

780 CMR 16.00

STRUCTURAL DESIGN

780 CMR 16.00 is unique to Massachusetts

780 CMR 1601.0 GENERAL

1601.1 Scope. Provisions of 780 CMR 16.00 shall govern the structural design of buildings, structures and portions thereof regulated by 780 CMR.

1601.2 Massachusetts Building Code for One- and Two Family Dwellings. Where structural design for buildings covered by the Massachusetts Building Code for One- and Two Family Dwellings is performed by a registered design professional (RDP), said RDP may follow the structural design provisions of the Massachusetts Basic Building Code in lieu of those in the Building Code for One- and Two Family Dwellings, and shall follow the Basic Building Code for the structural design of components and systems not addressed in the Building Code for One- and Two Family Dwellings.

780 CMR 1602.0 DEFINITIONS AND NOTATIONS

1602.1 Definitions. The following words and terms shall, for the purposes of 780 CMR 16.00, have the meanings shown in 780 CMR 1602.0. For definitions not contained in 780 CMR 1602.0, see appropriate sections of ASCE 7.

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

BALCONY. A floor projecting from and supported by a structure without additional independent supports.

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

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

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

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

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

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

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

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

GUARD. See 780 CMR 1002.1.

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

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

LATERAL FORCE-RESISTING SYSTEM. That part of the structural system that is considered in the design to provide resistance to the wind and seismic forces prescribed in 780 CMR.

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

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

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

LOAD AND RESISTANCE FACTOR DESIGN (LRFD). A method of proportioning structural members and their connections using load and resistance factors such that no applicable limit state is reached when the structure is subjected to appropriate load combinations. (Also called “strength design”).

THE MASSACHUSETTS STATE BUILDING CODE

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

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

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

NOMINAL LOADS. The magnitudes of the loads specified in 780 CMR 16.00 (dead, live, soil, wind, snow, rain, flood and earthquake).

NOTATIONS:

- D = Dead load.
- E = Combined effect of horizontal and vertical earthquake- induced forces.
- F = Load due to fluids.
- F_a = Flood load.
- H = Load due to lateral pressure of soil and water in soil.
- L = Live load, except roof live load, including any permitted live load reduction.
- L_r = Roof live load including any permitted live load reduction.
- R = Rain load.
- S = Snow load.
- T = Self-straining force arising from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep in component materials, movement due to differential settlement, or combinations thereof.
- W = Load due to wind pressure.

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

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

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

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

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

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

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

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

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

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

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

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

780 CMR 1603.0 CONSTRUCTION DOCUMENTS

1603.1 General. Construction documents shall show the size, section and relative locations of structural members with floor levels, column centers and offsets fully dimensioned. The design loads and other information pertinent to the structural design required by 780 CMR 1603.1.1 through 1603.1.8 shall be clearly indicated on the construction documents for parts of the building or structure.

1603.1.1 Floor Live Load. The uniformly distributed, concentrated, and impact floor live loads used in the design and where they are applied shall be indicated. Also, whether live load reduction has been applied, shall be indicated.

1603.1.2 Roof Live Load. The roof live load used in the design shall be indicated (780 CMR 1607.11).

1603.1.3 Roof Snow Load. The ground snow load, p_g , and the flat-roof snow load, p_f , and, if applicable, the sloped roof snow load, p_s , shall be indicated.

1603.1.4 Wind Load. 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 building:

1. Basic wind speed (three-second gust), miles per hour (km/hr).
2. Wind importance factor, I_w , and building category.
3. Wind exposure. If more than one wind exposure is utilized, the wind exposures and applicable wind directions shall be indicated.

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

1. Seismic importance factor, I_E , and seismic use group.
2. Mapped spectral response accelerations S_S and S_I .
3. Site class.
4. Spectral response coefficients S_{DS} and S_{DI} .
5. Seismic design category.
6. Basic seismic-force-resisting system(s).
7. Design base shear.
8. Seismic response coefficient(s), C_S .
9. Response modification factor(s), R .
10. Analysis procedure used.

Where the information along different horizontal axes of the building is different, the information for both orthogonal axes shall be show.

1603.1.6 Flood Load. For buildings located in flood-hazard areas, the design flood elevation and whether or not the construction is subject to high-velocity wave action shall be indicated. The following information, referenced to the datum on the community's flood insurance rate map (FIRM), shall be shown, regardless of whether flood loads govern the design of the building:

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

1603.1.7 Special Loads. Special loads that are applicable to the design of the building, structure or portions thereof shall be indicated.

1603.2 Structural Designs under the Control of the Construction Contractor. When structural components, assemblies, or systems are designed by design professionals under the control of the contractor, and said designs are not included with the application for permit, said designs shall be

submitted to the building official with an application for amendment of the permit.

780 CMR 1604.0 GENERAL DESIGN REQUIREMENTS

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

1604.2 Strength. Buildings and other structures, and parts thereof, shall be designed and constructed to support safely the factored loads in load combinations defined in 780 CMR 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 780 CMR without exceeding the appropriate specified allowable stresses for the materials of construction.

Loads and forces for occupancies or uses not covered in 780 CMR 16.00 shall be subject to the approval of the building official.

1604.3 Serviceability. Structural systems and members thereof shall be designed to have adequate stiffness to limit deflections and lateral drift.

1604.3.1 Deflections. The deflections of structural members shall not exceed the limitations of 780 CMR 1604.3.2 through 1604.3.5, as applicable; however, under no circumstance shall the deflections from gravity loads on floors or roofs exceed 1/240th of the span, nor shall the deflections of walls due to lateral loads exceed 1/180th of the span.

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

1604.3.3 Steel. The deflection of steel structural members shall not exceed that permitted by AISC LRFD, AISC HSS, AISC 335, AISI -NASPEC, AISI-General, AISI-Truss, ASCE 3, ASCE 8-SSD-LRFD/ASD, and the standard specifications of SJI Standard Specifications, Load Tables and Weight Tables for Steel Joists and Joist Girders as applicable.

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

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

1604.4 Analysis. Load effects on structural members and their connections shall be determined by methods of structural analysis that take into account

THE MASSACHUSETTS STATE BUILDING CODE

equilibrium, general stability, geometric compatibility, and both short- and long-term material properties.

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

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

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

Every structure shall be designed to resist the overturning effects caused by the lateral forces specified in 780 CMR 16.00. See 780 CMR 1613 for earthquake, 780 CMR 1609.0 for wind, and 780 CMR 1610.0 for lateral soil loads.

1604.5 Importance Factors. Importance factors shall be in accordance with ASCE 7.

1604.6 In-situ Load Tests. The building official is authorized to require an engineering analysis or a load test, or both, of any construction whenever there is reason to question the safety of the construction for the intended occupancy. Engineering analysis and load tests shall be conducted in accordance with 780 CMR 1624.0.

1604.7 Preconstruction Load Tests. Materials and methods of construction that are not capable of being designed by approved engineering analysis or that do not comply with the applicable material design standards listed in 780 CMR 35.00, or alternative test procedures in accordance with 780 CMR 1625.0, shall be load tested in accordance with 780 CMR 1626.0.

1604.8 Overturning, Sliding, and Anchorage.

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

1604.8.2 Concrete and Masonry Walls. Concrete and masonry walls shall be connected to floors, roofs and other structural elements that provide lateral support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces specified in 780 CMR 16.00 but not less than a minimum horizontal force of 190 pounds per linear foot (2.77 kN/m) of wall for allowable stress design, and 280 pounds per linear foot (4.09 kN/m) of wall for strength design. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds four feet (1219 mm). Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall.

1604.9 Application and Posting of Live Loads.

1604.9.1 Restrictions on Loading. It shall be unlawful to place, or cause or permit to be placed, on any floor or roof of a building, structure, or portion thereof, a load greater than the capacity of the floor or roof determined in accordance with the requirements of 780 CMR.

1604.9.2 Live Loads Posted: All buildings in Use Groups F and S shall be conspicuously posted for live loads by the owner, using durable signs. It shall be unlawful to remove or deface such notices.

1604.10 Snow, Wind, and Earthquake Design Factors: Ground snow load, p_g , basic wind speed (three second gust speed), V , and earthquake response accelerations for the maximum considered earthquake, S_s and S_1 , for each city and town in Massachusetts shall be as given in Table 1604.10.

NOTE: Table 1604.10 Ground Snow Loads; Basic Wind Speeds; Earthquake Design Factors applies, as applicable, to all use groups except for R-3 One- and Two-family dwellings.

For ground snow loads and basic wind speeds for R-3 of three stories or less one- and two-family stand alone dwellings, see 780 CMR 53.00.

TABLE 1604.10 GROUND SNOW LOADS; BASIC WIND SPEEDS; EARTHQUAKE DESIGN FACTORS

(For R-3 of three stories or less one- and two-family stand alone buildings, see 780 CMR 53.00 for snow and wind loads)

City/Town	Ground Snow Load p _g , psf	Basic Wind Speed V, MPH	Earthquake Design Factors	
			S _s	S ₁
Abington	45	110	0.26	0.064
Acton	55	100	0.29	0.071
Acushnet	45	110	0.23	0.058
Adams	65	90	0.22	0.068
Agawam	55	100	0.23	0.065
Alford	65	90	0.22	0.066
Amesbury	55	110	0.35	0.077
Amherst	55	100	0.23	0.067
Andover	55	110	0.32	0.075
Aquinnah (see Gay Head)				
Arlington	45	105	0.29	0.069
Ashburnham	65	100	0.27	0.072
Ashby	65	100	0.28	0.072
Ashfield	65	100	0.22	0.068
Ashland	55	100	0.25	0.066
Athol	65	100	0.25	0.070
Attleboro	55	110	0.24	0.062
Auburn	55	100	0.23	0.065
Avon	55	100	0.26	0.064
Ayer	65	100	0.28	0.071
Barnstable	35	120	0.20	0.054
Barre	55	100	0.24	0.068
Becket	65	90	0.22	0.066
Bedford	55	100	0.29	0.071
Belchertown	55	100	0.23	0.066
Bellingham	55	100	0.24	0.064
Belmont	45	105	0.28	0.069
Berkley	55	110	0.24	0.061
Berlin	55	100	0.26	0.068
Bernardston	65	100	0.23	0.070
Beverly	45	110	0.32	0.072
Billerica	55	100	0.30	0.072
Blackstone	65	100	0.24	0.064
Blandford	65	100	0.23	0.066
Bolton	55	100	0.26	0.069
Boston	45	105	0.29	0.068
Bourne	35	120	0.21	0.056
Boxborough	55	100	0.28	0.070
Boxford		110	0.33	0.075
Boylston	55	100	0.25	0.067
Braintree	45	105	0.27	0.066
Brewster	35	120	0.18	0.052
Bridgewater	45	110	0.24	0.062
Brimfield	55	100	0.23	0.065
Brockton	45	110	0.25	0.064
Brookfield	55	100	0.23	0.065
Brookline	45	105	0.28	0.068
Buckland	65	100	0.22	0.068
Burlington	55	105	0.30	0.071

THE MASSACHUSETTS STATE BUILDING CODE

TABLE 1604.10 GROUND SNOW LOADS; BASIC WIND SPEEDS; EARTHQUAKE DESIGN FACTORS

(For R-3 of three stories or less one- and two-family stand alone buildings, see 780 CMR 53.00 for snow and wind loads)

