating areas in grandstands, stadiums, and similar assembly structures shall be designed to resist the simultaneous application of a horizontal swaying load of at least 24 plf (36 kg/m) of seats applied in a direction parallel to the row of the seats, and of at least 10 plf (15 kg/m) of seats in a direction perpendicular to the row of the seats. When this load is used in combination with wind for outdoor structures, the wind load shall be one-half of the design wind load.

1607.11 Reduction in uniform live loads. Except for uniform live loads at roofs, all other minimum uniformly distributed live loads, L_o, in Table 1607.1 are permitted to be reduced in accordance with Section 1607.11.1 or 1607.11.2. Uniform live loads at roofs are permitted to be reduced in accordance with Section 1607.13.2.

1607.11.1 Basic uniform live load reduction. Subject to the limitations of Sections 1607.11.1.1 through 1607.11.1.4 and Table 1607.1, members for which a value of $K_{LL}A_T$ is 400 square feet (37.16 m^2) or more are permitted to be designed for a reduced uniformly distributed live load, L, in accordance with the following equation:

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$$ **(Equation 16-30)**

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

where:

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

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

K_{LL} = Live load element factor (see Table 1607.11.1).

A_T = Tributary area, in square feet (m^2).

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

TABLE 1607.11.1
LIVE LOAD ELEMENT FACTOR, K_{LL}

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

1607.11.1.1 One-way slabs. The tributary area, A_T, for use in Equation 16-23 for one-way slabs shall not exceed an area defined by the slab span times a width normal to the span of 1.5 times the slab span.

1607.11.1.2 Heavy live loads. Live loads that exceed 100 psf (4.79 kN/m^2) shall not be reduced.

Exceptions:

1. The live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall be not less than L as calculated in Section 1607.11.1.
2. For uses other than storage, where approved, additional live load reductions shall be permitted where shown by the registered design professional that a rational approach has been used and that such reductions are warranted.

1607.11.1.3 Passenger vehicle garages. The live loads shall not be reduced in passenger vehicle garages.

Exception: The live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than L as calculated in Section 1607.11.1.

‡‡‡ **1607.11.1.4 Flat slab and flat plate construction.** Live loads shall not be reduced for calculating shear stresses at the heads of columns in flat slab or flat plate construction.

1607.11.2 Alternative uniform live load reduction. As an alternative to Section 1607.11.1 and subject to the limitations of Table 1607.1, uniformly distributed live loads are permitted to be reduced in accordance with the following provisions. Such reductions shall apply to slab systems, beams, girders, columns, piers, walls and foundations.

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

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

2. A reduction shall not be permitted in passenger vehicle parking garages except that the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent.
3. For live loads not exceeding 100 psf (4.79 kN/m^2), the design live load for any structural member supporting 150 square feet (13.94 m^2) or more is permitted to be reduced in accordance with Equation 16-31.
4. For one-way slabs, the area, A, for use in Equation 16-31 shall not exceed the product of the slab span and a width normal to the span of 0.5 times the slab span.

$R = 0.08\ (A - 150)$ **(Equation 16-31)**

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

Such reduction shall not exceed the smallest of:

1. 40 percent for members supporting one floor;
2. 60 percent for members supporting two or more floors; or
3. R as determined by the following equation:

$R = 23.1\ (1 + D/L_o)$ **(Equation 16-32)**

where:

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

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

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

R = Reduction in percent.

1607.12 Distribution of floor loads. Where uniform floor live loads are involved in the design of structural members arranged so as to create continuity, the minimum applied loads shall be the full dead loads on all spans in combination with the floor live loads on spans selected to produce the greatest load effect at each location under consideration. Floor live loads are permitted to be reduced in accordance with Section 1607.11.

1607.13 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.

1607.13.1 Distribution of roof loads. Where uniform roof live loads are reduced to less than 20 psf (0.96 kN/m^2) in accordance with Section 1607.13.2.1 and are applied to the design of structural members arranged so as to create continuity, the reduced roof live load shall be applied to adjacent spans or to alternate spans, whichever produces the most unfavorable load effect. See Section 1607.13.2 for reductions in minimum roof live loads and Section 7.5 of ASCE 7 for partial snow loading.

1607.13.1.1 Arches and gabled frames. The following simplification is permissible:

1. Live load placed on one-half of the span adjacent to one support.
2. Live load placed on the center one-fourth of the span.
3. Live load placed on $^3/_8$ of the span adjacent to each support.

1607.13.2 General. The minimum uniformly distributed live loads of roofs and marquees, L_o, in Table 1607.1 are permitted to be reduced in accordance with Section 1607.13.2.1.

1607.13.2.1 Ordinary roofs, awnings and canopies. Ordinary flat, pitched and curved roofs, and awnings and canopies other than of fabric construction supported by a skeleton structure, are permitted to be designed for a reduced uniformly distributed roof live load, L_r, as specified in the following equations or other controlling combinations of loads as specified in Section 1605, whichever produces the greater load effect.

In structures such as greenhouses, where special scaffolding is used as a work surface for workers and materials during maintenance and repair operations, a lower roof load than specified in the following equations shall not be used unless approved by the commissioner. Such structures shall be designed for a minimum roof live load of 12 psf (0.58 kN/m^2).

$L_r = L_oR_1R_2$ **(Equation 16-33)**

where:

$12 \leq L_r \leq 20$

For SI: $L_r = L_oR_1R_2$

where:

$0.58 \leq L_r \leq 0.96$

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

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

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

$R_1 = 1$ for $A_t \leq 200$ square feet (18.58 m^2) **(Equation 16-34)**

$R_1 = 1.2 - 0.001A_t$ for 200 square feet $< A_t < 600$ square feet **(Equation 16-35)**

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

$R_1 = 0.6$ for $A_t \geq 600$ square feet (55.74 m^2) **(Equation 16-36)**

where:

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

$R_2 = 1$ for $F \leq 4$ **(Equation 16-37)**

$R_2 = 1.2 - 0.05\,F$ for $4 < F < 12$ **(Equation 16-38)**

$R_2 = 0.6$ for $F \geq 12$ **(Equation 16-39)**

where:

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

1607.13.3 Occupiable roofs. Areas of roofs that are occupiable, such as vegetative roofs, roof gardens or for assembly or other similar purposes, and marquees are permitted to have their uniformly distributed live loads reduced in accordance with Section 1607.11.

1607.13.3.1 Vegetative and landscaped roofs. The weight of all landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil as determined in accordance with Section 3.1.4, of ASCE 7. The uniform design live load in unoccupied landscaped areas on roofs shall be 20 psf (0.958 kN/m^2). The uniform design live load for occupied landscaped areas on roofs shall be determined in accordance with Table 1607.1 of this code.

1607.13.4 Awnings, canopies, and sun control devices. Awnings, canopies, and sun control devices shall be designed for uniform live loads as required in Table 1607.1 as well as for snow loads and wind loads as specified in Sections 1608 and 1609.

1607.13.5 Photovoltaic panel systems. Roof structures that provide support for photovoltaic panel systems shall be designed in accordance with Sections 1607.13.5.1 through 1607.13.5.4, as applicable.

1607.13.5.1 Roof live load. Roof structures that support photovoltaic panel systems shall be designed to resist each of the following conditions:

1. Applicable uniform and concentrated roof loads with the photovoltaic panel system dead loads.

   Exception: Roof live loads need not be applied to the area covered by photovoltaic panels where the clear space between the panels and the roof surface is 24 inches (609.6 mm) or less.

2. Applicable uniform and concentrated roof loads without the photovoltaic panel system present.

1607.13.5.2 Photovoltaic panels or modules. The structure of a roof that supports solar photovoltaic panels or modules shall be designed to accommodate the full solar photovoltaic panels or modules and ballast dead load, including concentrated loads from support frames in combination with the loads from Section 1607.13.5.1 and other applicable loads. Where applicable, snow drift loads created by the photovoltaic panels or modules shall be included.

1607.13.5.3 Photovoltaic panels or modules installed as an independent structure. Solar photovoltaic panels or modules that are independent structures and do not have accessible/occupied space underneath are not required to accommodate a roof photovoltaic live load, provided the area under the structure is restricted to keep the public away. All other loads and combinations in accordance with Section 1605 shall be accommodated.

Solar photovoltaic panels or modules that are designed to be the roof, span to structural supports, and have accessible/occupied space underneath shall have the panels or modules and all supporting structures designed to support a roof photovoltaic live load, as defined in Section 1607.13.5.1 in combination with other applicable loads. Solar photovoltaic panels or modules in this application are not permitted to be classified as "not accessible" in accordance with Section 1607.13.5.1.

1607.13.5.4 Ballasted photovoltaic panel systems. Roof structures that provide support for ballasted photovoltaic panel systems shall be designed, or analyzed, in accordance with Section 1604.4; checked in accordance with Section 1604.3.6 for deflections; and checked in accordance with Section 1611 for ponding.

1607.13.6 Hanging loads. Girders and roof trusses (other than joists) over garage areas regularly utilized for the repair of vehicles and over manufacturing floors or storage floors used for commercial purposes shall be capable of supporting, in addition to the specified live and wind loads, a concentrated live load of 2,000 pounds (907.2 kg) applied at any lower chord panel point for trusses, and at any point of the lower flange for girders.

1607.14 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.

1607.14.1 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.

1607.14.2 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

1607.14.3 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.

1607.14.4 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.

1607.15 Interior walls and partitions. Interior walls and partitions that exceed 6 feet (1828.8 mm) in height, including their finish materials, shall have adequate strength and stiffness to resist the loads to which they are subjected but not less than a horizontal load of 5 psf (0.240 kN/m^2).

1607.15.1 Fabric partitions. Fabric partitions that exceed 6 feet (1828.8 mm) in height, including their finish materials, shall have adequate strength and stiffness to resist the following load conditions:

1. The horizontal distributed load need only be applied to the partition framing. The total area used to determine the distributed load shall be the area of the fabric face between the framing members to which the fabric is

attached. The total distributed load shall be uniformly applied to such framing members in proportion to the length of each member.

2. A concentrated load of 40 pounds (0.176 kN) applied to an 8-inch diameter (203.2 mm) area of the fabric face at a height of 54 inches (1371.6 mm) above the floor.

1607.15.2 Fire walls. In order to meet the structural stability requirements of Section 706.2 where the structure on either side of the wall has collapsed, fire walls and their supports shall be designed to withstand a minimum horizontal allowable stress load of 5 psf (0.240 kN/m^2).

SECTION BC 1608
SNOW LOADS

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

1608.2 Ground snow loads. The ground snow loads to be used in determining the design snow loads for roofs is 25 psf (1.2 kN/m^2).

1608.3 Ponding instability. Susceptible bays of roofs shall be evaluated for ponding instability in accordance with Chapters 7 and 8 of ASCE 7.

SECTION BC 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.

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 31 of ASCE 7. The type of opening protection required, the basic design wind speed, V, and the exposure category for a site is permitted to be determined in accordance with Section 1609. Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of ICC 600 shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AWC WFCM.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AISI S230.
4. Designs using NAAMM FP 1001.
5. Designs using TIA-222 for antenna-supporting structures and antennas.
6. Wind tunnel tests in accordance with ASCE 49 and Sections 31.4 and 31.5 of ASCE 7.