City/Town	Ground Snow Load p _g , psf	Basic Wind Speed V, MPH	Earthquake Design Factors	
			S _s	S ₁
Cambridge	45	105	0.28	0.068
Canton	55	100	0.26	0.066
Carlisle	55	100	0.29	0.071
Carver	45	110	0.24	0.060
Charlemont	65	100	0.22	0.068
Charlton	55	100	0.23	0.065
Chatham	35	120	0.17	0.050
Chelmsford	55	100	0.30	0.073
Chelsea	45	105	0.29	0.069
Cheshire	65	90	0.22	0.068
Chester	65	100	0.22	0.066
Chesterfield	65	100	0.22	0.067
Chicopee	55	100	0.23	0.066
Chilmark	35	120	0.18	0.051
Clarksburg	65	90	0.22	0.069
Clinton	55	100	0.26	0.068
Cohasset	45	110	0.27	0.066
Colrain	65	100	0.23	0.069
Concord	55	100	0.29	0.070
Conway	65	100	0.22	0.068
Cummington	65	100	0.22	0.067
Dalton	65	90	0.22	0.067
Danvers	45	110	0.32	0.073
Dartmouth	45	110	0.23	0.058
Dedham	55	100	0.26	0.066
Deerfield	65	100	0.23	0.068
Dennis	35	120	0.19	0.052
Dighton	55	110	0.24	0.061
Douglas	55	100	0.23	0.064
Dover	55	100	0.26	0.066
Dracut	55	100	0.33	0.075
Dudley	55	100	0.23	0.064
Dunstable	65	100	0.31	0.074
Duxbury	45	110	0.25	0.062
East Bridgewater	45	110	0.25	0.063
East Brookfield	55	100	0.23	0.066
East Longmeadow	55	100	0.23	0.065
Eastham	35	120	0.19	0.052
Easthampton	55	100	0.23	0.066
Easton	55	110	0.25	0.064
Edgartown	35	120	0.18	0.050
Egremont	65	90	0.23	0.066
Erving	65	100	0.23	0.069
Essex	45	110	0.33	0.073
Everett	45	105	0.29	0.069
Fairhaven	45	110	0.22	0.057
Fall River	45	110	0.23	0.059
Falmouth	35	120	0.20	0.054
Fitchburg	65	100	0.27	0.071
Florida	65	90	0.22	0.069

TABLE 1604.10 GROUND SNOW LOADS; BASIC WIND SPEEDS; EARTHQUAKE DESIGN FACTORS

(For R-3 of three stories or less one- and two-family stand alone buildings, see 780 CMR 53.00 for snow and wind loads)

City/Town	Ground Snow Load p _g , psf	Basic Wind Speed V, MPH	Earthquake Design Factors	
			S _s	S ₁
Foxborough	55	100	0.25	0.064
Framingham	55	100	0.26	0.067
Franklin	55	100	0.24	0.064
Freetown	45	110	0.23	0.060
Gardner	65	100	0.26	0.070
Gay Head (a.k.a Aquinnah)	35	120	0.18	0.051
Georgetown	55	110	0.34	0.075
Gill	65	100	0.23	0.069
Gloucester	45	110	0.33	0.073
Goshen	65	100	0.22	0.067
Gosnold	35	120	0.19	0.053
Grafton	55	100	0.24	0.066
Granby	55	100	0.23	0.066
Granville	65	100	0.23	0.066
Great Barrington	65	90	0.22	0.066
Greenfield	65	100	0.23	0.069
Groton	65	100	0.30	0.073
Groveland	55	110	0.34	0.076
Hadley	55	100	0.23	0.067
Halifax	45	110	0.25	0.062
Hamilton	45	110	0.33	0.074
Hampden	55	100	0.23	0.065
Hancock	65	90	0.22	0.068
Hanover	45	110	0.26	0.064
Hanson	45	110	0.25	0.063
Hardwick	55	100	0.23	0.067
Harvard	55	100	0.28	0.070
Harwich	35	120	0.18	0.051
Hatfield	55	100	0.22	0.067
Haverhill	55	110	0.35	0.077
Hawley	65	100	0.22	0.068
Heath	65	100	0.22	0.069
Hingham	45	110	0.27	0.066
Hinsdale	65	90	0.22	0.067
Holbrook	45	105	0.26	0.065
Holden	55	100	0.25	0.068
Holland	55	100	0.23	0.064
Holliston	55	100	0.25	0.066
Holyoke	55	100	0.23	0.066
Hopedale	55	100	0.24	0.065
Hopkinton	55	100	0.25	0.066
Hubbardston	65	100	0.25	0.069
Hudson	55	100	0.26	0.068
Hull	45	110	0.28	0.067
Huntington	65	100	0.22	0.066
Ipswich	45	110	0.34	0.074
Kingston	45	110	0.24	0.061
Lakeville	45	110	0.24	0.061
Lancaster	55	100	0.27	0.070
Lanesborough	65	90	0.22	0.068

THE MASSACHUSETTS STATE BUILDING CODE

TABLE 1604.10 GROUND SNOW LOADS; BASIC WIND SPEEDS; EARTHQUAKE DESIGN FACTORS

(For R-3 of three stories or less one- and two-family stand alone buildings, see 780 CMR 53.00 for snow and wind loads)

City/Town	Ground Snow Load p _g , psf	Basic Wind Speed V, MPH	Earthquake Design Factors	
			S _s	S ₁
Lawrence	55	110	0.33	0.075
Lee	65	90	0.22	0.066
Leicester	55	100	0.24	0.066
Lenox	65	90	0.22	0.067
Leominster	65	100	0.26	0.070
Leverett	65	100	0.23	0.068
Lexington	55	105	0.29	0.070
Leyden	65	100	0.23	0.069
Lincoln	55	100	0.28	0.069
Littleton	55	100	0.29	0.071
Longmeadow	55	100	0.23	0.065
Lowell	55	100	0.31	0.074
Ludlow	55	100	0.23	0.066
Lunenburg	65	100	0.28	0.071
Lynn	45	110	0.31	0.071
Lynnfield	45	110	0.31	0.072
Malden	45	105	0.29	0.069
Manchester	45	110	0.32	0.072
Mansfield	55	110	0.25	0.063
Marblehead	45	110	0.31	0.071
Marion	45	110	0.22	0.057
Marlborough	55	100	0.26	0.068
Marshfield	45	110	0.26	0.064
Mashpee	35	120	0.20	0.054
Mattapoissett	45	110	0.22	0.057
Maynard	55	100	0.27	0.069
Medfield	55	100	0.25	0.065
Medford	45	105	0.29	0.070
Medway	55	100	0.25	0.065
Melrose	45	105	0.30	0.070
Mendon	55	100	0.24	0.064
Merrimac	55	110	0.35	0.077
Methuen	55	110	0.34	0.076
Middleborough	45	110	0.24	0.061
Middlefield	65	100	0.22	0.066
Middleton	45	110	0.32	0.073
Milford	55	100	0.24	0.065
Millbury	55	100	0.24	0.065
Millis	55	100	0.25	0.065
Millville	55	100	0.24	0.064
Milton	45	105	0.27	0.066
Monroe	65	100	0.22	0.069
Monson	55	100	0.23	0.065
Montague	65	100	0.23	0.068
Monterey	65	90	0.22	0.066
Montgomery	65	100	0.23	0.066
Mount Washington	65	90	0.23	0.066
Nahant	45	110	0.30	0.070
Nantucket	35	120	0.15	0.047
Natick	55	100	0.26	0.067

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(For R-3 of three stories or less one- and two-family stand alone buildings, see 780 CMR 53.00 for snow and wind loads)

City/Town	Ground Snow Load p _g , psf	Basic Wind Speed V, MPH	Earthquake Design Factors	
			S _s	S ₁
Needham	55	100	0.27	0.067
New Ashford	65	90	0.22	0.068
New Bedford	45	110	0.23	0.058
New Braintree	55	100	0.23	0.067
New Marlborough	65	90	0.23	0.066
New Salem	65	100	0.24	0.068
Newbury	55	110	0.35	0.076
Newburyport	55	110	0.35	0.077
Newton	55	105	0.27	0.068
Norfolk	55	100	0.25	0.065
North Adams	65	90	0.22	0.069
North Andover	55	110	0.33	0.075
North Attleborough	55	110	0.24	0.063
North Brookfield	55	100	0.23	0.066
North Reading	55	105	0.32	0.073
Northampton	55	100	0.22	0.066
Northborough	55	100	0.25	0.067
Northbridge	55	100	0.24	0.065
Northfield	65	100	0.24	0.070
Norton	55	110	0.24	0.063
Norwell	45	110	0.26	0.064
Norwood	55	100	0.26	0.065
Oak Bluffs	35	120	0.18	0.051
Oakham	55	100	0.24	0.067
Orange	65	100	0.24	0.070
Orleans	35	120	0.18	0.051
Otis	65	90	0.23	0.066
Oxford	55	100	0.23	0.065
Palmer	55	100	0.23	0.066
Paxton	55	100	0.24	0.067
Peabody	45	110	0.31	0.072
Pelham	55	100	0.23	0.067
Pembroke	45	110	0.25	0.063
Pepperell	65	100	0.30	0.073
Peru	65	90	0.22	0.067
Petersham	65	100	0.24	0.068
Phillipston	65	100	0.24	0.069
Pittsfield	65	90	0.22	0.067
Plainfield	65	100	0.22	0.068
Plainville	55	100	0.24	0.063
Plymouth	45	110	0.24	0.060
Pympton	45	110	0.24	0.061
Princeton	65	100	0.25	0.069
Provincetown	35	120	0.22	0.058
Quincy	45	105	0.27	0.067
Randolph	45	105	0.26	0.065
Raynham	55	110	0.24	0.062
Reading	55	105	0.31	0.072
Rehoboth	55	110	0.24	0.062
Revere	45	105	0.30	0.070
Richmond	65	90	0.22	0.067

THE MASSACHUSETTS STATE BUILDING CODE

TABLE 1604.10 GROUND SNOW LOADS; BASIC WIND SPEEDS; EARTHQUAKE DESIGN FACTORS

(For R-3 of three stories or less one- and two-family stand alone buildings, see 780 CMR 53.00 for snow and wind loads)

City/Town	Ground Snow Load p _g , psf	Basic Wind Speed V, MPH	Earthquake Design Factors	
			S _s	S ₁
Rochester	45	110	0.23	0.059
Rockland	45	110	0.26	0.064
Rockport	45	110	0.33	0.073
Rowe	65	100	0.22	0.069
Rowley	55	110	0.34	0.075
Royalston	65	100	0.25	0.070
Russell	65	100	0.23	0.066
Rutland	55	100	0.24	0.068
Salem	45	110	0.31	0.071
Salisbury	55	110	0.35	0.077
Sandisfield	65	90	0.23	0.066
Sandwich	35	120	0.22	0.058
Saugus	45	110	0.30	0.070
Savoy	65	90	0.22	0.068
Scituate	45	110	0.27	0.065
Seekonk	55	110	0.24	0.062
Sharon	55	100	0.25	0.065
Sheffield	65	90	0.23	0.066
Shelburne	65	100	0.23	0.068
Sherborn	55	100	0.26	0.066
Shirley	65	100	0.28	0.072
Shrewsbury	55	100	0.25	0.067
Shutesbury	65	100	0.23	0.068
Somerset	55	110	0.23	0.060
Somerville	45	105	0.28	0.069
South Hadley	55	100	0.23	0.066
Southampton	55	100	0.23	0.066
Southborough	55	100	0.26	0.067
Southbridge	55	100	0.23	0.064
Southwick	55	100	0.23	0.065
Spencer	55	100	0.23	0.066
Springfield	55	100	0.23	0.065
Sterling	55	100	0.26	0.069
Stockbridge	65	90	0.22	0.066
Stoneham	45	105	0.30	0.071
Stoughton	55	100	0.26	0.065
Stow	55	100	0.27	0.069
Sturbridge	55	100	0.23	0.065
Sudbury	55	100	0.27	0.069
Sunderland	65	100	0.23	0.068
Sutton	55	100	0.24	0.065
Swampscott	45	110	0.30	0.070
Swansea	55	110	0.24	0.061
Taunton	55	110	0.24	0.062
Templeton	65	100	0.25	0.070
Tewksbury	55	100	0.31	0.073
Tisbury	35	120	0.18	0.052
Tolland	65	100	0.23	0.066
Topsfield	45	110	0.33	0.074
Townsend	65	100	0.28	0.072
Truro	35	120	0.22	0.057

TABLE 1604.10 GROUND SNOW LOADS; BASIC WIND SPEEDS; EARTHQUAKE DESIGN FACTORS

(For R-3 of three stories or less one- and two-family stand alone buildings, see 780 CMR 53.00 for snow and wind loads)

City/Town	Ground Snow Load p _g , psf	Basic Wind Speed V, MPH	Earthquake Design Factors	
			S _s	S ₁
Tyngsborough	55	100	0.31	0.074
Tyringham	65	90	0.22	0.066
Upton	55	100	0.24	0.065
Uxbridge	55	100	0.24	0.064
Wakefield	45	105	0.31	0.071
Wales	55	100	0.23	0.065
Walpole	55	100	0.25	0.065
Waltham	55	105	0.28	0.069
Ware	55	100	0.23	0.066
Wareham	45	110	0.23	0.058
Warren	55	100	0.23	0.066
Warwick	65	100	0.24	0.070
Washington	65	90	0.22	0.067
Watertown	45	105	0.28	0.068
Wayland	55	100	0.27	0.068
Webster	55	100	0.23	0.064
Wellesley	55	100	0.27	0.067
Wellfleet	35	120	0.20	0.054
Wendell	65	100	0.23	0.069
Wenham	45	110	0.32	0.073
West Boylston	55	100	0.25	0.067
West Bridgewater	45	110	0.25	0.063
West Brookfield	55	100	0.23	0.066
West Newbury	55	110	0.35	0.077
West Springfield	55	100	0.23	0.065
West Stockbridge	65	90	0.22	0.066
West Tisbury	35	120	0.18	0.052
Westborough	55	100	0.25	0.067
Westfield	55	100	0.23	0.066
Westford	55	100	0.30	0.073
Westhampton	65	100	0.22	0.066
Westminster	65	100	0.26	0.071
Weston	55	100	0.27	0.068
Westport	45	110	0.23	0.058
Westwood	55	100	0.26	0.066
Weymouth	45	105	0.27	0.066
Whately	65	100	0.22	0.067
Whitman	45	110	0.25	0.063
Wilbraham	55	100	0.23	0.065
Willamsburg	65	100	0.22	0.067
Williamstown	65	90	0.23	0.069
Wilmington	55	105	0.31	0.073
Winchendon	65	100	0.26	0.071
Winchester	55	105	0.29	0.070
Windsor	65	90	0.22	0.067
Winthrop	45	105	0.29	0.068
Woburn	55	105	0.30	0.071
Worcester	55	100	0.24	0.067
Worthington	65	100	0.22	0.067
Wrentham	55	100	0.24	0.064
Yarmouth	35	120	0.19	0.052

THE MASSACHUSETTS STATE BUILDING CODE

780 CMR 1605.0 LOAD COMBINATIONS

1605.1 General. Buildings and other structures and portions thereof shall be designed to resist the load combinations specified in 780 CMR 1605.2 or 1605.3 and 780 CMR 18.00 through 23.00, and load combinations which include the special seismic load of ASCE 7, Section 9.5.2.7.1, where required by ASCE 7, Section 9.5.2.6.2.11 or 9.5.2.6.3.1. 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.

1605.2 Load Combinations Using Strength Design or Load and Resistance Factor Design.

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

EQUATION 16-1

$$1.4(D+F)$$

EQUATION 16-2

$$1.2(D+F) + 1.6(L+H) + 0.5(L_r \text{ or } S \text{ or } R)$$

EQUATION 16-3

$$1.2(D+F) + 1.6(L_r \text{ or } S \text{ or } R) + (f_1 L \text{ or } 0.8W) + 1.6H$$

EQUATION 16-4

$$1.2D + 1.6W + f_1 L + 0.5(L_r \text{ or } S \text{ or } R) + 1.6H$$

EQUATION 16-5

$$1.2D + 1.0E + f_1 L + 0.5S + 1.6H$$

EQUATION 16-6

$$0.9D + (1.0E \text{ or } 1.6W) + 1.6H$$

where:

$f_1 = 1.0$ for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79 kN/m²), and for parking garage live load.

$f_1 = 0.5$ for other live loads.

Where lateral soil pressure provides resistance to structural actions from other forces, it shall not be included in H, but shall be included in the design resistance.

Exception. Where other factored load combinations are specifically required by the provisions of 780 CMR, such combinations shall take precedence.

1605.2.2 Other Loads.

1605.2.2.1 Load Combinations Including Flood Load. When a structure is included in a flood zone (see 780 CMR 1612.0), the following load combinations shall be considered:

1. In areas subject to high velocity wave action, $1.6W$ in Equations 16-4 and 16-6 shall be replaced with $1.6W + 2.0 F_a$, where F_a is the flood load.

2. In other areas, $1.6W$ in Equations 16-4 and 16-6 shall be replaced with $0.8W + 1.0 F_a$:

Said load combinations are in addition to and shall not replace those of Equations 16-4 and 16-6.

1605.2.2.2 Effects of Temperature and Ponding. The forces and stresses generated by the effects of temperature and ponding shall be considered.

1605.2.2.3 Load Combinations Including Crane Load. 1.6 times the effects of vertical and horizontal crane loads shall be added to Equations 16-2, 16-3, 16-4, and 16-6. When combined with crane loads in these equations:

0.75S may be substituted for S;

0.5W may be substituted for W;

L_r may be taken as 0; and

E may be taken as 0.

(The substituted values shall then be multiplied by the numerical coefficients given in the equations).

1605.3 Load Combinations Using Allowable Stress Design.

1605.3.1 Basic Load Combinations. Where allowable stress design (working stress design), as permitted by 780 CMR, is used, structures and portions thereof shall resist the most critical effects resulting from the following combinations of loads:

EQUATION 16-7

$$D + F$$

EQUATION 16-8

$$D + H + F + L$$

EQUATION 16-9

$$D + H + F + L + (L_r \text{ or } S \text{ or } R)$$

EQUATION 16-10

$$2/3 [1.2D + (1.6W \text{ or } 1.0E) + f_1 L + 0.5(L_r \text{ or } S \text{ or } R) + 1.6H]$$

EQUATION 16-11

$$0.6D + W + H$$

EQUATION 16-12

$$0.6D + 0.7E + H$$

where f_1 is defined in 780 CMR 1605.2.1.

Increases in allowable stresses allowed in the various materials standards referenced by the materials chapters of 780 CMR shall not be used,

except that a duration of load increase shall be permitted in accordance with 780 CMR 23.00.

Where lateral soil pressure provides resistance to structural actions from other forces, it shall not be included in H , but shall be included in the design resistance.

1605.3.2 Other Loads.

1605.3.2.1 Load Combinations Including Flood Load. When a structure is included in a flood zone (see 780 CMR 1612), the following load combinations shall be considered:

1. In areas subject to high velocity wave action, $1.5 F_a$ shall be added to other loads in Equations 16-10 and 16-11, where F_a is the flood load, and E shall be set to zero in Equation 16-10.

2. In other areas, $0.75 F_a$ shall be added to other loads in Equations 16-10 and 16-11, and E shall be set to zero in Equation 16-10. Said load combinations are in addition to and shall not replace those of Equations 16-10 and 16-11.

1605.3.2.2 Effects of Temperature and Ponding. The forces and stresses generated by the effects of temperature and ponding shall be considered.

1605.3.2.3 Load Combinations Including Crane Load. 1.0 times the effects of vertical and horizontal crane loads shall be added to Equations 16-8, 16-9, 16-10, and 16-12. When combined with crane loads in these equations:

- 0.75S may be substituted for S;
- 0.5W may be substituted for W;
- L_r may be taken as 0; and
- E may be taken as 0.

(The substituted values shall then be multiplied by the numerical coefficients, if any, given in the equations).

1605.4 Heliports and Helistops. Heliport and helistop landing or touchdown areas shall be designed for the following loads, combined in accordance with 780 CMR 1605.0:

1. Dead load, D , plus the gross weight of the helicopter, D_h , plus snow load, S .
2. Dead load, D , plus two single concentrated impact loads, L , approximately 8 feet (2438 mm) apart applied anywhere on the touchdown pad (representing each of the helicopter's two main landing gear, whether skid type or wheeled type), having a magnitude of 0.75 times the gross weight of the helicopter. Both loads acting together total 1.5 times the gross weight of the helicopter.
3. Dead load, D , plus a uniform live load, L , of 100 pounds per square foot (4.79 kN/m²).

780 CMR 1606.0 DEAD LOADS

1606.1 Weights of Materials and Construction. In

determining dead loads for purposes of design, the actual weights of materials and construction shall be used but not less than the applicable minimum design dead loads given in Table C3-1 of ASCE 7. In the absence of definite information, values used shall be subject to the approval of the building official.

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

1606.3 Weight of Concrete Due to Deflection. Account shall be taken of the additional weight of concrete resulting from the deflection of the supporting deck and framing when placing concrete.

1606.4 Weight of Ceilings: Provision shall be made for the weight of ceilings, lights, ducts, and pipes which can be supported within or under the framing of floors or roofs, whether they are presently installed or whether they can be subsequently installed.