The wind speeds in Section 1609.3 are basic design wind speeds, V, and shall be converted in accordance with Section 1609.3.1 to allowable stress design wind speeds, V_{asd}, when the provisions of the standards referenced in Exceptions 4 and 5 are used.

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

1. The hill, ridge or escarpment is 60 feet (18 288 mm) or higher if located in Exposure B or 30 feet (9144 mm) or higher if located in Exposure C;
2. The maximum average slope of the hill exceeds 10 percent; and

3. The hill, ridge or escarpment is unobstructed upwind by other such topographic features for a distance from the high point of 50 times the height of the hill or 2 miles (3.22 km), whichever is greater.

1609.1.2 Protection of openings. In wind-borne debris regions, glazing in buildings shall be impact resistant or protected with an impact-resistant covering meeting the requirements of an approved impact-resistant standard or ASTM E1996 and ASTM E1886 referenced herein as follows:

1. Glazed openings located within 30 feet (9144 mm) of grade shall meet the requirements of the large missile test of ASTM E1996.
2. Glazed openings located more than 30 feet (9144 mm) above grade shall meet the provisions of the small missile test of ASTM E1996.

Exceptions:

1. Wood structural panels with a minimum thickness of $^{7}/_{16}$ inch (11.1 mm) and maximum panel span of 8 feet (2438.4 mm) shall be permitted for opening protection in buildings with a mean roof height of 33 feet (10 058 mm) or less that are classified as a Group R-3 or R-4 occupancy. Panels shall be precut so that they shall be attached to the framing surrounding the opening containing the product with the glazed opening. Panels shall be predrilled as required for the anchorage method and shall be secured with the attachment hardware provided. Attachments shall be designed to resist the components and cladding loads determined in accordance with the provisions of ASCE 7, with corrosion-resistant attachment hardware provided and anchors permanently installed on the building. Attachment in accordance with Table 1609.1.2 of this code with corrosion-resistant attachment hardware provided and anchors permanently installed on the building is permitted for buildings with a mean roof height of 45 feet (13 716 mm) or less where V_{asd} determined in accordance with Section 1609.3.1 of this code does not exceed 140 mph (63 m/s).
2. Glazing in Risk Category I or II buildings, including greenhouses that are occupied for growing plants on a production or research basis, without public access shall be permitted to be unprotected.
3. Glazing in Risk Category III or IV buildings located over 60 feet (18 288 mm) above the ground.

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

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

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

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

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

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

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

1609.1.2.1 Louvers. Louvers protecting intake and exhaust ventilation ducts not assumed to be open that are located within 30 feet (9144 mm) of grade shall meet the requirements of AMCA 540.

1609.1.2.2 Application of ASTM E1996. The text of Section 6.2.2 of ASTM E1996 shall be substituted as follows:

6.2.2 Unless otherwise specified, select the wind zone based on the strength design wind speed, V, as follows:

6.2.2.1 Wind Zone 1—130 mph ≤ basic design wind speed, V < 140 mph.

6.2.2.2 Delete.

6.2.2.3 Delete.

6.2.2.4 Delete.

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

1609.1.3 Minimum wind loads. The design wind pressure, p, used in the design of the main wind-force-resisting system shall not be less than 16 psf (0.766 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 pressure, p, used in the design of 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 32 psf (1.533 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 16 psf (0.766 kN/m^2) multiplied by the area A_f.

1609.1.4 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, including the weight of soils and foundations, shall be taken as the minimum dead load likely to be in place during a design wind event.

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.

1609.1.6 Wind tunnel test limitations. The lower limit on pressures for main wind-force-resisting systems and components and cladding shall be in accordance with this section.

1609.1.6.1 Lower limits on main wind-force-resisting system. Base overturning moments determined from wind tunnel testing shall be limited to not less than 80 percent of the design base overturning moments determined in accordance with Section 27.2 of ASCE 7 unless the wind tunnel results and testing procedure are peer reviewed by an independent wind tunnel laboratory. In no case shall the limiting value be less than 70 percent of the design base overturning moments determined in accordance with Section 27.2 of ASCE 7.

1609.1.6.2 Lower limits on components and cladding. The design pressures for components and cladding on walls or roofs shall be selected as the greater of: (i) the wind tunnel test results; or (ii) 80 percent of the pressure obtained for Zone 4 for walls and Zone 1 for roofs, as determined in Section 30.5 of ASCE 7, unless the wind tunnel results and testing procedure are peer reviewed by an independent wind tunnel laboratory. In no case shall the limiting value be less than 70 percent of the pressure obtained for Zone 4 for walls and Zone 1 for roofs, as determined in Section 30.5 of ASCE 7.

1609.1.6.3 Scope of peer review by independent wind tunnel laboratory. When applicable, the peer reviewer of the wind tunnel test shall:

1. Be retained by or on behalf of the owner and shall be independent from the wind tunnel laboratory that performed the tests and the report shall bear no conflict of interest.
2. Have a minimum of 10 years of technical expertise in the application of wind tunnel studies on buildings comparable to that being reviewed.
3. Have experience in performing or evaluating boundary layer wind tunnel studies and shall be familiar with the technical issues and regulations governing the wind tunnel procedure in ASCE 49.
4. Review the wind tunnel report, including but not limited to wind climate data and modeling, data analysis, boundary layer modeling, building modeling, significant influential buildings, resulting wind loads and other relevant issues identified by the reviewer.
5. Review the wind tunnel report for cladding and envelope pressures, if applicable.

6. Submit a report to the department summarizing the results of their review, and shall state whether the wind tunnel test and the results meet the industry standard, the *New York City Building Code* and are appropriate for the project.

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

BUILDINGS AND OTHER STRUCTURES, FLEXIBLE.

BUILDING, ENCLOSED.

BUILDING, LOW-RISE.

BUILDING, OPEN.

BUILDING, PARTIALLY ENCLOSED.

BUILDING, SIMPLE DIAPHRAGM.

COMPONENTS AND CLADDING.

EAVE HEIGHT, *h*.

EFFECTIVE WIND AREA.

HURRICANE-PRONE REGIONS.

IMPORTANCE FACTOR, *I*.

MAIN WIND FORCE-RESISTING SYSTEM.

MEAN ROOF HEIGHT.

WIND SPEED, V (BASIC).

WIND SPEED, V_{asd} (ALLOWABLE STRESS DESIGN).

WIND-BORNE DEBRIS REGION.

1609.3 Basic design wind speed. The basic design wind speed, V, in mph, for determining wind speed shall be that of Table 1609.3.

TABLE 1609.3
BASIC DESIGN WIND SPEED

Risk Category	Basic Design Wind Speed, mph (3-sec gust, 33 ft. for Exposure Category C)	Mean Recurrence Interval, years	Probability of Exceedance in 50 years, %
I	110	300	15
II	117	700	7
III	127	1700	3
IV	132	3000	1.6

Note: Values are ultimate design 3-second gust wind speeds in miles per hour at 33 feet (10 058 mm) above ground for Exposure Category C.

1609.3.1 Wind speed conversion. Where required, the basic design wind speed, V, of Table 1609.3 shall be converted to allowable stress design wind speed, V_{asd}, using Equation 16-40.

$$V_{asd} = 0.79V \qquad \textbf{(Equation 16-40)}$$

1609.3.2 Wind speed for serviceability design. Table 1609.3.2 presents 3-second gust wind speeds for mean recurrence intervals (MRI) of 10, 25, 50 and 100 years. These wind speeds may be used for serviceability applications such as drift and habitability.

TABLE 1609.3.2
SERVICEABILITY DESIGN WIND SPEED

Mean Recurrence Interval (years)	Serviceability Design Wind Speed, mph (3-sec gust, 33 ft. for Exposure Category C)
10	80
25	87
50	93
100	100

Note: Values are 3-second gust wind speeds in miles per hour at 33 feet (10 058 mm) above ground for Exposure Category C.

1609.4 Exposure category. For each wind direction considered, an exposure category that adequately reflects the characteristics of ground surface irregularities shall be determined for the site at which the building or structure is to be constructed.

Account shall be taken of variations in ground surface roughness that arise from natural topography and vegetation as well as from constructed features.

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

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

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

Surface Roughness C. Open terrain with scattered obstructions having heights generally less than 30 feet (9144 mm). This category includes flat open country, grasslands, and limited water surfaces per Figure 1609.4.3.

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

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

Figure 1609.4.3 provides the exposure categories at the shore lines for wind directions approaching over the water within the city boundaries.

Exposure B. For buildings with a mean roof height of less than or equal to 30 feet (9144 mm), Exposure B shall apply where the ground surface roughness, as defined by Surface Roughness B, prevails in the upwind direction for a distance of at least 1,500 feet (457.2 m). For buildings with a mean roof height greater than 30 feet (9144 mm), Exposure B shall apply where Surface Roughness B prevails in the upwind direction for a distance of at least 2,600 feet (792.5 m) or 20 times the height of the building, whichever is greater.

Exposure C. Exposure C shall apply where it is shown in Figure 1609.4.3 or for all cases where Exposure B or D does not apply.

Exposure D. Exposure D shall apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance of at least 5,000 feet (1524 m) or 20 times the height of the building, whichever is greater. Exposure D shall also apply where the ground surface roughness immediately upwind of the site is B or C, and the site is within a distance of 600 feet (182.9 m) or 20 times the building height, whichever is greater, from an Exposure D condition as defined in the previous sentence.

Figure 1609.4.3 (1)
NEW YORK CITY WIND EXPOSURE: MANHATTAN SHORELINE

Notes:

Exposure C: Buildings within a distance of 2,600 feet from shoreline.

Exposure C: Buildings within a distance of 2,600 feet or 20 times building height from the shoreline, whichever is greater.

Exposure D: Coastal zone, see Section 1609.4.