780 CMR 1607.0 LIVE LOADS

1607.1 General. Live loads are those loads defined in 780 CMR 1602.1.

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

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

TABLE 1607.1 MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS AND MINIMUM CONCENTRATED LIVE LOADS

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
1. Apartments (see residential)		
2. Access floor systems		
Office use	50	2,000
Computer use	100	2,000
3. Armories and drill rooms	150	
4. Assembly areas and theaters		
Fixed seats (fastened to floor)	60	
Lobbies	100	
Movable seats	100	
Stages	150	3,000
platforms	100	
Follow spot, projections and control rooms	50	
Catwalks	40	

780 CMR: STATE BOARD OF BUILDING REGULATIONS AND STANDARDS

THE MASSACHUSETTS STATE BUILDING CODE

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
5. Balconies (exterior) On one- and two-family residences only, and not exceeding 100 ft. ²	100 60	
6. Decks	Same as occupancy served	
7. Bowling alleys	75	
8. Cornices	60	See 780 CMR 1607.11.2.5
9. Corridors, except as otherwise indicated	100	
10. Dance halls and ballrooms	100	
11. Dining rooms and restaurants	100	
12. Dwellings (see residential)		
13. Elevator machine room grating (on area of 4 in. ²)		300
14. Finish light floor plate construction (on area of 1 in. ²)		200
15. Fire escapes On single-family dwellings only	100 40	
16. Garages (passenger vehicles only) Trucks and buses	50 See 780 CMR 1607.6	Note a
17. Grandstands (see stadium and arena bleachers)		
18. Gymnasiums, main floors and balconies	100	
19. Handrails, guards and grab bars	See 780 CMR 1607.7	
20. Hospitals Operating rooms, laboratories Private rooms Wards Corridors above first floor	100 40 40 80	1,000 1,000 1,000 1,000
21. Hotels (see residential)		
22. Laboratories	100	2,000
23. Libraries Reading rooms Stack rooms Corridors above first floor	60 150 ^b 80	1,000 1,000 1,000
24. Manufacturing Light Heavy	125 250	2,000 3,000
25. Marquees	75	
26. Office buildings File and computer rooms shall be designed for heavier loads based on anticipated occupancy Lobbies and first-floor corridors Offices Corridors above first floor	100 50 80	2,000 2,000 2,000
27. Penal institutions Cell blocks Corridors	40 100	

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
28. Residential Group R-3 Uninhabitable attics without storage Uninhabitable attics with storage Habitable attics and sleeping areas All other areas except balconies and decks Hotels and multifamily dwellings Corridors above first floor serving guest rooms Private rooms Public rooms and corridors serving them	10 20 30 40 80 40 100	
29. Reviewing stands, grandstands and bleachers	Note c	
30. Roofs	See 780 CMR 1607.11	
31. Schools Classrooms Corridors above first floor First-floor corridors	50 80 100	1,000 1,000 1,000
32. Scuttles, skylight ribs and accessible ceilings		200
33. Sidewalks, vehicular driveways and yards, subject to trucking	250 ^d	8,000 ^e
34. Skating rinks	100	
35. Stadiums and arenas Bleachers Fixed seats (fastened to floor)	100 ^c 60 ^c	
36. Stairs and exits One- and two-family dwellings All other	100 40 100	Note f
37. Storage warehouses (shall be designed for heavier loads if required for anticipated storage) Light Heavy	125 250	
38. Stores Retail First floor Upper floors Wholesale, all floors	100 75 125	1,000 1,000 1,000
39. Vehicle barriers	See 780 CMR 1607.7	
40. Walkways and elevated platforms (other than exitways)	60	
41. Yards and terraces, pedestrians	100	

For SI: 1 square inch = 645.16 mm², 1 pound per square foot = 0.0479 kN/m², 1 pound = 0.004448 kN.

- a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 1607.1 or the following concentrated load:
 - (1) for passenger cars accommodating not more than nine passengers, 3,000 pounds acting on an area of 4.5 inches by 4.5 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 weight of books and shelving shall be computed using an assumed density of 65 pounds per cubic foot and converted to a uniformly distributed load; this load shall be used if it exceeds 150 pounds per square foot. The 150 psf load requirement does not apply to libraries that are incidental to other uses.
- c. Design in accordance with the ICC *Standard on Bleachers, Folding and Telescopic Seating and Grandstands*.
- d. Other uniform loads in accordance with an approved method which contains provisions for truck loadings shall also be considered where appropriate.
- e. The concentrated wheel load shall be applied on an area of 20 square inches.
- f. Minimum concentrated load on stair treads (on area of four square inches) is 300 pounds.
- g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official. See 780 CMR 1608.0. For special-purpose roofs, see 780 CMR 1607.11.2.2.

1607.4 Concentrated Loads. Floors and other similar surfaces shall be designed to support the uniformly distributed live loads prescribed in 780 CMR 1607.2 or the concentrated load, in pounds (kilonewtons), given in Table 1607.1, whichever produces the greater load effects. Unless otherwise specified, the indicated concentration shall be assumed to be uniformly distributed on a floor or roof over an area of 2.5 feet square [6.25 ft² (0.58 m²)] and shall be located so as to produce the maximum load effects in the structural members. The aforementioned distribution of concentrated load does not apply for concentrated load applied to framing members, scuttles, skylight ribs, hung ceiling supports, cornices, and similar elements for which there is no deck to distribute load.

1607.5 Partition Loads. In office buildings and in other buildings where partition locations are subject to change, provision for partition weight shall be made, whether or not partitions are shown on the construction documents, unless the specified live load exceeds 80 pounds per square foot (3.83 kN/m²). Such partition load shall not be less than a uniformly distributed live load of 20 pounds per square foot (0.96kN/m²). Partition loads are non-reducible live load.

1607.6 Truck and Bus Garages. Minimum live loads for garages having trucks or buses shall be as specified in AASHTO Standard for H20-44 and HS20-44 Truck loadings, and for lane loads, but shall not be less than 100 pounds per square foot (4.80 kN/m²), unless other loads are specifically justified. Actual loads shall be used where they are greater than the above specified loads.

1607.6.1 Truck and Bus Garage Live Load Application. Lane loads shall be applied in aisles and ramps, as specified in AASHTO Standard. Truck loads shall be applied in aisles, ramps, in parking spaces, and berths, in positions that will maximize load effects.

1607.6.2 Horizontal Forces. Longitudinal forces and curb forces shall be as specified in AASHTO Standard. Vehicle barriers shall be designed for a concentrated 10,000 lb lateral force acting at a minimum height of two feet-zero inches above a floor or ramp.

1607.7 Loads on Handrails, Guards, Grab Bars and Vehicle Barriers. Handrails, guards, grab bars, and vehicle barriers shall be designed and constructed to the structural loading conditions set forth in 780 CMR 1607.7.

1607.7.1 Handrails and Guards. Handrail assemblies and guards shall be designed to resist a load of 100 pounds per lineal foot for grandstands, stadia, arenas, and similar structures used for public assembly, and 50 pounds per lineal foot (pound per foot) (0.73 kN/m) for other uses, applied in any direction at the top and to transfer this load through the supports to the structure.

Exception. For one- and two-family dwellings, only the single, concentrated load required by 780 CMR 1607.7.1.1 shall be applied.

1607.7.1.1 Concentrated Load. Handrail assemblies and guards shall be able to resist a single concentrated load of 300 pounds for grandstands, stadia, arenas, and similar structures used for public assembly, and 200 pounds (0.89 kN) for other uses, applied in any direction at any point along the top, and have attachment devices and supporting structure to transfer this loading to appropriate structural elements of the building. This load need not be assumed to act concurrently with the loads specified in 780 CMR 1607.7.1.

1607.7.1.2 Components. Intermediate rails (all those except the handrail), balusters and panel fillers shall be designed to withstand a horizontally applied normal load of 200 pounds (0.88 kN) on an area equal to one square foot (305 mm²) including openings and space between rails. Reactions due to this loading are not required to be superimposed with those

THE MASSACHUSETTS STATE BUILDING CODE

of 780 CMR 1607.7.1 or 1607.7.1.1.

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

1607.7.3 Vehicle Barriers. Vehicle barrier systems for passenger cars shall be designed to resist a single load of 7,000 pounds (31.2 kN) applied horizontally in any direction to the barrier system and shall have anchorage or attachment capable of transmitting this load to the structure. For design of the system, the load shall be assumed to act at a minimum height of one foot, six inches (457 mm) above the floor or ramp surface on an area not to exceed one square foot (305 mm²), and is not required to be assumed to act concurrently with any handrail or guard loadings specified in the preceding paragraphs of 780 CMR 1607.7.1. Garages accommodating trucks and buses shall be designed in accordance with an approved method that contains provision for traffic railings.

1607.8 Impact Loads. The live loads specified in 780 CMR 1607.2 include allowance for impact conditions. Provision shall be made in the structural design for uses and loads that involve unusual vibration and impact forces.

1607.8.1 Elevators. Structural supports for elevators, dumbwaiters, escalators and moving walks shall be designed for the *loads* and within the limits of the deflection specified in the Board Elevator Regulations (524 CMR 1.0 through 34.0), listed in 780 CMR 35.00. (In accordance with 524 CMR, all suspended elevator *loads* shall be increased 100% for impact).

1607.8.2 Machinery. For the purpose of design, the weight of machinery and moving loads shall be increased as follows to allow for impact:

1. elevator machinery, 100%;
2. light machinery, shaft- or motor-driven, 20%;
3. reciprocating machinery or power-driven units, 50%.

Percentages shall be increased where specified by the manufacturer.

1607.8.3 Hangers. Live loads on hangers supporting floors, balconies, stairs, walkways, or platforms shall be multiplied by an impact factor of 1.33.

1607.9 Reduction in Live Loads. The minimum uniformly distributed live loads, L_o , in Table 1607.1 are permitted to be reduced according to the 780 CMR 1607.9.1.1 through 1607.9.1.7.

1607.9.1 General. Subject to the limitations of 780 CMR 1607.9.1.1 through 1607.9.1.7, members for which a value of K_{LLAT} is 400 square

feet (37.16 m²) or more are permitted to be designed for a reduced live load in accordance with the following equation:

EQUATION 16-21

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

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

where:

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

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

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

A_T = Tributary area, in square feet (square meters). L shall not be less than $0.50L_o$ for members supporting one floor and L shall not be less than $0.40L_o$ for members supporting two or more floors.

TABLE 1607.9.1 LIVE LOAD ELEMENT FACTOR K_{LL}

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

1607.9.1.1 Heavy Live Loads. Live loads that exceed 100 pounds per foot squared (4.79 kN/m²) shall not be reduced except the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20%, but the live load shall not be less than L as calculated in 780 CMR 1607.9.1.

1607.9.1.2 Passenger Car Garages. The live loads shall not be reduced in passenger vehicle garages except the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20%, but the live load shall not be less than L as calculated in 780 CMR 1607.9.1.

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

occupancies, except that the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20%, but the live load shall not be less than L as calculated in 780 CMR 1607.9.1.

1607.9.1.4 Special Structural Elements. Live loads shall not be reduced for one-way slabs except as permitted in 780 CMR 1607.9.1.1. Live loads of 100 pound per foot squared (4.79 kN/m²) or less shall not be reduced for roof members except as specified in 780 CMR 1607.11.2.

1607.9.1.5 Hangers. Live load shall not be reduced for hangers.

1607.9.1.6 Open Web Steel Joists. Live load shall not be reduced for open web steel bar joists.

1607.9.1.7 Concrete Flat Slabs and Plates. Live load shall not be reduced for peripheral (two-way action) shear around columns, capitals, and drop panels of concrete flat slabs, flat plates, and grid slabs.

1607.10 Distribution of Floor Loads. Where uniform floor live loads are involved in the design of structural members arranged so as to create continuity, the minimum applied loads shall be the full dead loads on all spans in combination with the floor live loads on spans selected to produce the greatest effect at each location under consideration. It shall be permitted to reduce floor live loads in accordance with 780 CMR 1607.9.

1607.11 Roof Loads. The structural supports of roofs and marquees shall be designed to resist wind and, where applicable, snow and earthquake loads, in addition to the dead load of construction and the appropriate live loads as prescribed in this 780 CMR, or as set forth in Table 1607.1. The live loads acting on a sloping surface shall be assumed to act vertically on the horizontal projection of that surface.

1607.11.1 Distribution of Roof Loads. Where uniform roof live loads are involved in the design of structural members arranged so as to create continuity, the minimum applied loads shall be the full dead loads on all spans in combination with the roof live loads on adjacent spans or on alternate spans, whichever produces the greatest effect. See 780 CMR 1607.11.2 for minimum roof live loads and ASCE 7, Section 7.5 for partial snow loading.

1607.11.2 Minimum Roof Live Loads. Minimum roof loads shall be determined for the specific conditions in accordance with 780 CMR 1607.11.2.1 through 1607.11.2.5.

1607.11.2.1 Flat, Pitched and Curved Roofs. Ordinary flat, pitched and curved roofs shall be designed for the live loads specified in the

following equation or other controlling combinations of loads in 780 CMR 1605.0, whichever produces the greater load. In structures, where special scaffolding is used as a work surface for workers and materials during maintenance and repair operations, a lower roof load than specified in the following equation shall not be used unless approved by the building official. Greenhouses shall be designed for a minimum roof live load of ten pounds per square foot (0.479 kN/m²).

EQUATION 16-24

$$L_r = 20R_1R_2$$

where: $12 \leq L_r \leq 20$

For SI: $L_r = 0.96 R_1R_2$

where: $0.58 \leq L_r \leq 0.96$

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

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

EQUATION 16-25

$$R_1 = 1 \quad \text{for } A_t \leq 200 \text{ square feet (18.58 m}^2\text{)}$$

EQUATION 16-26

$$R_1 = 1.2 - 0.001A_t \quad \text{for } 200 \text{ square feet} < A_t < 600 \text{ square feet}$$

$$\text{For SI: } 1.2 - 0.011A_t \quad \text{for } 18.58 \text{ square meters} < A_t < 55.74 \text{ square meters}$$

EQUATION 16-27

$$R_1 = 0.6 \quad \text{for } A_t > 600 \text{ square feet (55.74 m}^2\text{)}$$

where:

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

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

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

EQUATION 16-28

$$R_2 = 1 \quad \text{for } F \leq 4$$

EQUATION 16-29

$$R_2 = 1.2 - 0.05 F \quad \text{for } 4 < F < 12$$

EQUATION 16-30

$$R_2 = 0.6 \quad \text{for } F \geq 12$$

1607.11.2.2 Special-purpose Roofs. Roofs used for promenade purposes shall be designed for a minimum live load of 60 pounds per

THE MASSACHUSETTS STATE BUILDING CODE

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

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

1607.11.2.4 Awnings and Canopies. Awnings and canopies shall be designed for a uniform live load of five pounds per square foot (0.240 kN/m²) as well as for snow loads and wind loads as specified in 780 CMR 1608.0 and 1609.0.

1607.11.2.5 Cornices and Overhanging Eaves. Cornices, overhanging eaves and other similar projections shall be designed for a minimum uniform load of 60 pounds per square foot or a load of 100 pounds per lineal foot of projection, whichever gives the most severe structural effect. The linear load shall be positioned to maximize the various structural effects.

1607.12 Crane Loads. The crane live load shall be the rated capacity of the crane. Design loads for the runway beams, including connections and support brackets, of moving bridge cranes and monorail cranes shall include the maximum wheel loads of the crane and the vertical impact, lateral, and longitudinal forces induced by the moving crane.

1607.12.1 Maximum Wheel Load. The maximum wheel loads shall be the wheel loads produced by the weight of the bridge, as applicable, plus the sum of the rated capacity and the weight of the trolley with the trolley positioned on its runway at the location where the resulting load effect is maximum.

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

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

1607.12.3 Lateral Force. The lateral force on

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

1607.12.4 Longitudinal Force. The longitudinal force on crane runway beams, except for bridge cranes with hand-gear bridges, shall be calculated as 10% of the maximum wheel loads of the crane. The longitudinal force shall be assumed to act horizontally at the traction surface of a runway beam, in either direction parallel to the beam.

1607.13 Interior Walls and Partitions. Interior walls and partitions, including their finish materials, shall have adequate strength to resist the loads to which they are subjected but not less than a horizontal load of five pounds per square foot (0.240 kN/m²). Self-supporting portable partitions used in office furnishings are exempt from this requirement.

780 CMR 1608.0 SNOW LOADS

780 CMR 1608.0 is unique to Massachusetts

1608.1 General. Design snow loads shall be determined in accordance with ASCE 7, Section 7, except as provided otherwise in 780CMR 1608.0. The design roof load shall not be less than that determined by 780 CMR 1607.0.

1608.2 Ground Snow Loads. Disregard ASCE 7, Section 7.2, Figure 7.1, and Table 7-1. Ground snow loads, p_g , to be used in the determination of design snow loads for roofs and other surfaces exposed to snow shall be as set forth in 780 CMR 1604.10.

1608.3 Convex Curved Roofs. ASCE 7, Section 7.4.3 applies to convex curved roofs only. See 780 CMR 1608.4 for concave curved roofs.

1608.4 Concave Curved Roofs. The effective loaded area of a concave curved roof shall be that area of the surface of the roof where the tangents to the surface have a slope of 50 degrees or less. The total uniform snow load for concave curved roofs shall be the basic snow load, P_f , multiplied by the total horizontal projected area of the roof. This total load shall be applied uniformly over the effective loaded area of the roof.

1608.5 Drifts on Lower Roofs.

1608.5.1 Width of Windward drifts. The width of drift, w , in Figure 7-8 of ASCE 7, shall be eight times the height of the drift (8 times h_d or h_c as applicable) for all windward drifts.

1608.5.2 Multiple Level Roofs. For multiple stepped roofs similar to that shown in Figure 1608.5.1, the sum of all the roof lengths upwind above the drift under consideration, l_u^* , in Figure 1608.5.1, shall replace l_u in Figure 7-8 of ASCE 7. For multiple level roofs similar to that shown in

Figure 1608.5.2, if the total calculated height of a drift and the underlying uniform snow layer on the upwind side of a higher roof ($h_d + h_b$) is equal to or greater than $0.7h_r$, then the length, l_u^* , as shown in Figure 1608.5.2, shall be used in place of l_u in Figure 7-8 of ASCE 7.

FIGURE 1608.5.1 DRIFTING SNOW AT MULTIPLE LEVEL ROOFS

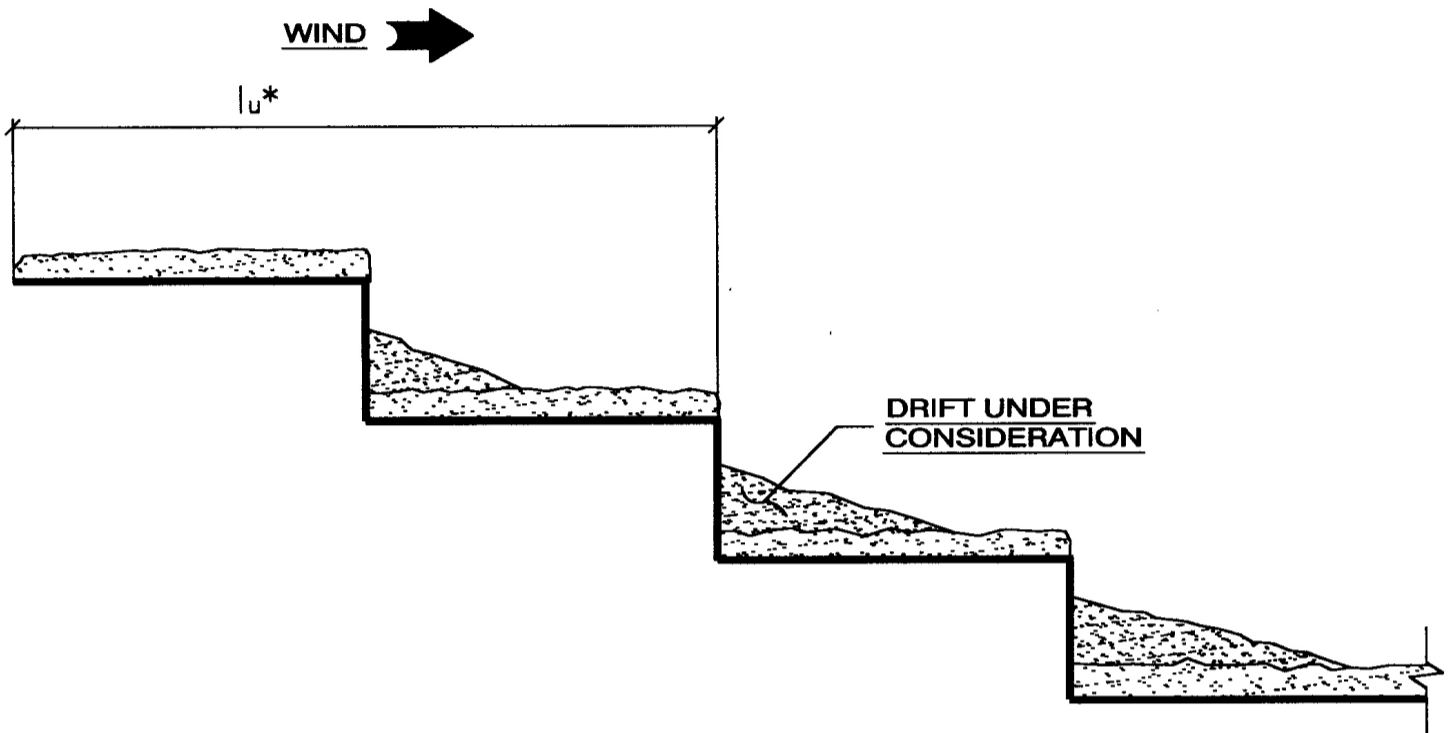
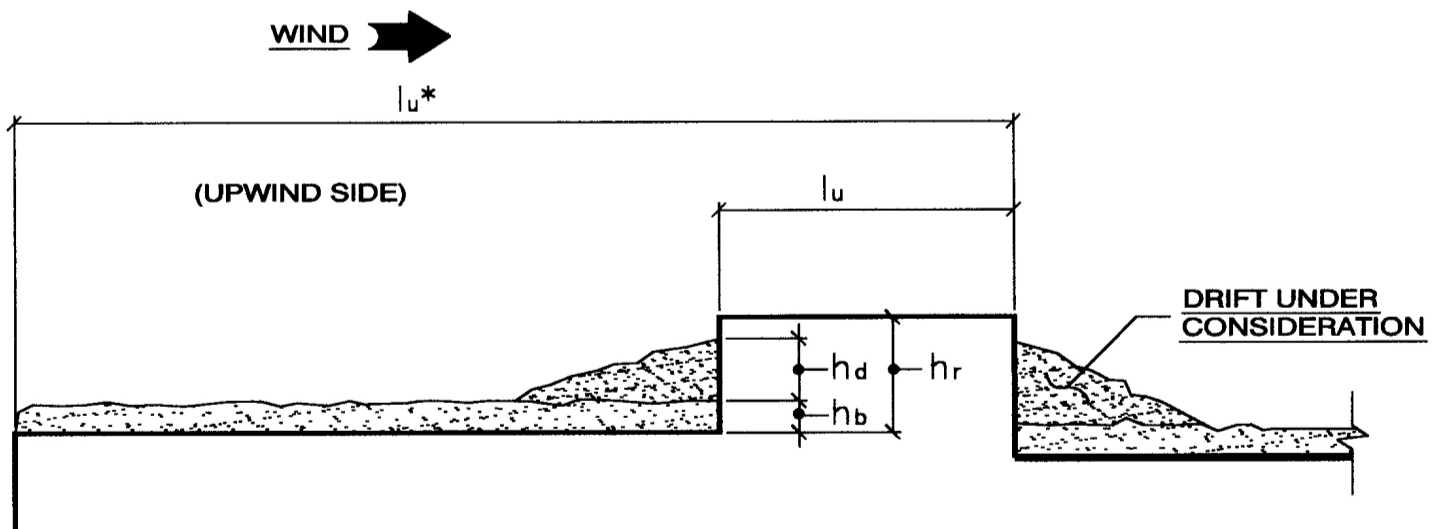


FIGURE 1608.5.2 LEEWARD DRIFT AFFECTED BY UPWIND WINDWARD DRIFT



THE MASSACHUSETTS STATE BUILDING CODE

1608.5.3 Very High Roof Separations. When the ratio h_r/L_T is greater than 1.0, where L_T is the dimension in feet of the upper roof perpendicular to the wind flow (perpendicular to l_u in Figure 7-8 of ASCE 7), the drift surcharge load on the lower roof due to drifting of snow from the upper roof may be reduced. The reduced height of the drift surcharge, h_{dr} , shall be not less than:

$$h_{dr} = h_r (2 - h_r/L_T)$$

except that when h_r/L_T is greater than 2.0, h_{dr} shall be equal to zero.