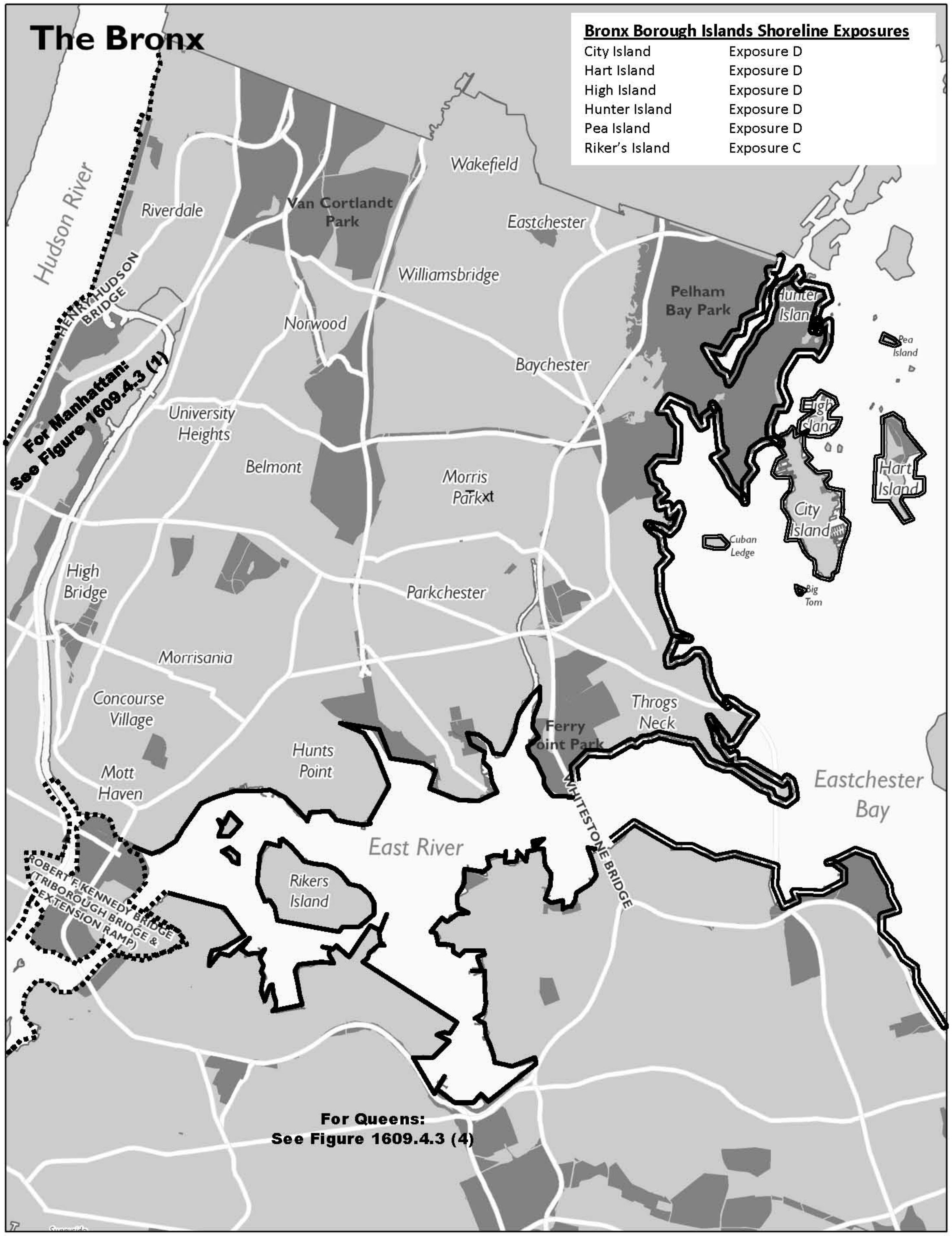

Figure 1609.4.3 (2)
NEW YORK CITY WIND EXPOSURE: BRONX SHORELINE

Notes:

Exposure C: Buildings within a distance of 2,600 feet from shoreline.

Exposure C: Buildings within a distance of 2,600 feet or 20 times building height from the shoreline, whichever is greater.

Exposure D: Coastal zone, see Section 1609.4.

1. All islands northeast of Whitestone Bridge are Exposure Category D.

Figure 1609.4.3 (3)
NEW YORK CITY WIND EXPOSURE: BROOKLYN SHORELINE

Notes:

- ▪▪▪▪▪ Exposure C: Buildings within a distance of 2,600 feet from shoreline.
- ▬▬ Exposure C: Buildings within a distance of 2,600 feet or 20 times building height from the shoreline, whichever is greater.
- ══ Exposure D: Coastal zone, see Section 1609.4.

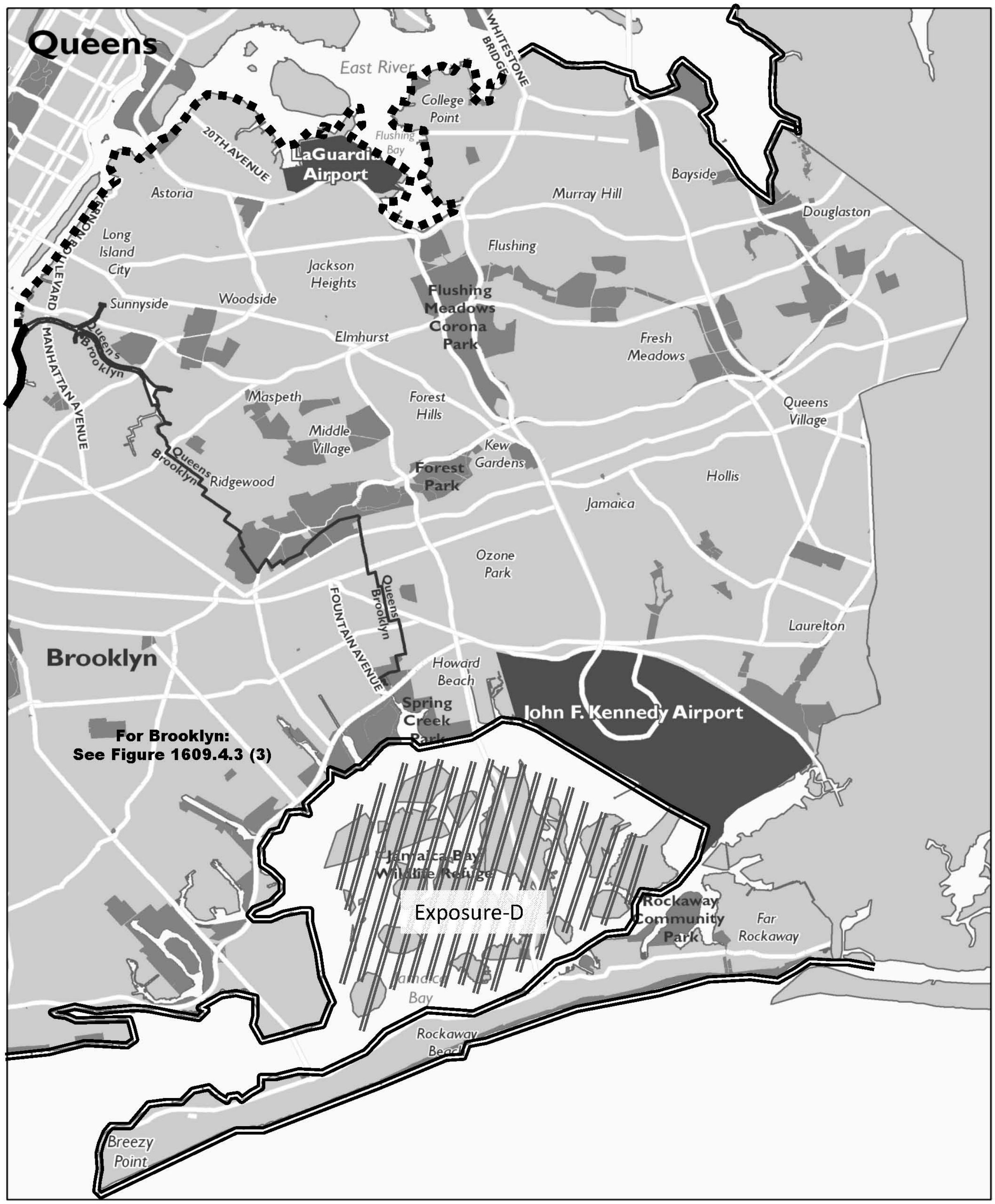

Figure 1609.4.3 (4)
NEW YORK CITY WIND EXPOSURE: QUEENS SHORELINE

Notes:

Exposure C: Buildings within a distance of 2,600 feet from shoreline.
Exposure C: Buildings within a distance of 2,600 feet or 20 times building height from the shoreline, whichever is greater.
Exposure D: Coastal zone, see Section 1609.4.
1. All islands located in the Jamaica Bay area are Exposure Category D.

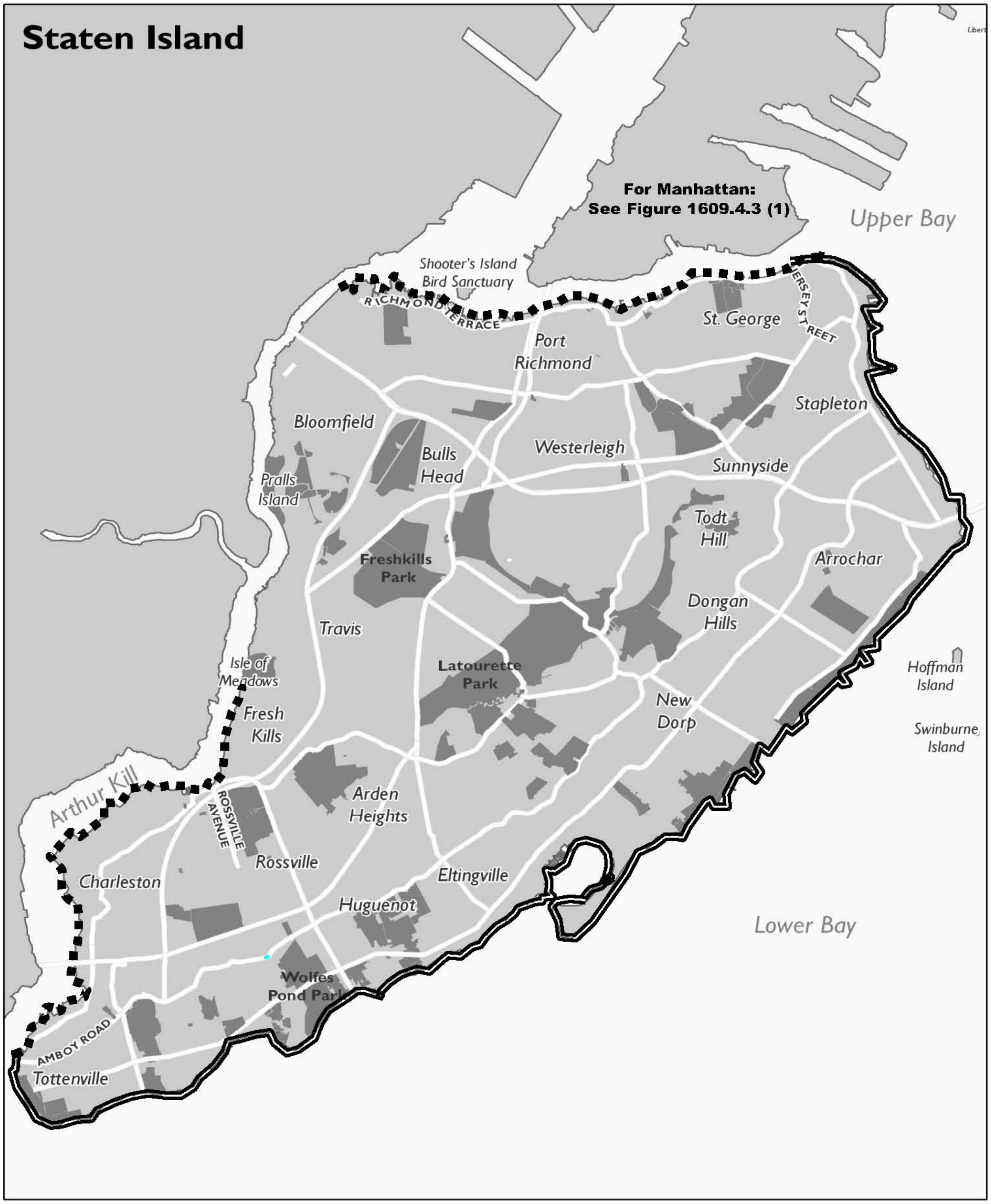

Figure 1609.4.3 (5)
NEW YORK CITY WIND EXPOSURE: STATEN ISLAND SHORELINE

Notes:

Exposure C: Buildings within a distance of 2,600 feet from shoreline.