1608.5.4 Intersecting Drifts. Where there are drifts at walls intersecting at an angle, the unit snow load at any point on the lower roof shall be not less than the greater of the unit loads from the two individual drift surcharges, plus the unit load of the balanced snow load.

1608.5.5 Snow Pockets or Wells. Account shall be taken of the load effects of potentially excessive snow accumulation in pockets or wells of roofs or decks.

1608.6 Roof Projections. The term roof projections used in 780 CMR and In ASCE 7, Section 7.8 shall be interpreted to include screen walls, parapets, fire wall projections, and mechanical equipment. Drift loads at roof projections shall be in accordance with ASCE 7, Section 7.8, except that the width of drift shall be 8 times the height of the drift (eight times h_d or h_c in Figure 7-8, as applicable).

1608.7 Sliding Snow. In addition to the sliding snow load on a lower roof as required in ASCE 7, Section 7.9, the lower roof shall be designed for a windward drift surcharge at the wall separating the upper and lower roofs in accordance with 780 CMR 1608.5.1 and ASCE 7, Section 7.8. The sliding snow load and the windward drift surcharge need not be considered to act concurrently.

1608.7.1 Snow Guards. Sliding snow from an adjacent sloping high roof need not be considered on the low roof if snow guards, as specified in 780 CMR 1608.7.1, are provided on the high roof. In this case, the sloping roof with snow guards shall be designed for the unit snow loads required for a flat roof. Snow guards shall be designed by a registered design professional (RDP) qualified in the structural design of buildings. The design of the snow guards shall be shown on the construction documents. The RDP shall insure that there are adequate load paths from the snow guards into the supporting members and from the supporting members into the structure. The structural design of snow guards shall account for the impact of the sliding snow. The effectiveness in preventing the sliding of snow of proprietary snow guard systems shall be demonstrated by tests.

1608.8 Snow Storage and Collection Areas. Consideration of potentially excessive snow

accumulation shall be given to portions of structures designated or used as snow collection or storage areas during and after snow removal operations (e.g. temporary snow collection areas when mechanically removing snow from a roof; snow storage areas for parking structures).

780 CMR 1609 WIND LOADS

780 CMR 1609.0 is unique to Massachusetts

1609.1 General. Design wind loads shall be determined from ASCE 7, Section 6 except as provided otherwise in 780 CMR 1609.0.

1609.1.1 Limit to Wind Tunnel Procedure. Wind loads determined in accordance with ASCE 7, section 6.6 (Method 3) shall have a lower bound of $\frac{2}{3}$ of the wind loads determined for Exposure B in accordance with ASCE 7, Section 6.5 (Method 2).

1609.2 Basic Wind Speed and Wind Directionality Factor.

1609.2.1 Basic Wind Speed. In ASCE 7, disregard the body of the text after the section number and title of Section 6.5.4, disregard Subsection 6.5.4.1, and disregard Figure 6-1. The basic wind speed, V , used in the determination of design wind loads on buildings and other structures shall be as given in 780 CMR 1604.10, except as provide in 780 CMR 1609.2.2. V is the nominal three-second gust wind speed in miles per hour at 33 feet (10 m) above ground for Exposure C category. The wind shall be assumed to come from any horizontal direction.

1609.2.2 Estimation of Basic Wind Speeds from Regional Climatic Data. In ASCE 7, Subsection 6.5.4.2, replace "Fig. 6-1" wherever it occurs with "780 CMR 1604.10".

1609.3 Anchorage Against Overturning, Uplift and Sliding. Structural members and systems, and components and cladding in a building or structure shall be anchored to resist wind-induced overturning, uplift and sliding and to provide continuous load paths for these forces to the foundation. Where a portion of the resistance to these forces is provided by dead load, the dead load, 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.

1609.4 Wind and Seismic Detailing. Lateral-force-resisting systems shall meet seismic detailing requirements and limitations prescribed in 780 CMR, even when wind code prescribed load effects are greater than seismic load effects.

1609.5 Enclosure Classifications. Replace ASCE 7, Section 6.5.9.3 with 780 CMR 1609.5.1.

1609.5.1 Protection of Openings. In wind-borne debris regions, glazing in the lower 60 feet (18 288 mm) in buildings shall be assumed to be

openings unless such glazing is impact resistant or protected with an impact-resistant covering meeting the requirements of an approved impact-resisting standard or ASTM E 1996 and of ASTM E 1886 referenced 780 CMR as follows:

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

Exception. Wood structural panels with a minimum thickness of $\frac{7}{16}$ inch (11.1 mm) and maximum panel span of eight feet (2438 mm) shall be permitted for opening protection in one- and two-story buildings. Panels shall be precut to cover the glazed openings with attachment hardware provided. Attachments shall be designed to resist the components and cladding loads determined in accordance with the provisions of 780 CMR 1609.6.5. Attachment in accordance with Table 1609.5.1 is permitted for buildings with a mean roof height of 33 feet (10 058 mm) or less where wind speeds (three-second gust) do not exceed 130 miles per hour.

TABLE 1609.5.1 WINDBORN DEBRIS PROTECTION FASTENING SCHEDULE FOR WOOD STRUCTURAL PANELS^{a, b, c}

FASTENER TYPE	FASTENER SPACING (inches)			
	Panel span ≤ 2 feet	2 feet ≤ Panel span ≤ 4 feet	4 feet < Panel span ≤ 6 feet	6 feet < Panel span ≤ 8 feet
2½ # 6 Wood Screws	16	16	12	9
2½ # 6 Wood Screws	16	16	16	12

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

- a. Table 1609.5.1 is based on a maximum wind speed (three-second gust) of 130 mph and mean roof height of 33 feet or less.
- b. Fasteners shall be installed at opposing ends of the wood structural panel.
- c. Where screws are attached to masonry or masonry/stucco, they shall be attached utilizing vibration-resistant anchors having a minimum withdrawal capacity of 490 pounds.

1609.6 Rigid Tile Roof Coverings. Rigid tile roof coverings that are air-permeable are permitted to be designed in accordance with 780 CMR 1609.6.1

1609.6.1 Wind loads on air permeable rigid tile roof coverings may be determined in accordance with the following equation:

EQUATION 16-36

$$M_a = q_h C_L b L L_a [1.0 - G_{c_p}]$$

where:

b = Exposed width feet (m) 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 780 CMR 1715.2.

G_{c_p} = Roof pressure coefficient for each applicable roof zone determined from ASCE 7, Section 6. Roof coefficients shall not be adjusted for internal pressure.

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

L_a = Moment arm feet (m) 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 (kN-m) acting to raise the tail of the tile.

q_h = Wind velocity pressure psf (kN/m²) determined from ASCE 7, Section.

Concrete and clay roof tiles complying with the following limitations shall be designed to withstand the aerodynamic uplift moment as determined by 780 CMR 1609.6.

1. The roof tiles shall be either loose laid on battens, mechanically fastened, mortar set or adhesive set.
2. The roof tiles shall be installed on solid sheathing which has been designed as components and cladding.
3. An underlayment shall be installed in accordance with 780 CMR 15.00.
4. The tile shall be single lapped interlocking with a minimum head lap of not less than two inches (51 mm).
5. The length of the tile shall be between 1.0 and 1.75 feet (305 mm and 533 mm).
6. The exposed width of the tile shall be between 0.67 and 1.25 feet (204 mm and 381 mm).
7. The maximum thickness of the tail of the tile shall not exceed 1.3 inches (33 mm).
8. Roof tiles using mortar set or adhesive set systems shall have at least $\frac{2}{3}$ of the tile's area free of mortar or adhesive contact.

THE MASSACHUSETTS STATE BUILDING CODE

780 CMR 1610.0 LATERAL SOIL AND HYDROSTATIC LOADS*780 CMR 1610.0 is unique to Massachusetts*

1610.1 General. Basement, foundation, and retaining walls shall be designed to resist lateral loads due to soil and water pressure. Lateral soil pressure on said walls shall be determined in accordance with the principles of soil mechanics and as provided in 780 CMR 18.00. Floors or similar elements below the water table shall be designed to resist the upward pressure of the water.

Exception. Uninhabitable spaces with concrete floors on the ground with an under-slab drainage system, including sump pits and sump pumps, designed to keep the water level a minimum of one foot below the bottom of the floor slab need not be designed to resist water pressure.

780 CMR 1611.0 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.

EQUATION 16-37

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

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

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

1611.3 Controlled Drainage. Roofs equipped with hardware to control the rate of drainage shall be equipped with a secondary drainage system at a

higher elevation that limits accumulation of water on the roof above that elevation. Such roofs shall be designed to sustain the load of rainwater that will accumulate on them to the elevation of the secondary drainage system plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow determined from 780 CMR 1611.1. Such roofs shall also be checked for ponding instability in accordance with 780 CMR 1611.2.

780 CMR 1612.0 FLOOD LOADS*780 CMR 1612.0 is unique to Massachusetts*

1612.1 General. Flood loads shall be determined in accordance with ASCE 7, Section 5.3. Design and construction in flood zones shall be in accordance with ASCE 24 and 780 CMR 120.G and the more stringent design and construction requirements of ASCE 24 or 780 CMR 120.G, as applicable (“right to construct and structure allowed/required elevations shall be governed by the last sentence of 780 CMR 1612.1), shall be expressly identified at the building permit application stage and the more stringent requirements of either reference shall govern, except that flood loads shall be in accordance with the issue of ASCE 7 cited in 780 CMR 35.00 and “right-to construct” and structure-allowed elevations shall be governed by 780 CMR 120.G, M.G.L. c. 131, § 40 and any bylaws or ordinances that have legal standing in a community.

780 CMR 1613.0 EARTHQUAKE LOADS – PURPOSE

780 CMR 1613.0 through 1615.0 present criteria for the design and construction of buildings and nonbuilding structures subject to earthquake ground motion. The specified earthquake loads rely on post-elastic energy dissipation in the structure, and because of this fact, the provisions for design, detailing and construction shall be satisfied even for structures and members for which load combinations containing earthquake load produce lesser effects than other load combinations.

The purposes of 780 CMR 1613.0 through 1615.0 are to minimize the hazard to life of occupants of all buildings and nonbuilding structures, to increase the expected performance of high occupancy assembly and education buildings as compared to ordinary buildings, and to improve the capability of essential facilities to function during and after an earthquake. Because of the complexity of and the great number of variables involved in seismic design (e.g. variability in ground motion, soil types, dynamic characteristics of the structure, material strength properties and construction practices), 780 CMR 1613.0 through 1615.0 present only minimum criteria in general terms. These minimum criteria are considered to be prudent and economically justified for the protection of life safety in buildings subject to earthquakes and for improved capability of

essential facilities to function immediately following an earthquake.