Exposure C: Buildings within a distance of 2,600 feet or 20 times building height from the shoreline, whichever is greater.

Exposure D: Coastal zone, see Section 1609.4.

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

1609.5.1 Roof deck. The roof deck shall be designed to withstand the wind pressures determined in accordance with Section 1609.1.1 for buildings of any height.

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

Exceptions:

1. Rigid tile roof coverings that are air permeable and installed over a roof deck complying with Section 1609.5.1 are permitted to be designed in accordance with Section 1609.5.3.
2. Asphalt shingles installed over a roof deck complying with Section 1609.5.1 shall comply with the wind-resistance requirements of Section 1504.1.1.

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

$M_a = q_h C_L b L L_a (1.0 - GC_p)$ **(Equation 16-41)**

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

where:

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

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

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

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

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

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

q_h = Wind velocity pressure, psf (kN/m^2) determined from Section 26.10.2 of ASCE 7.

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

1. The roof tiles shall be either loose laid on battens, mechanically fastened, mortar set or adhesive set.
2. The roof tiles shall be installed on solid sheathing that 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 (50.8 mm).
5. The length of the tile shall be between 1 and 1.75 feet (304.8 mm and 533.4 mm).
6. The exposed width of the tile shall be between 0.67 and 1.25 feet (204.2 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.

1609.6 Simplified design procedure. Use of the simplified design procedure shall comply with Sections 1609.6.1 through 1609.6.5.

1609.6.1 Scope. The procedures in Section 1609.6 shall be permitted to be used for determining and applying wind pressures in the design of enclosed buildings in Risk Category I and II as listed below:

1. For buildings located within any borough with a mean roof height of not more than 200 feet (60 960 mm) and not located in Exposure C or D in accordance with Section 1609.4, the use of Section 1609.6 shall be permitted.
2. For buildings located within the Borough of Manhattan with a mean roof height of not more than 300 feet (91 440 mm) and not located in Exposure C or D in accordance with Section 1609.4, the use of Section 1609.6 shall be permitted.

1609.6.2 Main windforce-resisting systems. Main windforce-resisting systems shall comply with the following:

1. The building shall be designed for the following net lateral wind pressure to be applied to the horizontal projection of the building surfaces:

   1.1. From 0 to 100 feet (0 to 30 480 mm) elevation 32 psf (1.54 kN/m^2).

   1.2. From 100 to 300 feet (30 480 to 91 440 mm) elevation 40 psf (1.92 kN/m^2).

1609.6.3 Design wind load cases. The main windforce-resisting system of buildings, whose wind loads have been determined pursuant to Section 1609.6.1, shall be designed for wind load cases as defined below:

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Seventy-five percent of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure to be applied eccentric to the center of the exposure with eccentricity equal to 15 percent of the exposure width, considered separately for each principal direction.

Case 3. Wind loading as defined in Case 1 for each orthogonal direction, but considered to act simultaneously at 75 percent of the specified value.

1609.6.4 Components and cladding. Net design wind positive and negative pressures (pressure and suction) for the components and cladding of buildings represent the net pressures (sum of internal and external) to be applied normal to each building surface. The net design wind positive and negative pressures shall not be less than 48 psf (12.30 kN/m^2), except at the corners of the building with a width equivalent to 10 percent of the building's width at its side, the net design wind negative pressure for the components and cladding shall not be less than: (i) 72 psf (3.46 kN/m^2) for the portion of the building between 200 feet (60 960 mm) to 300 feet (91 440 mm) height above ground and (ii) 64 psf (3.07 kN/m^2) for the portion of the building between 100 feet (30 480 mm) to 199 feet (60 656 mm) in height above ground.

1609.6.5 Roof. The design pressure and suction acting over the entire roof including purlins, roofing, and other roof elements (including their fastenings) shall be not less than 48 psf (2.30 kN/m^2).

1609.7 Attached canopies on buildings. Canopies attached to buildings shall comply with Section 1609.7.1 or 1609.7.2, as applicable.

TABLE 1609.7
NET PRESSURE COEFFICIENT, (GC_{PN}) FOR ATTACHED CANOPIES ON BUILDINGS WITH H>60 FT

Net Pressure Coefficient (GC_{pn})	h/hc						
	2	3	6	12	18	24	30
Upward	-0.60	-0.70	-0.95	-1.04	-1.16	-1.3	-1.3
Downward	0.90	0.90	0.90	0.90	0.90	0.90	0.90

For SI: 1 foot = 304.8 mm.

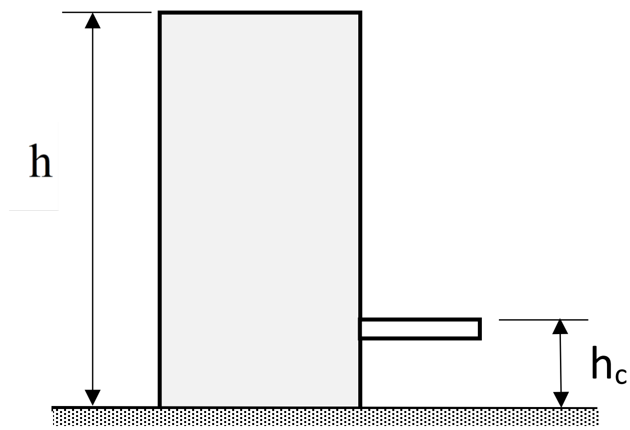

FIGURE 1609.7
SCHEMATIC OF ATTACHED CANOPY
ON BUILDINGS WITH *h* > 60 FT

1609.7.1 Buildings with *h* ≤ 60 ft. The design wind pressure for canopies attached to walls of buildings with *h* ≤ 60 ft. shall follow Chapter 30 of ASCE 7.

1609.7.2 Buildings with *h* > 60 ft. The design wind pressure for canopies attached to walls of buildings with *h* > 60 ft. and *hc*/*h* < 0.5, as shown on Figure 1609.7, shall be from the following equation:

$P = q_h(GC_{pn})$ (lb/ft^2) **(Equation 16-42)**

Where:

q_h = velocity pressure at mean roof height *h* per Chapter 26 of ASCE 7.

GC_{pn} =net pressure coefficient for attached canopies given in Table 1609.7.

1609.8 Wind load on structures during construction. Structures during construction shall comply with Sections 1609.8.1 and 1609.8.2.

1609.8.1 Design wind speed. Design wind speed during construction for a construction period of less than 5 years may be reduced as follows:

Unless otherwise directed by the commissioner, the basic design wind speed may be reduced to 110 mph for all risk categories, corresponding to 1.7 percent probability of exceedance in 5 years. The basic design wind speed refers to the ultimate design 3-second gust wind speed at 33 feet (10 058.4 mm) above ground for Exposure Category C, as defined in Section 1609.3.

1609.8.2 Frameworks without cladding. For unenclosed frames and elements, the appropriate drag factors and wind effect on successive elements shall be accounted for per recognized standards and references, or in accordance with suitable wind tunnel testing.

SECTION BC 1610
SOIL LATERAL LOADS

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

Exception: Foundation walls extending not more than 8 feet (2438.4 mm) below grade and laterally supported at the top by flexible diaphragms shall be permitted to be designed for active pressure.

TABLE 1610.1
LATERAL SOIL LOAD

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

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

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

b. Unsuitable as backfill material.

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

SECTION BC 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.

$R = 5.2\ (d_s + d_h)$ **(Equation 16-43)**

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

where:

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

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

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

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

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 BC 1612
FLOOD LOADS

1612.1 General. Buildings and other structures located in whole or in part in flood hazard areas shall meet the requirements of Section 5.3 of ASCE 7 and Appendix G of this code. The structural design shall be based on the design loads stated in Section 5.4 of ASCE 7, the load combinations of Section 1605, and Appendix G of this code.

1612.2 Reserved.

1612.3 Reserved.

1612.4 Reserved.

1612.5 Reserved.

SECTION BC 1613
EARTHQUAKE LOADS

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

Exceptions:

1. One- and two-family dwellings three stories or less in height.
2. The seismic force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.
3. Agricultural storage structures intended only for incidental human occupancy.
4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.

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

DESIGN EARTHQUAKE GROUND MOTION.

MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION.

MAXIMUM CONSIDERED EARTHQUAKE GEOMETRIC MEAN (MCEG) PEAK GROUND ACCELERATION.

MECHANICAL SYSTEMS.

ORTHOGONAL.

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCER) GROUND MOTION RESPONSE ACCELERATIONS.

SEISMIC DESIGN CATEGORY.

SEISMIC FORCE-RESISTING SYSTEM.

SITE CLASS.

SITE COEFFICIENTS.

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

1613.3.1 Mapped acceleration parameters. The mapped maximum considered earthquake spectral response acceleration at short periods (S_s) shall be 0.296 g and at 1-s period (S_1) shall be 0.061 g. The mapped long-period transition period (T_L) shall be 6.0 seconds. Alternatively, electronic values of mapped acceleration parameters S_s and S_1 and other seismic design parameters provided at the U.S. Geological Survey (USGS) website at https://doi.org/10.5066/F7NK3C76 may be used as per the guidelines of ASCE 7, Section 11.4.

1613.3.2 Site class definitions. Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E or F in accordance with Chapter 20 of ASCE 7, and this code. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the commissioner or geotechnical data determines Site Class E or F soils are present at the site.

1613.3.3 Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. The maximum considered earthquake spectral response acceleration for short periods, S_{MS}, and at 1-second period, S_{M1}, adjusted for site class effects shall be determined by Equations 16-44 and 16-45, respectively:

$S_{MS} = F_a S_S$ **(Equation 16-44)**

$S_{M1} = F_v S_1$ **(Equation 16-45)**

where:

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

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

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

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

Exception: When electronic values of mapped acceleration parameters are used as per Section 1613.3.1 of this code, the site coefficients, F_a and F_v, cannot be taken from Tables 1613.3.3(1) and 1613.3.3(2) of this code. Rather, general procedures in Chapter 11 of ASCE 7 shall be followed to determine F_a and F_v.

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a AS A FUNCTION OF SITE CLASS AND MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS (SS)[a]

SITE CLASS	F_a
A	0.80
B (V_s measured)	0.90

B (V_s unmeasured)	1.00
C	1.30
D	1.57
E	2.28
F	Note a

a. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values, except that for structures with periods of vibration equal or less than 0.5 second, values of F_a for liquefiable soils are permitted to be taken equal to the values for the site class determined without regard to liquefaction in Section 1613.3.4.1.

TABLE 1613.3.3(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	F_v
A	0.80
B (V_s measured)	0.90
B (V_s unmeasured)	1.00
C	1.50
D	2.40
E	4.20
F	Note a

a. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values, except that for structures with periods of vibration equal or less than 0.5 second, values of F_v for liquefiable soils are permitted to be taken equal to the values for the site class determined without regard to liquefaction in Section 1613.3.4.1.

1613.3.4 Design spectral response acceleration parameters. Five-percent damped design spectral response acceleration at short periods, S_{DS}, and at 1-s period, S_{D1}, shall be determined from Equations 16-46 and 16-47, respectively:

$S_{DS} = {}^2/_3 S_{MS}$ **(Equation 16-46)**

$S_{D1} = {}^2/_3 S_{M1}$ **(Equation 16-47)**

S_{MS} =The MCE_R spectral response accelerations for short periods as determined in Section 1613.3.3.

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

1613.3.4.1 Site classification for seismic design. Site classification for Site Class C, D or E shall be determined from Table 1613.3.4.1. The notations presented below apply to only materials encountered above rock meeting Class 1a, 1b, or 1c as defined in Section 1803 or rock with shear wave velocity greater than 2,500 feet per second (762 meters per second) to a maximum depth of 100 feet (30 480 mm). Profiles containing distinctly different soil and rock layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there is 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. For situations in which site investigations, performed in accordance with Chapter 20 of ASCE 7, reveal rock conditions consistent with Site Class B, but site-specific velocity measurements are not made, the site coefficients F_a, F_v, and F_{PGA} shall be taken as unity (1.0).

where:

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

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

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

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

(Equation 16-48)

where:

N_i is the Standard Penetration Resistance (ASTM D1586) not to exceed 100 blows/foot (328 blows/m) as directly measured in the field without corrections. When refusal is met for a rock layer of Class 1d, Ni shall be less than or equal to 100 blows/foot (328 blows/m) provided that the extent of the Class 1d material is confirmed by a boring to a depth where Class 1c or better rock is determined, not to exceed 100 feet (30 480 mm). Alternatively, if this boring is not performed, site classification should be based on all soil material that is above the Class 1d layer.

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

(Equation 16-49)

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

(Equation 16-50)

where:

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

Use d_i and N_i for cohesionless soil layers only in Equation 16-50.

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

m = The number of cohesionless soil layers in the top 100 feet (30 480 mm).

S_{ui} = The undrained shear strength in psf (kPa), not to exceed 5,000 psf (240 kPa), ASTM D 2166 or ASTM D 2850.

$$\bar{S}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{S_{ui}}}$$

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).

k = The number 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.

Where a site does not qualify under the criteria for Site Class F and there is a total thickness of soft clay greater than 10 feet (3048 mm) where a soft clay layer is defined by s_u < 500 psf (24 kPa), w > 40 percent, and PI > 20, it shall be classified as Site Class E. 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 category, Site Class A, 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 are permitted to be extrapolated to assess 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.

TABLE 1613.3.4.1
SITE CLASSIFICATION[a]

SITE CLASS	v_s	N or N_{ch}	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 s_v method is used and the N_{ch} and s_v criteria differ, select the category with the softer soils (for example, use Site Class E instead of D).

1613.3.4.1.1 Steps for classifying a site. The following steps shall be performed during the classification of 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 according to ASCE 7 and the requirements of Section 1815.
2. Check for the existence of a total thickness of soft clay > 10 feet (3048 mm) where a soft clay layer is defined by: s_u < 500 psf (24 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 v_s, N, or s_u and computed in all cases as specified.
   3.1. v_s for the top 100 feet (30 480 mm) (v_s method).
   3.2. N for the top 100 feet (30 480 mm) (N method).
   3.3. N_{ch} for cohesionless soil layers (PI < 20) in the top 100 feet (30 480 mm) and average, s_u, for cohesive soil layers (PI > 20) in the top 100 feet (30 480 mm) (s_u method).

1613.3.5 Determination of seismic design category. All structures shall be assigned a seismic design category based on their risk category and the design spectral response acceleration parameters, S_{DS} and S_{D1}, determined in accordance with Section 1613.3.4 of this code or the site-specific procedures of ASCE 7. Each building and structure shall be assigned to the seismic design category in accordance with Table 1613.3.5 of this code, irrespective of the fundamental period of vibration of the structure, T.

Exception: When electronic values of mapped acceleration parameters are used as per Section 1613.3.1, the seismic design category for Risk Category IV structures shall not differ from the values in Table 1613.3.5.

TABLE 1613.3.5
SEISMIC DESIGN CATEGORY

RISK CATEGORY	SITE CLASS		
	A	B/C/D	E
I/II/III	A	B	C
IV	A	C	D

1613.3.5.1 Alternative seismic design category determination. The seismic design category is permitted to be determined from Table 11.6-1 of ASCE 7 alone, except for Risk Category IV structures for which Table 1613.3.5 of this code shall be used, when all of the following apply:

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, T_a, in each of the two orthogonal directions determined in accordance with Section 12.8.2.1 of ASCE 7, is less than 0.8 T_s determined in accordance with Section 11.4.6 of ASCE 7.
2. In each of the two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.
3. Equation 12.8-2 of ASCE 7 is used to determine the seismic response coefficient, C_s.
4. The diaphragms are rigid or are permitted to be idealized as rigid in accordance with Section 12.3.1 of ASCE 7 or, for diaphragms permitted to be idealized as flexible in accordance with Section 12.3.1 of ASCE 7, the distances between vertical elements of the seismic force-resisting system do not exceed 40 feet (12 192 mm).

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

1613.4 Structural separations. All structures shall be separated from adjacent structures. When a structure adjoins a property line not common to a public way (typically side or rear lot lines), that structure shall also be set back from the property line by at least 1 inch (25.4 mm) for each 50 feet (15 240 mm) of height and a minimum of 1 inch (25.4 mm) for structures with heights less than 50 feet (15 240 mm). For structures in Seismic Design Category D, refer to ASCE 7 for additional requirements.

Exception: Smaller separations or property line setbacks shall be permitted when justified by rational analysis based on maximum expected ground motions with a minimum separation of 1 inch (25.4 mm) along the full height of the structure.

1613.4.1 Masonry structures. For structures adjacent to existing unreinforced masonry bearing wall structures, the structural separation shall be filed with a material with a minimum compressive strength of 25 psi (172.37 kPa) and a maximum compressive strength of 100 psi (689.74 kPa). Additionally, when the adjacent wall is a party wall, the party wall shall be made secure by the party responsible for the new construction as per Chapter 33.

1613.4.2 Covers. The infill material shall be covered on all sides and shall meet the appropriate provisions of Chapter 26. The covering must be of adequate strength to resist the wind loads for cladding as specified in Chapter 16 and shall conform to all applicable provisions in Chapter 14.

1613.4.3 Covers wider than 5 inches (127 mm). When a building separation wider than 5 inches (127 mm) is created pursuant to Section 1613.4, such separation, at the roof level of the proposed new building, or at the roof level of an existing adjoining building where if that building is lower than the proposed new building, shall have a horizontal cover/closure that conforms with the following:

1. The cover/closure material shall be noncombustible; and

   Exception: The cover/closure material used shall be permitted to be combustible material in accordance with Section 1510.9 if all the material on the appropriate roof conforms to the limitations therein, there are no masonry openings in either wall abutting the building separation, and both buildings are noncombustible.

2. The cover/closure shall be capable of withstanding the roof live load of 30 psf (1.43 kPa), securely fastened to the new building, and be of a type that would be capable of preventing unauthorized or accidental access to the space.

1613.5 ASCE 7, Table 12.2-1. Modify ASCE 7, Table 12.2-1 as follows:

TABLE 1613.5
DESIGN COEFFICIENT AND FACTORS FOR BASIC SEISMIC C-FORCE-RESISTING SYSTEMS

					STRUCTURAL SYSTEM LIMITATIONS INCLUDING STRUCTURAL HEIGHT, h_n (ft), LIMITS[c]		
Seismic Force-Resisting System	**ASCE 7 Section Where Detailing Requirements Are Specified**	**Response Modification Coefficient**	**Overstrength Factor**	**Deflection Amplification Factor**	**Seismic Design Category**		
A. BEARING WALL SYSTEMS		**R**[a]	**Ω_0**[g]	**C_d**[b]	**B**	**C**	**D**[d]
1. Special reinforced concrete shear walls[l, m]	14.2	5	2.5	5	NL	NL	160
2. Ordinary reinforced concrete shear walls[l]	14.2	4	2.5	4	NL	NL	NP
3. Detailed plain concrete shear walls[l]	14.2	2	2.5	2	NL	NP	NP
4. Ordinary plain concrete shear walls[l]	14.2	1.5	2.5	1.5	NL	NP	NP
5. Intermediate precast shear walls[l]	14.2	4	2.5	4	NL	NL	40[k]
6. Ordinary precast shear walls[l]	14.2	3	2.5	3	NL	NP	NP
7. Special reinforced masonry shear walls	14.4	5	2.5	3.5	NL	NL	160
8. Intermediate reinforced masonry shear walls	14.4	3.5	2.5	2.25	NL	NL	NP

(continued)

TABLE 1613.5—continued
DESIGN COEFFICIENT AND FACTORS FOR BASIC SEISMIC C-FORCE-RESISTING SYSTEMS

					STRUCTURAL SYSTEM LIMITATIONS INCLUDING STRUCTURAL HEIGHT, h_n (ft), LIMITS[c]		
Seismic Force-Resisting System	**ASCE 7 Section Where Detailing Requirements Are Specified**	**Response Modification Coefficient**	**Overstrength Factor**	**Deflection Amplification Factor**	**Seismic Design Category**		
A. BEARING WALL SYSTEMS, cont.		**R**[a]	**Ω_0**[g]	**C_d**[b]	**B**	**C**	**D**[d]
9. Ordinary reinforced masonry shear walls	14.4	2	2.5	1.75	NL	160	NP
10. Detailed plain masonry shear walls	14.4	2	2.5	1.75	NL	NP	NP
11. Ordinary plain masonry shear walls	14.4	1.5	2.5	1.25	NL	NP	NP
12. Prestressed masonry shear walls	14.4	1.5	2.5	1.75	NL	NP	NP

13. Ordinary reinforced Autoclaved Aerated Concrete (AAC) masonry shear walls	14.4	2	2.5	2	NL	35	NP
14. Ordinary plain (unreinforced) Autoclaved Aerated Concrete (AAC) masonry shear walls	14.4	1.5	2.5	1.5	NL	NP	NP
15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1 and 14.5	6.5	3	4	NL	NL	65
16. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6.5	3	4	NL	NL	65
17. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	2.5	2	NL	NL	35
18. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1	4	2	3.5	NL	NL	65
B. BUILDING FRAME SYSTEMS		**R**[a]	**Ω_0**[g]	**C_d**[b]	**B**	**C**	**D**[d]
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160
3. Steel ordinary concentrically braced frames	14.1	3.25	2	3.25	NL	NL	35[j]
4. Special reinforced concrete shear walls [l, m]	14.2	6	2.5	5	NL	NL	160
5. Ordinary reinforced concrete shear walls [l]	14.2	5	2.5	4.5	NL	NL	NP
6. Detailed plain concrete shear walls [l]	14.2 and 14.2.2.7	2	2.5	2	NL	NP	NP
7. Ordinary plain concrete shear walls [l]	14.2	1.5	2.5	1.5	NL	NP	NP
8. Intermediate precast shear walls [l]	14.2	5	2.5	4.5	NL	NL	40[k]
9. Ordinary precast shear walls [l]	14.2	4	2.5	4	NL	NP	NP
10. Steel and concrete composite eccentrically braced frames	14.3	8	2.5	4	NL	NL	160
11. Steel and concrete composite special concentrically braced frames	14.3	5	2	4.5	NL	NL	160
12. Steel and concrete composite ordinary braced frames	14.3	3	2	3	NL	NL	NP
13. Steel and concrete composite plate shear walls	14.3	6.5	2.5	5.5	NL	NL	160
14. Steel and concrete composite special shear walls	14.3	6	2.5	5	NL	NL	160
15. Steel and concrete composite ordinary shear walls	14.3	5	2.5	4.5	NL	NL	NP
16. Special reinforced masonry shear walls	14.4	5.5	2.5	4	NL	NL	160
17. Intermediate reinforced masonry shear walls	14.4	4	2.5	4	NL	NL	NP
18. Ordinary reinforced masonry shear walls	14.4	2	2.5	2	NL	160	NP
19. Detailed plain masonry shear walls	14.4	2	2.5	2	NL	NP	NP
20. Ordinary plain masonry shear walls	14.4	1.5	2.5	1.25	NL	NP	NP
21. Prestressed masonry shear walls	14.4	1.5	2.5	1.75	NL	NP	NP

22. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	7	2.5	4.5	NL	NL	65

(continued)

TABLE 1613.5—continued
DESIGN COEFFICIENT AND FACTORS FOR BASIC SEISMIC C-FORCE-RESISTING SYSTEMS

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient	Overstrength Factor	Deflection Amplification Factor	STRUCTURAL SYSTEM LIMITATIONS INCLUDING STRUCTURAL HEIGHT, h_n (ft), LIMITS[c] Seismic Design Category		
B. BUILDING FRAME SYSTEMS, cont.		**R[a]**	**Ω_0[g]**	**C_d[b]**	**B**	**C**	**D[d]**
23. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	7	2.5	4.5	NL	NL	65
24. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2.5	2.5	2.5	NL	NL	35
25. Steel buckling-restrained braced frames	14.1	8	2.5	5	NL	NL	160
26. Steel special plate shear walls	14.1	7	2	6	NL	NL	160
C. MOMENT-RESISTING FRAME SYSTEMS		**R[a]**	**Ω_0[g]**	**C_d[b]**	**B**	**C**	**D[d]**
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5.5	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5.5	NL	NL	160
3. Steel intermediate moment frames	14.1 and 12.2.5.7	4.5	3	4	NL	NL	35[h]
4. Steel ordinary steel moment frames	14.1 and 12.2.5.6	3.5	3	3	NL	NL	NP[i]
5. Special reinforced concrete moment frames [n]	14.2 and 12.2.5.5	8	3	5.5	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4.5	NL	NL	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2.5	NL	NP	NP
8. Steel and concrete composite special moment frames	14.3 and 12.2.5.5	8	3	5.5	NL	NL	NL
9. Steel and concrete composite intermediate moment frames	14.3	5	3	4.5	160	160	NP
10. Steel and concrete composite partially restrained moment frames	14.3	6	3	5.5	NL	NL	100
11. Steel and concrete composite ordinary moment frames	14.3	3	3	2.5	NL	NP	NP
12. Cold-formed steel—special bolted moment frame [p]	14.1	3.5	3[o]	3.5	35	35	35
D. DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	**12.2.5.1**	**R[a]**	**Ω_0[g]**	**C_d[b]**	**B**	**C**	**D[d]**

1. Steel eccentrically braced frames	14.1	8	2.5	4	NL	NL	NL
2. Steel special concentrically braced frames	14.1	7	2.5	5.5	NL	NL	NL
3. Special reinforced concrete shear walls [l]	14.2	7	2.5	5.5	NL	NL	NL
4. Ordinary reinforced concrete shear walls [l]	14.2	6	2.5	5	NL	NL	NP
5. Steel and concrete composite eccentrically braced frames	14.3	8	2.5	4	NL	NL	NL
6. Steel and concrete composite special concentrically braced frames	14.3	6	2.5	5	NL	NL	NL
7. Steel and concrete composite plate shear walls	14.3	7.5	2.5	6	NL	NL	NL
8. Steel and concrete composite special shear walls	14.3	7	2.5	6	NL	NL	NL
9. Steel and concrete composite ordinary shear walls	14.3	6	2.5	5	NL	NL	NP
10. Special reinforced masonry shear walls	14.4	5.5	3	5	NL	NL	NL
11. Intermediate reinforced masonry shear walls	14.4	4	3	3.5	NL	NL	NP
12. Steel buckling-restrained braced frames	14.1	8	2.5	5	NL	NL	NL
13. Steel special plate shear walls	14.1	8	2.5	6.5	NL	NL	NL

(continued)

TABLE 1613.5—continued
DESIGN COEFFICIENT AND FACTORS FOR BASIC SEISMIC C-FORCE-RESISTING SYSTEMS

					STRUCTURAL SYSTEM LIMITATIONS INCLUDING STRUCTURAL HEIGHT, h_n (ft), LIMITS[c]		
Seismic Force-Resisting System	**ASCE 7 Section Where Detailing Requirements Are Specified**	**Response Modification Coefficient**	**Overstrength Factor**	**Deflection Amplification Factor**	**Seismic Design Category**		
E. DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	**12.2.5.1**	**R[a]**	**Ω_0[g]**	**C_d[b]**	**B**	**C**	**D[d]**
1. Steel special concentrically braced frames[f]	14.1	6	2.5	5	NL	NL	35
2. Special reinforced concrete shear walls [l]	14.2	6.5	2.5	5	NL	NL	160
3. Ordinary reinforced masonry shear walls	14.4	3	3	2.5	NL	160	NP
4. Intermediate reinforced masonry shear walls	14.4	3.5	3	3	NL	NL	NP
5. Steel and concrete composite special concentrically braced frames	14.3	5.5	2.5	4.5	NL	NL	NL
6. Steel and concrete composite ordinary braced frames	14.3	3.5	2.5	3	NL	NL	NP
7. Steel and concrete composite ordinary shear walls	14.3	5	3	4.5	NL	NL	NP
8. Ordinary reinforced concrete shear walls [l]	14.2	5.5	2.5	4.5	NL	NL	NP

F. SHEAR WALL-FRAME INTERACTIVE SYSTEM WITH ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS[l]	**14.2 and 12.2.5.8**	**4.5**	**2.5**	**4**	**NL**	**NP**	**NP**
G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR:	**12.2.5.2**	**R[a]**	**Ω_0[g]**	**C_d[b]**	**B**	**C**	**D[d]**
1. Steel special cantilever column systems	14.1	2.5	1.25	2.5	35	35	35
2. Steel ordinary cantilever column systems	14.1	1.25	1.25	1.25	35	35	NP[i]
3. Special reinforced concrete moment frames[n]	14.2 and 12.2.5.5	2.5	1.25	2.5	35	35	35
4. Intermediate reinforced concrete moment frames	14.2	1.5	1.25	1.5	35	35	NP
5. Ordinary reinforced concrete moment frames	14.2	1	1.25	1	35	NP	NP
6. Timber frames	14.5	1.5	1.5	1.5	35	35	35
H. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE, EXCLUDING CANTILEVER COLUMN SYSTEMS	**14.1**	**3**	**3**	**3**	**NL**	**NL**	**NP**

a. Response modification coefficient, *R*, for use throughout the standard. Note *R* reduces forces to a strength level, not an allowable stress level.
b. Deflection amplification factor, C_d, for use in Sections 12.8.6, 12.8.7, and 12.9.1.2 of ASCE 7.
c. NL = Not Limited and NP = Not Permitted. For metric units use 30.5 m for 100 ft and use 48.8 m for 160 ft.
d. See Section 12.2.5.4 of ASCE 7 for a description of seismic force-resisting systems limited to buildings with a structural height, h_n, of 240 ft (73.2 m) or less.
e. See Section 12.2.5.4 of ASCE 7 for seismic force-resisting systems limited to buildings with a structural height, h_n, of 160 ft (48.8 m) or less.
f. Ordinary moment frame is permitted to be used in lieu of intermediate moment frame for Seismic Design Categories B or C.
g. Where the tabulated value of the overstrength factor, Ω_0, is greater than or equal to $2^1/_2$, Ω_0 is permitted to be reduced by subtracting the value of $^1/_2$ for structures with flexible diaphragms.
h. See Section 12.2.5.7 of ASCE 7 for limitations in structures assigned to Seismic Design Category D.
i. See Section 12.2.5.6 of ASCE 7 for limitations in structures assigned to Seismic Design Category D.
j. Steel ordinary concentrically braced frames are permitted in single-story buildings up to a structural height, h_n, of 60 ft (18.3 m) where the dead load of the roof does not exceed 20 psf (0.96 kN/m^2) and in penthouse structures.
k. An increase in structural height, h_n, to 45 ft (13.7 m) is permitted for single story storage warehouse facilities.
l. In Section 2.3 of ACI 318. A shear wall is defined as a structural wall.
m. In Section 2.3 of ACI 318. The definition of "special structural wall" includes precast and cast-in-place construction.
n. In Section 2.3 of ACI 318. The definition of "special moment frame" includes precast and cast-in-place construction.
o. Alternately, the seismic load effect with overstrength, E_{mh}, is permitted to be based on the expected strength determined in accordance with AISI S400.
p. Cold-formed steel – special bolted moment frames shall be limited to one-story in height in accordance with AISI S400.

1613.6 Ballasted photovoltaic panel systems. Ballasted, roof-mounted photovoltaic panel systems need not be rigidly attached to the roof or supporting structure. Ballasted non-penetrating systems shall be designed and installed only on roofs with slopes not more than one unit vertical in 12 units horizontal. Ballasted nonpenetrating systems shall be designed to resist sliding and uplift resulting from lateral and vertical forces as required by Section 1605 of this code, using a coefficient of friction determined by acceptable engineering principles. In structures assigned to Seismic Design Category C or D, ballasted nonpenetrating systems shall be designed to accommodate seismic displacement determined by nonlinear response-history analysis or shake-table testing, using input motions consistent with ASCE 7 lateral and vertical seismic forces for nonstructural components on roofs.

SECTION BC 1614
ATMOSPHERIC ICE LOADS

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

SECTION BC 1615
STRUCTURAL INTEGRITY DEFINITIONS

1615.1 Definitions. The following terms are defined in Chapter 2:

ALTERNATE LOAD PATH.

ALTERNATE LOAD PATH METHOD.

ASPECT RATIO.

COLLAPSE (STRUCTURAL).

ELEMENT (STRUCTURAL).

KEY ELEMENT.

LOCAL COLLAPSE.

RESPONSE RATIO.

ROTATION.

SPECIFIC LOCAL LOAD.

SPECIFIC LOCAL RESISTANCE METHOD.

SECTION BC 1616
STRUCTURAL INTEGRITY—PRESCRIPTIVE REQUIREMENTS

1616.1 Scope. The intent of these provisions is to enhance structural performance under extreme event scenarios by providing additional overall system redundancy and local robustness. All structures shall be designed to satisfy the prescriptive requirements of this section.

> **Exception:** Structures in Risk Category I of Table 1604.5 and structures in Occupancy Group R-3 are exempt from the requirements of Sections 1615 through 1617.

1616.2 Continuity and ties. All structural elements shall have a minimum degree of continuity and shall be tied together horizontally and vertically as specified in Chapters 19, 21 and 22 for concrete, masonry and steel, respectively.

1616.3 Lateral bracing. Floor and roof diaphragms or other horizontal elements shall be tied to the lateral load-resisting system.