Absolute safety and prevention of damage, even in an earthquake event with a reasonable probability of occurrence, cannot be achieved economically for most buildings. The “design earthquake” ground motion levels specified in 780 CMR may result in both structural and non-structural damage. For most buildings designed and constructed according to 780 CMR 1613.0 through 1615.0, it is expected that structural damage from a major earthquake may be repairable but the repair may not be economically feasible. For ground motions larger than the design levels, the intent of 780 CMR 1613.0 through 1615.0 is that there will be a low likelihood of building collapse.

780 CMR 1614.0 EARTHQUAKE LOADS – GENERAL

1614.1 Scope. Every building and portion thereof, and certain nonbuilding structures as provided 780 CMR 1614.0 shall as a minimum be designed and constructed to resist the effects of earthquake motions and assigned a seismic design category as set forth in 780 CMR 1614.0

Exceptions:

1. Detached one- and two-family dwellings are exempt from the requirements of these provisions.
2. Agricultural storage buildings that are intended only for incidental human occupancy are exempt from the requirements of these provisions.
3. Additions to existing buildings. An addition that is attached to an existing building shall be designed and constructed in accordance with the requirements of 780 CMR 34.00. Structurally separate additions shall be designed and constructed in accordance with 780 CMR 16.00.
4. Change of occupancy: Where a change of occupancy occurs in an existing building, the building shall conform to the provisions of 780 CMR 34.00.
5. Alterations. Where alterations are made to an existing building, the building shall conform to the provisions of 780 CMR 34.00.
6. Special structures including, but not limited to, vehicular bridges, transmission towers, hydraulic structures, and nuclear power generating facilities shall be designed to resist earthquake loads, but require special consideration of their response characteristics and environment that is beyond the scope of 780 CMR 1614.0.

Structures shall be designed in accordance with ASCE 7. Only the following sections, as modified in 780 CMR 1614.0, shall apply: 9.1.2.4, 9.1.2.5, 9.1.3, 9.1.4, 9.2 through 9.6, 9.13, and 9.14. Within the above-noted sections, all references to Sections 9.7 through 9.12 shall be revised as follows:

ASCE 7

ASCE 7	MASS STATE BUILDING CODE
9.7 Foundation Design Requirements	780 CMR 18.00 Soils and Foundations
9.8 Steel	780 CMR 22.00 Steel
9.9 Structural Concrete	780 CMR 19.00 Concrete
9.10 Composite Structure	(Not Applicable)
9.11 Masonry	780 CMR 21.00 Masonry
9.12 Wood	780 CMR 23.00 Wood

Appendices A.9 and B.0 of ASCE 7 are not applicable.

1614.2 Additions to Existing Buildings. See provisions of 780 CMR 34.00, except that 780 CMR 16.00 applies to structurally separate additions.

1614.3 Quality Assurance. A Quality Assurance Plan shall be provided where required by 780 CMR 17.00.

780 CMR 1615.0 EARTHQUAKE LOADS – MODIFICATIONS TO APPLICABLE PROVISIONS OF ASCE 7

(Note that the following subsections, commencing with the prefix number “9” are referring to applicable sections of ASCE 7).

1615.1 Changes to Applicable ASCE 7 Sections. The provisions of ASCE 7, Sections 9.1.2.4, 9.1.2.5, 9.1.3, 9.1.4, 9.2 through 9.6, 9.13 and 9.14 and all associated sub-sections shall apply except for certain paragraphs of those sections and sub-sections which have been modified or added as provided below:

9.1.2.4.1 New Buildings. New buildings and structures shall be designed and constructed in accordance with the quality assurance requirements of 780 CMR 17.00. The analysis and design of structural systems and components, including foundations, frames, walls, floors and roofs shall be in accordance with the applicable requirements of Section 9.5. The additional foundation requirements of 780 CMR 18.00 shall be followed. Materials used in construction and components made of these materials shall be designed and constructed to meet the requirements of 780 CMR 19.00 through 23.00. Architectural, electrical and mechanical systems and components, including tenant improvements, shall be designed in accordance with Section 9.6.

9.2.1 Definitions. The definitions presented in this Section provide the meaning of the terms used in these provisions. Definitions of terms that have a specific meaning relative to the use of concrete, masonry, steel or wood are presented in the Chapter devoted to the material (780 CMR 19.00, 21.00, 22.00 and 23.00). Disregard ASCE 7 references to non-applicable Sections and Appendix A9. (Note: See ASCE 7, Section 9.2.1 for definitions.)

THE MASSACHUSETTS STATE BUILDING CODE

9.2.2 Symbols. The unit dimensions used with the items covered by the symbols shall be consistent throughout except where specifically noted. The symbols and definitions presented in this Section apply to these provisions as indicated, except that references to A9.8 apply to 780 CMR 22.00 and references to A9.9 apply to 780 CMR 19.00. (Note: See ASCE 7, Section 9.2.2 for symbols.)

SECTION 9.4 – Site Ground Motion

9.4.1 Procedures for Determining Maximum Considered Earthquake and Design Earthquake Ground Motion Accelerations and Response Spectra. Ground motion accelerations, represented by response spectra and coefficients derived from these spectra, shall be determined in accordance with the general procedure of Section 9.4.1.2 or the site-specific procedure of Section 9.4.1.3. The general procedure in which spectral response acceleration parameters for the maximum considered earthquake ground motions are derived using Table 1604.10 of the Massachusetts State Building Code, modified by site coefficients to include local site effects and scaled to design values, are permitted to be used for any structure except as specifically indicated in these provisions. The site-specific procedure also is permitted to be used for any structure and shall be used where specifically required by these provisions.

9.4.1.1 Maximum Considered Earthquake Ground Motions. The maximum considered earthquake ground motions shall be as represented by the spectral response acceleration at short periods (S_s) and at 1-sec (S_1) obtained from Table 1604.10 of the Massachusetts State Building Code and adjusted for Site Class effects using the site coefficients of Section 9.4.1.2.4. When a site-specific procedure is used, maximum considered earthquake ground motion shall be determined in accordance with Section 9.4.1.3.

9.4.1.2 General Procedure for Determining Maximum Considered Earthquake and Design Spectral Response Accelerations. The maximum considered earthquake spectral response acceleration at short periods (S_s) and at 1-sec (S_1) shall be determined from Table 1604.10 of the Massachusetts State Building Code based on the municipality in which the site is located. Where a site is located in more than one municipality, the higher spectral response acceleration values shall be used.

For buildings and structures included in the scope of this Standard as specified in Section 9.1.2.1, the Site Class shall be determined in accordance with Section 9.4.1.2.1. The maximum considered earthquake spectral response accelerations adjusted for Site Class effects, S_{MS} and S_{M1} , shall be determined in accordance with Section 9.4.1.2.4 and the design spectral response accelerations, S_{DS} and S_{D1} , shall be determined in accordance with Section 9.4.1.2.5. The general response spectrum, when required by these provisions, shall be

determined in accordance with Section 9.4.1.2.6.

9.4.1.2.1 Site Class Definitions. The site shall be classified as one of the following classes:

A = Hard rock with measured shear wave velocity, $\bar{v}_s > 5000$ ft/s (1500 m/s)

B = Rock with 2500 ft/s $< \bar{v}_s \leq 5000$ ft/s (760 m/s $< \bar{v}_s \leq 1500$ m/s)

C = Very dense soil and soft rock with 1200 ft/s $\leq \bar{v}_s \leq 2500$ ft/s (370 m/s $\leq \bar{v}_s \leq 760$ m/s) or \bar{N} or $\bar{N}_{ch} > 50$ or $\bar{s}_u \geq 2000$ psf (100 kPa)

D = Stiff soil with 600 ft/s $\leq \bar{v}_s \leq 1200$ ft/s (180 m/s $\leq \bar{v}_s \leq 370$ m/s) or with $15 \leq \bar{N}$ or $\bar{N}_{ch} \leq 50$ or 1000 psf $\leq \bar{s}_u \leq 2000$ psf (50 kPa $\leq \bar{s}_u \leq 100$ kPa)

E = A soil profile with $\bar{v}_s < 600$ ft/s (180 m/s) or any profile with more than 10 ft (3 m) of soft clay. Soft clay is defined as soil with $PI > 20$, $w \geq 40\%$, and $s_u < 500$ psf (25 kPa)

F = Soils requiring site-specific evaluations:

1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. Potential for liquefaction shall be evaluated in accordance with 780 CMR 1804.6: *Liquefaction*.

Exception. None.

2. Peats and/or highly organic clays ($H > 10$ ft [3 m] of peat and/or highly organic clay where H = thickness of soil).
3. Very high plasticity clays ($H > 25$ ft [7.6 m] with $PI > 75$).
4. Very thick soft/medium stiff clays ($H > 120$ ft [37 m]).

Exception. None.

The following standards are referenced for determining the seismic coefficients:

- (a) ASTM. "Test Method for Penetration Test and Split-Barrel Sampling of Soils." *ASTM D1586-84*, 1984.
- (b) ASTM. "Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils." *ASTM D4318-93*, 1993.
- (c) ASTM. "Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock." *ASTM D2216-92*, 1992.
- (d) ASTM. "Test Method for Unconfined Compressive Strength of Cohesive Soil." *ASTM D2166-1991*, 1991.
- (e) ASTM. "Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression." *ASTM D2850-87*, 1987.

9.4.1.2.2 Steps for Classifying a Site. The Site Class of a site shall be determined using the following steps:

Step 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.

Exception. Refer to exception in Note a in Table 9.4.1.2.4a and in Table 9.4.1.2.4b.

Step 2: Check for the existence of a total thickness of soft clay > 10 ft (3 m) where a soft clay layer is defined by $s_u < 500$ psf (25 kPa), $w > 40\%$, and $Pl > 20$. If this criterion is satisfied, classify the site as Site Class E.

Step 3: Categorize the site using one of the following three methods with \bar{v}_s , \bar{N} , and \bar{s}_u computed in all cases as specified by the definitions in Section 9.4.1.2.3.

a. **The \bar{v}_s Method:** Determine \bar{v}_s for the top 100 ft (30m) of soil. Compare the value of \bar{v}_s with those given in Section 9.4.1.2 and Table 9.4.1.2 and assign the corresponding Site Class.

\bar{v}_s for rock, Site Class B, shall be measured on-site or estimated by a geotechnical engineer or engineering geologist/seismologist for competent rock with moderate fracturing and weathering. \bar{v}_s for softer and more highly fractured and weathered rock shall be measured on-site or shall be classified as Site Class C. The Classification of hard rock, Site Class A, shall be supported by on-site measurements of \bar{v}_s 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 at least 100 ft (30 m), surficial measurements of v_s are not prohibited from being extrapolated to assess \bar{v}_s .

b. **The \bar{N} Method:** Determine \bar{N} for the top 100 ft (30 m) of soil. Compare the value of \bar{N} with those given in Section 9.4.1.2 and Table 9.4.1.2 and assign the corresponding Site Class.

c. **The \bar{s}_u Method:** For cohesive soil layers, determine \bar{s}_u for the top 100 ft (30 m) of soil. For cohesionless soil layers, determine \bar{N}_{ch} for the top 100 ft (30 m) of soil. Cohesionless soil is defined by a $Pl <$

20 where cohesive soil is defined by a $Pl > 20$. Compare the values of \bar{s}_u and \bar{N}_{ch} with those given in Section 9.4.1.2 and Table 9.4.1.2 and assign the corresponding Site Class. When the \bar{N}_{ch} and \bar{s}_u criteria differ, assign the category with the softer soil (Site Class E soil is softer than D).