1616.4 Vehicular impact. Structural columns that are directly exposed to vehicular traffic shall be designed for vehicular impact. Structural columns that are adequately protected by bollards, guard walls, vehicle arrest devices or other elements do not need to be designed for vehicular impact. The load combinations for vehicular impact shall be as specified in Section 1605.5.

Specific loads for vehicular impact shall be as follows:

1. Exterior corner columns shall be designed for a concentrated load of 40 kips applied horizontally in any direction from which a vehicle can approach at a height of either 18 inches (457.2 mm) or 36 inches (914.4 mm) above the finished driving surface, whichever creates the worst effect.
2. All other exterior columns exposed to vehicular traffic, and columns within loading docks, and columns in parking garages along the driving lane shall be designed for a concentrated load of 20 kips applied horizontally in

any direction from which a vehicle can approach at a height of either 18 inches (457.2 mm) or 36 inches (914.4 mm) above the finished driving surface, whichever creates the worst effect.

1616.5 Gas explosions. In buildings with gas piping operating at pressures in excess of 15 psig (193.4 kPa gauge), all key elements and their connections within 15 feet (4572 mm) of such piping shall be designed to resist a potential gas explosion. The structure shall be designed to account for the potential loss of the affected key elements one at a time by the alternate load path method. Load combinations for the alternate load path shall be as specified in Section 1605.4. In lieu of the alternate load path method, the affected key elements shall be designed to withstand a load of 430 psf (20.6 kPa) applied using the load combinations specified in Section 1605.5. The load shall be applied along the entire length of the element, and shall be applied in the manner and direction that produces the most damaging effect.

Exceptions:

1. If a structural enclosure designed to resist the specified pressure is provided around the high-pressure gas piping, only the key elements within the structural enclosure need to comply with this section.
2. A reduced pressure for gas explosions can be used based on an engineering analysis approved by the commissioner.

1616.5.1 Explosion prevention and deflagration venting. The structural design and installation of explosion prevention systems and deflagration venting shall be in accordance with the requirements of Appendices E and G of the *New York City Fuel Gas Code*, as well as the *New York City Fire Code*, and the rules and regulations of the department.

SECTION BC 1617
STRUCTURAL INTEGRITY—KEY ELEMENT ANALYSIS

1617.1 Scope. A key element analysis shall be performed for the following buildings:

1. Buildings included in Risk Category IV as defined in this chapter.
2. Buildings with the aspect ratios of seven or greater.
3. Buildings greater than 600 feet (182.9 m) in height or more than 1,000,000 square feet (92 903 m^2) in gross floor area.
4. Buildings taller than seven stories where any element, except for walls greater than 10 feet (3.1 m) in length, supports in aggregate more than 15 percent of the building area.
5. Buildings designed for areas with 3,000 or more occupants in one area in close proximity, including fixed seating and grandstand areas.
6. When specifically ordered by the commissioner.

1617.2 Load combinations. Where specifically required by Section 1617.1, elements and components shall be designed to resist the forces calculated using the combination specified in Section 1605.4 or 1605.6 as applicable.

1617.3 Seismic and wind. When the code-prescribed seismic or wind design produces greater effects, the seismic or wind design shall govern, but the detailing requirements and limitations prescribed in this and referenced sections shall also be followed.

1617.4 Joints. Where a structure is divided by joints that allow for movement, each portion of the structure between joints shall be considered as a separate structure.

1617.5 Key element analysis. Where key elements are present in a structure, the structure shall be designed to account for their potential loss one at a time by the alternate load path method or by the specific local resistance method as specified in Section 1617.6.

1617.6 The specific local resistance method. Where the specific local resistance method is used, key elements shall be designed using specific local loads as follows:

1. Each compression element shall be designed for a concentrated load equal to 2 percent of its axial load but not less than 15 kips, applied at midspan in any direction, perpendicular to its longitudinal axis. This load shall be applied in combination with the full dead load and 50 percent of the live load in the compression element.
2. Each bending element shall be designed for the combination of the principal acting moments plus an additional moment, equal to 10 percent of the principal acting moment applied in the perpendicular plane.
3. Connections of each tension element shall be designed to develop the smaller of the ultimate tension capacity of the member or three times the force in the member.
4. All structural elements shall be designed for a reversal of load. The reversed load shall be equal to 10 percent of the design load used in sizing the member.

1617.7 Design criteria. Alternate load path method and/or specific local resistance method for key elements shall conform to the appropriate design criteria as determined from Sections 1617.8 through 1617.10.

1617.8 Analysis procedures. All structural analysis for specific local loads or alternate load paths shall be made by one of the following methods:

1617.8.1 Static elastic analysis. For analysis of this type, dynamic effects of member loss or dynamic effects of specific local loads need not be considered. The structural demand is obtained from linear static analysis. However, structural member capacity is based on ultimate capacity of the entire cross section. The demand/capacity ratio of structural elements shall not exceed one.

1617.8.2 Dynamic inelastic analysis. For analysis of this type, dynamic effects of member loss or specific local loads shall be considered. The structure does not need to remain elastic; however, the response ratio and rotation limits obtained from Table 1617.8.3 shall not be exceeded.

1617.8.3 Energy methods. Static inelastic analysis using energy equilibrium may also be used. The structure does not need to remain elastic; however, the response ratio and rotation limits obtained from Table 1617.8.3 shall not be exceeded.

TABLE 1617.8.3
RESPONSE RATIO AND ROTATION LIMITS

ELEMENT	RESPONSE RATIO	ROTATION
Concrete slabs	$\mu < 10$	$\theta < 4°$
Post-tensioned beams	$\mu < 2$	$\theta < 1.5°$
Concrete beams	$\mu < 20$	$\theta < 6°$
Concrete columns	$\mu < 2$	$\theta < 6°$
Long span acoustical deck	$\mu < 2$	$\theta < 3°$
Open web steel joists	$\mu < 2$	$\theta < 6°$
Steel beams	$\mu < 20$	$\theta < 10°$
Steel columns	$\mu < 5$	$\theta < 6°$

For SI: 1 degree = 0.01745 rad.

Note: Table 1617.8.3 is intended for SDOF and simplified MDOF response calculations and a low level of protection. Table 1617.8.3 does not apply for explicit finite element methods that calculate the performance of the structural elements in response to the specified loading intensity. Steel joists: downward loading 6 degrees, upward loading ductility of 2.

1617.9 Minimum response. Structural response of elements determined using a dynamic inelastic analysis shall not be less than 80 percent of the structural response determined using a static elastic analysis.

1617.10 Strength reduction factors. For structural design for specific local loads or alternate load paths, all strength reduction factors may be multiplied by 1.1.

SECTION BC 1618
STRUCTURAL PEER REVIEW

1618.1 General. The provisions of this section specify where structural peer review is required, how and by whom it is to be performed.

1618.2 Where required. A structural peer review of the primary structure shall be performed and a report provided for the following buildings:

1. Buildings included in Risk Category IV as defined in this chapter and more than 50,000 square feet (4645.2 m^2) of framed area.
2. Buildings with aspect ratios of seven or greater.
3. Buildings greater than 600 feet (183 m) in height or more than 1,000,000 square feet (92 903 m^2) in gross floor area. Rooftop structures may be excluded from building height in accordance with Section 504.3.
4. Buildings taller than seven stories where any element, except for walls greater than 10 feet (3.048 meters) in length, supports in aggregate more than 15 percent of the building area.
5. Buildings designed using nonlinear time history analysis or with special seismic energy dissipation systems.
6. Buildings designed for areas with 3,000 or more occupants in one area in close proximity, including fixed seating and grandstand areas.
7. Buildings where a structural peer review is requested by the commissioner.
8. Buildings designed by performance-based methods.

1618.3 Structural peer review. It shall be verified that the structural design is in general conformance with the requirements of this code.

‡‡‡ **1618.4 Geotechnical peer review.** Geotechnical peer reviews are required to be performed in accordance with Section 1818‡.

1618.5 Structural peer reviewer. The structural peer review shall be performed by a qualified independent structural engineer who has been retained by or on behalf of the owner. A structural peer reviewer shall meet the requirements of the rules of the department.

1618.5.1 Independence. In order to satisfy the requirement that the peer reviewer be independent, the department requires that the peer reviewer must not engage in any activities that may conflict with their objective judgment and integrity, including but not limited to having a financial and/or other interest in the design, construction, installation, manufacture or maintenance of structures or components that they are reviewing.

1618.6 Extent of the structural peer review. The structural peer review shall comply with Sections 1618.6.1 and 1618.6.2.

1618.6.1 Scope. The structural peer reviewer shall review the plans and specifications submitted with the permit application for general compliance with the structural and foundation design provisions of this code. The reviewing engineer shall perform the following tasks at a minimum:

1. Confirm that the design loads conform to this code.
2. Confirm that other structural design criteria and design assumptions conform to this code and are in accordance with generally accepted engineering practice.
3. Review geotechnical and other engineering investigations that are related to the foundation and structural design and confirm that the design properly incorporates the results and recommendations of the investigations.
4. Review the structural frame and the load supporting parts of floors, roofs, walls, shallow foundation elements, deep foundation caps, connection of deep foundation elements to deep foundation caps, and permanent rock or soil anchor connection to the structural elements. Cladding, cladding framing, stairs, equipment supports, ceiling supports, non-loadbearing partitions, railings and guards, and other secondary structural items shall be excluded.
5. Confirm that the structure has a complete load path.

6. Perform independent calculations for a representative fraction of systems, members, and details to check their adequacy. The number of representative systems, members, and details verified shall be sufficient to form a basis for the reviewer's conclusions.
7. Verify that performance-specified structural components (such as certain precast concrete elements) have been appropriately specified and coordinated with the primary building structure.
8. Verify that the design engineer of record complied with the structural integrity provisions of the code.
9. Review the structural and architectural plans for the building. Confirm that the structural plans are in general conformance with the architectural plans regarding loads and other conditions that may affect the structural design.
10. Confirm that major mechanical items are accommodated in the structural plans.
11. Attest to the general completeness of the structural plans and specifications.
12. Review performance based design.

1618.6.2 Structural design criteria. If the design criteria and design assumptions are not shown on the drawings or in the computations, the structural engineer of record shall provide a statement of these criteria and assumptions for the reviewer. In addition, the design engineer shall provide information and/or calculations, if requested by the peer reviewer.

1618.7 Structural peer review report. The structural peer review report shall comply with Sections 1618.7.1 through 1618.7.3.

1618.7.1 General. The reviewing engineer shall submit a report to the department stating whether or not the structural design shown on the plans and specifications generally conforms to the structural and foundation requirements of this code.

1618.7.2 Contents. The report shall demonstrate, at a minimum, compliance with Items 1 through 12 of Section 1618.6.1. In addition, the report shall also include the following:

1. A list of construction documents, including the latest revision dates submitted to the department for approval, as well as geotechnical reports, wind tunnel studies and other reports utilized in the design.
2. The codes and standards used in the structural design of the project.
3. The structural design criteria, including loads and performance requirements.
4. The basis for design criteria that are not specified directly in applicable codes and standards. This should include reports by specialty consultants such as wind tunnel study reports and geotechnical reports. Generally, the report should confirm that existing conditions at the site have been investigated as appropriate and that the design of the proposed structure is in general conformance with these conditions.
5. The basis of performance-based design and its appropriateness of use.
6. A statement that the qualification and independence of the peer reviewer is in accordance with Section 1618.5.
7. Peer review reports shall provide an unconditional statement that must be either an acceptance or rejection of code compliance of the structural design required to be reviewed by this section.
8. The structural peer review reports shall contain a statement that geotechnical peer review reports and/or wind tunnel peer review reports, when required, have been received with an unconditional statement of acceptance by the authors of those reports.

1618.7.3 Phased submission. If an application is submitted for a permit for the construction of foundations or any other part of a building before the construction documents for the whole building have been submitted, then the structural peer review and report shall be phased. The structural peer reviewer shall be provided with sufficient information on which to make a structural peer review of the phased submission. In addition to the requirements of this section, phased submission of the report shall include any changes made to the initial structural peer review evaluation. The geotechnical peer review report need not be submitted concurrently with the structural peer review report.

1618.8 Responsibility. The responsibilities of the engineer of record and peer reviewer shall be as laid out in Sections 1618.8.1 through 1618.8.4.

1618.8.1 Structural engineer of record. The structural engineer of record shall retain sole responsibility for the structural design. The activities and reports of the structural peer reviewer shall not relieve the structural engineer of record of this responsibility.

1618.8.2 Structural peer reviewer. The structural peer reviewer's report states his or her opinion regarding the design by the engineer of record. The standard of care to which the structural peer reviewer shall be held in the performance of the structural peer review and report is that level of skill and care consistent with structural peer review services performed by professional engineers licensed in the State of New York for similar type projects. The structural peer reviewer shall not be responsible for the conclusions of the wind and geotechnical peer review reports.

1618.8.3 Revisions of design. The structural engineer of record shall identify that a new review is required based upon changes to the design documents relevant to the scope of the peer review. The structural peer reviewer shall review the items that have been affected and shall revise the structural peer review report.

1618.8.4 Disputes. When a dispute arises between the applicant of record and the peer reviewer regarding compliance with Section 1618 that cannot be resolved by these parties, such dispute shall be reported to the department in the form of a letter from the applicant of record. The department may make a final decision or may

accept a change of the peer reviewer.

1618.9 Change to peer reviewer. The peer reviewer cannot be changed without the express consent of the department. The current peer reviewer must submit both a written letter to the borough commissioner detailing the reason for the withdrawal request and a report of the structural peer review findings to date. If a change of peer reviewer is approved, a revised submission indicating the replacement peer reviewer shall be filed with the department.

SECTION 1619
TEMPORARY STRUCTURES AND TEMPORARY CONSTRUCTION INSTALLATIONS

1619.1 General. Temporary construction installations that are required by this code to be designed by a registered design professional, and all temporary structures, shall comply with all the provisions of this code, except as described in Sections 1619.3 and 1619.4.

1619.2 Definitions. The following terms are defined in Chapter 2:

TEMPORARY CONSTRUCTION INSTALLATIONS.

TEMPORARY STRUCTURES.

1619.3 Temporary construction installations. Temporary construction installations shall be subject to the requirements of this section and Chapter 33.

1619.3.1 Design drawings for temporary construction installations. Design loads, including location and intensity of live loads, shall be indicated on the design drawings for temporary construction installations. Drawings for multilevel temporary construction installations shall indicate the maximum number of levels to be loaded simultaneously and the maximum design loads. Any temporary construction installation utilizing authorized exemptions and load reductions in the structural design shall be indicated on drawings as temporary, and all reduced loads shall be indicated on the drawings. The environmental load mitigations shall be indicated on the design drawings. The design drawings shall be maintained at the site of the temporary construction installation and be made available to the department upon request.

1619.3.2 Loads for temporary construction installations. Temporary construction installations shall be designed and constructed to resist the loads and load combinations required by this chapter for permanent construction and Chapter 33 of the building code. Live loads shall not be less than the code prescribed value or the maximum loads for the use of a temporary construction installation as noted on the design drawings.

Exceptions:

1. Load reductions may be utilized where permitted by Section 1619.3.3.

2. Derricks not permanently mounted to a building shall not be subject to the loads and load combinations of this chapter. Such derricks shall instead be subject to the requirements of Section 3319 and rules promulgated by the commissioner. However, temporary construction installations used to support such derricks shall be subject to the loads and load combinations of this chapter.
3. Mobile cranes shall not be subject to the loads and load combinations of this chapter. Mobile cranes shall instead be subject to the requirements of Section 3319 and rules promulgated by the commissioner. However, temporary construction installations used to support mobile cranes shall be subject to the loads and load combinations of this chapter.

1619.3.3 Load reductions for temporary construction installations. Temporary construction installations shall be permitted to reduce the design environmental loads required by this chapter in accordance with the specifications below. An action plan for load reductions shall be required in accordance with Section 1619.5.

1. Seismic resistance. Temporary construction installations do not need to consider seismic design. These installations, other than supports of excavation, shall be designed for a minimum of 2 percent of the dead and live load as a service level lateral force. This load shall be used in lieu of seismic forces required by Section 1613 in load combinations including seismic forces. This load shall be distributed in proportion to the design loads, applied in any horizontal direction, and need not be combined with other environmental loads.
2. Wind. The wind design for temporary construction installations shall be computed as required by Section 1609. The basic wind speed used to design the temporary construction installation shall be permitted to be reduced by applying a factor of 0.8.

   Exceptions: The following are exceptions to item 2 concerning load reductions for wind design:

   1. Mast climbers shall not be permitted to reduce wind loads.
   2. Suspended scaffolds shall be designed for an out of service wind speed of at least 45 mph (72.4 kph), or a lower speed when so specified by the scaffold manufacturer.
   3. Other environmental forces. Other environmental forces, including but not limited to snow, flood, ice, and temperature differential effects, shall be permitted to be reduced as appropriate for the limited seasonal exposure of the temporary construction installation.

1619.3.4 Permit renewal for temporary construction installations that utilize an action plan. Where a temporary construction installation requires an action plan in accordance with Section 1619.5, the application to renew the permit for such temporary construction installation shall be accompanied by the submission of a report from a registered design professional certifying, based on an inspection of the temporary construction installation performed by the registered design professional within 30 days prior to the submittal of the permit renewal application, that the action plan:

1. Is still in effect;
2. Has been revised to reflect the current conditions of the temporary structure; or
3. Is no longer required, as the temporary structure has been retrofitted to comply with the loads for new construction without any reduction, pursuant to Section 1619.3.2.

1619.4 Temporary structures. Temporary structures shall be subject to the requirements of Section 3103.1 and this section.

1619.4.1 Design drawings for temporary structures. Design loads, including location and intensity of live loads, shall be indicated on the design drawings for temporary structures. Drawings for multilevel temporary structures shall indicate the maximum number of levels to be loaded simultaneously and the maximum design loads. Any temporary structure utilizing authorized exemptions and load reductions in the structural design shall be indicated on drawings as temporary, and all reduced loads shall be indicated on the drawings. The environmental load mitigations shall be indicated on the design drawings. The design drawings shall be maintained at the site of the temporary structure and be made available to the department upon request.

1619.4.2 Loads for temporary structures. Temporary structures shall be designed and constructed to resist the loads and load combinations required by this chapter. Live loads shall not be less than the code prescribed value or the maximum loads for the use of a temporary structure as noted on the design drawings.

Exception: Load reductions may be utilized where permitted by Section 1619.4.3.

1619.4.3 Load reductions for temporary structures. Temporary structures shall be permitted to reduce the design environmental loads required by this chapter in accordance with the specifications below. An action plan for load reductions shall be required in accordance with Section 1619.5.

1. Seismic resistance. Temporary structures do not need to consider seismic design. Temporary structures shall be designed for a minimum of 2 percent of the dead and live load as a service level lateral force. This load shall be used in lieu of seismic forces required by Section 1613 in load combinations including seismic forces. This load shall be distributed in proportion to the design loads, applied in any horizontal direction, and need not be combined with other environmental loads.

2. Wind. The wind design for temporary structures shall be computed as required by Section 1609. The basic wind speed used to design the structure shall be permitted to be reduced by applying a factor of 0.8.

   Exception: In lieu of applying a factor of 0.8, temporary structures may be removed from service at the environmental conditions determined by the manufacturer of the temporary structure. The threshold for removal of service shall be indicated on the design drawings required by Section 1619.4.1 as well as the action plan required by Section 1619.5.

3. Other environmental forces. Other environmental forces, including but not limited to snow, flood, ice, and temperature differential effects, shall be permitted to be reduced as appropriate for the limited seasonal exposure of the temporary structure.

1619.4.4 Duration of time for temporary structures. Temporary structures shall be erected for a limited period not exceeding ninety days in accordance with Article 111 of Title 28 of the *Administrative Code*. Such limited period shall be counted from the date the temporary structure is substantially completed.

Exception: Where the commissioner has approved a request to extend the duration of time for a temporary structure in accordance with Section 1619.4.5.

1619.4.5 Extension of time for temporary structures. Subject to the approval of the commissioner, the duration of time for a temporary structure may be extended by increments not to exceed ninety days. Each request to extend the time for a temporary structure shall be accompanied by the submission of a report from a registered design professional certifying the following:

1. Such registered design professional performed an inspection within the last 30 days to confirm that the temporary structure complies with the requirements of the approved construction documents for the temporary structure; and

2. Where an action plan is utilized in accordance with Section 1619.5:

   2.1. It is still in effect;

   2.2. It has been revised to reflect the current conditions of the temporary structure; or

   2.3. It is no longer required, as the temporary structure has been retrofitted to comply with the loads for new construction without any reduction, pursuant to Section 1619.4.2.

‡‡‡ **1619.5 Action plan.** All temporary structures or temporary construction installations with reduced design environmental loads in accordance with Section 1619.3.3 or 1619.4.3 shall include environmental load mitigation measures as part of an action plan to protect the public. The action plan measures shall be indicated on the design drawings required by Section 1619.3.1 or 1619.4.1.

Exception: An action plan is not required for:

1. Derricks not permanently mounted to a building. Derricks not permanently mounted to a building shall be secured in accordance with a wind action plan as specified in rules promulgated by the commissioner.

2. Mobile cranes. Mobile cranes shall be secured in accordance with a wind action plan as specified in rules promulgated by the commissioner.

3. A suspended scaffold. Suspended scaffolds shall be secured in accordance with the requirements of Section 3314; requirements for securing suspended scaffolds shall be indicated on design drawings when required by Section 3314.

1619.5.1 Implementation. The action plan shall be such that it can be reliably implemented within one day of notice or less as appropriate for the actions.

1619.5.2 Components. The action plan shall, at a minimum, include the following:

1. Threshold of predicted environmental loads;
2. Method of monitoring environmental loads;
3. Name of party responsible for monitoring loads and determining implementation of action plan;
4. Name of party responsible for implementing the action plan;
5. Evacuation procedures;
6. Safety zone, standoff distance or standoff perimeter as appropriate. Safety zone, standoff distance or standoff perimeter shall not extend beyond the property line;
7. Any other activities, such as the addition or removal of structural and/or nonstructural elements, removal of loads or creating sacrificial elements so that the structure may resist unreduced forces as required for permanent structures;
8. Plan to prevent wind-born debris; and
9. Verification that the design and procedures shall not adversely impact other structures.