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

e. The rock categories, Site Classes A and B, shall not be assigned to a site if there is more than ten feet (3 m) of soil between the rock surface and the bottom of the spread footing or mat foundation.

9.4.1.2.3 Definitions of Site Class Parameters. The definitions presented below apply to the upper 100 ft (30 m) of the site profile. Profiles containing distinctly different soil and/or rock layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there are a total of n distinct layers in the upper 100 ft (30 m). Where some of the n layers are cohesive and others are not, k is the number of cohesive layers and m is the number of cohesionless layers. The symbol i refers to any one of the layers between 1 and n .

v_{si} is the shear wave velocity in ft/s (m/s).

d_i is the thickness of any layer between 0 and 100 ft (30 m).

\bar{v}_s is **EQUATION 9.4.1.2-1**

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

whereby $\sum_{i=1}^n d_i$ is equal to 100 ft (30 m)

N_i is the standard penetration resistance, ASTM D156-84 not to exceed 100 blows/ft as directly measured on the field without corrections. When refusal is met for a rock layer, N_i shall be taken as 100 blows/foot.

\bar{N} is **EQUATION 9.4.1.2-2**

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

where N_i and d_i in Equation 9.4.1.2-2 are for cohesionless soil, cohesive soil, and rock layers.

\bar{N}_{ch} is EQUATION 9.4.1.2-3

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

whereby $\sum_{i=1}^m d_i = d_s$. (Use only d_i and N_i for cohesionless soils.)

d_s is the total thickness of cohesionless soil layers in the top 100 ft (30 m)

\bar{S}_u is the undrained shear strength in psf (kPa), not to exceed 5000 psf (240 kPa), ASTM D2166-91 or D2850-87.

\bar{S}_u is EQUATION 9.4.1.2-4

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

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

d_c is the total thickness (100 - d_s) of cohesive soil layers in the top 100 ft (30 m)

Pl is the plasticity index, ASTM D4318-93

w is the moisture content in percent, ASTM D2216-92

Figures 9.4.1.1(a) through 9.4.1.1(j)

These figures are not included. Refer to Table 1604.10 of the Massachusetts State Building Code for all references to these figures.

9.4.1.2.4 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-sec (S_{M1}), adjusted for site class effects, shall be determined by Equations 9.4.1.2.4-1 and 9.4.1.2.4-2, respectively.

EQUATION 9.4.1.2.4-1

$$S_{M1} = F_a S_s$$

EQUATION 9.4.1.2.4-2

$$S_{m1} = F_v S_1$$

where

S_1 is the tabulated maximum considered earthquake spectral response acceleration at a period of 1-sec as determined in accordance with Section 9.4.1

S_s is the tabulated maximum considered earthquake spectral response acceleration at short periods as determined in accordance with Section 9.4.1

where site coefficients F_a and F_v are defined in Tables 9.4.1.2.4a and b, respectively.

TABLE 9.4.1.2.4a VALUES OF F_a AS A FUNCTION OF SITE CLASS AND SHORT PERIOD MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL ACCELERATION

Site Class	Tabulated Maximum Considered Earthquake Spectral Response Acceleration at Short Periods					
	$S_s \leq 0.26$	$0.27 \leq S_s \leq 0.29$	$0.30 \leq S_s \leq 0.32$	$0.33 \leq S_s \leq 0.35$	$0.36 \leq S_s \leq 0.38$	$S_s \geq 0.39$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.2	1.2	1.2	1.2
D	1.6	1.6	1.55	1.5	1.5	1.5
E	2.5	2.4	2.3	2.2	2.1	2.0
F	Note a	Note a	Note a	Note a	Note a	Note a

Note a: Site-specific geotechnical investigation and dynamic site response analyses shall be performed except that for structures with periods of vibration equal to or less than 0.5-seconds, values of F_a for liquefiable soils may be assumed equal to the values for the site class determined without regard to liquefaction in Step 3 of 9.4.1.2.2.

TABLE 9.4.1.2.4b VALUES OF F_v AS A FUNCTION OF SITE CLASS

Site Class	Tabulated Maximum Considered Earthquake Spectral Response Acceleration at 1-Second Periods
	$S_T \leq 0.1$
A	0.8
B	1.0
C	1.7
D	2.4
E	3.5
F	Note a

Note a: Site-specific geotechnical investigation and dynamic site response analyses shall be performed except that for structures with periods of vibration equal to or less than 0.5-seconds, values of F_v for liquefiable soils may be assumed equal to the values for the site class determined without regard to liquefaction in Step 3 of 9.4.1.2.2.

9.4.2 Seismic Design Category. Structures shall be assigned a Seismic Design Category in accordance with Section 9.4.2.1. Seismic Design Categories A, E, and F are not applicable in Massachusetts.

9.4.2.1 Determination of Seismic Design Category. All structures shall be assigned to a Seismic Design Category based on their Seismic Use Group and the design spectral response acceleration coefficients, S_{DS} and S_{DI} , determined in accordance with Section 9.4.1.2.5. Each building and structure shall be assigned to the most severe Seismic Design Category in accordance with Table 9.4.2.1a or 9.4.2.1b, irrespective of the fundamental period of vibration of the structure, T .

Exception. The seismic Design Category is permitted to be determined from Table 9.4.2.1a alone when all of the following apply:

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, T_a , determined in accordance with Section 9.5.5.3.2, is less than $0.8T_s$, where T_s is determined in accordance with Section 9.4.1.2.6 and
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 and
3. Equation 9.5.5.2.1-1 is used to determine the seismic response coefficient, C_s and
4. The diaphragms are rigid or for diaphragms that are flexible, the distance between vertical elements of the seismic force-resisting system does not exceed 40 feet.

TABLE 9.4.2.1a SEISMIC DESIGN CATEGORY BASED UPON SHORT PERIOD RESPONSE ACCELERATIONS

Value of S_{DS}	Seismic Use Group		
	I	II	III
$S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

TABLE 9.4.2.1b SEISMIC DESIGN CATEGORY BASED UPON 1-SECOND PERIOD RESPONSE ACCELERATIONS

Value of S_{DI}	Seismic Use Group		
	I	II	III
$S_{DI} < 0.133g$	B	B	C
$0.133g \leq S_{DI} < 0.20g$	C	C	D
$0.20g \leq S_{DI}$	D	D	D

9.4.3 Quality Assurance. See 780 CMR 1614.3.

9.5 STRUCTURAL DESIGN CRITERIA, ANALYSIS, AND PROCEDURES

9.5.2.1 Design Basis. The seismic analysis and design procedures to be used in the design of structures and their components shall be as prescribed in this Section. The structure shall include complete lateral and vertical force-resisting systems capable of providing adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand. The design ground motions shall be assumed to occur along any horizontal direction of a structure. The adequacy of the structural systems shall be demonstrated through the construction of a mathematical model and evaluation of this model for the effects of design ground motions. The design seismic forces, and their distribution over the height of the structure, shall be established in accordance with one of the applicable procedures indicated in Section 9.5.2.5 and the corresponding internal forces and

THE MASSACHUSETTS STATE BUILDING CODE

deformations in the members of the structure shall be determined. An approved alternative procedure shall not be used to establish the seismic forces and their distribution unless the corresponding internal forces and deformations in the members are determined using a model consistent with the procedure adopted.

Individual members shall be provided with adequate strength to resist the shears, axial forces, and moments determined in accordance with these provisions, and connections shall develop the strength of the connected members or the forces indicated above. The deformation of the structure shall not exceed the prescribed limits when the structure is subjected to the design seismic forces.

A continuous load path, or paths, with adequate strength and stiffness shall be provided to transfer all forces from the point of application to the final point of resistance. The foundation shall be designed to resist the forces developed and accommodate the movements imparted to the structure by the design ground motions. The dynamic nature of the forces, the expected ground motion, and the design basis for strength and energy dissipation capacity of the structure shall be included in the determination of the foundation design criteria.

Allowable Stress Design is permitted to be used to evaluate sliding, overturning, and soil bearing at the soil-structure interface regardless of the design approach used in the design of the structure.

The foundation elements shall be designed to

resist the forces developed and shall accommodate the movements imparted to the building by the design ground motions. The foundation design criteria shall account for the dynamic nature of the seismic forces, the design ground motions, and the design basis for strength and ductility of the structure.

Consideration shall be given to the manner in which the earthquake lateral force, computed in accordance with Section 9.5.2.5, will be transmitted from the soil or rock to the structure. Transmission of the lateral force will occur through one or more of the following foundation elements:

1. Lateral soil pressure against foundation walls, footings, grade beams and/or pile caps;
2. Lateral soil pressure against piles, piers or caissons;
3. Side or bottom friction on walls, footings or mats; or
4. Batter piles.

Bottom friction under pile caps shall be assumed to be ineffective in transmitting horizontal forces.

The horizontal force shall be distributed among the various elements of the foundation in proportion to their estimated rigidities. Any element which will participate in the transfer of horizontal forces from the soil or rock to the structure shall be designed to resist forces in such a way that its ability to sustain static load will not be impaired.

TABLE 9.5.2.2 DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC-FORCE-RESISTING SYSTEMS

BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R^a	SYSTEM OVER-STRENGTH FACTOR, $-\Omega_o^c$	DEFLECTION AMPLIFICATION FACTOR, C_d^b	STRUCTURAL SYSTEM LIMITATIONS AND BUILDING HEIGHT (ft) LIMITATIONS BY SEISMIC DESIGN CATEGORY ^c		
					B	C	D ^d
1. Bearing Wall Systems							
A. Ordinary steel braced frames in light-gage construction	2211	4	2	3½	NL	NL	65
B. Ordinary steel concentrically braced frames	NP	NP	NP	NP A	NP	NP	NP
C. Special reinforced concrete shear walls	1910.2.4	5	2½	5	NL	NL	160
D. Ordinary reinforced concrete shear walls	1910.2.3	4	2½	4	NL	NL	NP
E. Detailed plain concrete shear walls	NP	NP	NP	NP	NP	NP	NP
F. Ordinary plain concrete shear walls	NP	NP	NP	NP	NP	NP	NP

STRUCTURAL DESIGN

BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R^a	SYSTEM OVER-STRENGTH FACTOR, $-\Omega_o^c$	DEFLECTION AMPLIFICATION FACTOR, C_d^b	STRUCTURAL SYSTEM LIMITATIONS AND BUILDING HEIGHT (ft) LIMITATIONS BY SEISMIC DESIGN CATEGORY ^c		
					B	C	D ^d
G. Special reinforced masonry shear walls	2106	5	2½	3½	NL	NL	160
H. Intermediate reinforced masonry shear walls	2106	3½	2½	2¼	NL	NL	NP
I. Ordinary reinforced masonry shear walls	NP	NP	NP	NP	NP	NP	NP
J. Detailed plain masonry shear walls	NP	NP	NP	NP	NP	NP	NP
K. Ordinary plain masonry shear walls	NP	NP	NP	NP	NP	NP	NP
L. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	2211/2305/ 2306	6	3	4	NL	NL	65
M. Light-framed walls with shear panels of all other materials	2211/2305 /2306	2	2½	2	35	35	NP
N. Light-framed wall systems using flat strap bracing	2211/2305/ 2306	4	2	3½	NL	NL	65
O. Ordinary plain prestressed masonry shear walls	NP	NP	NP	NP	NP	NP	NP
P. Intermediate prestressed masonry shear walls	2106	2½	2½	2½	NL	NP	NP
Q. Special prestressed masonry shear walls	NP	NP	NP	NP	NP	NP	NP
2. Building Frame Systems							
A. Steel eccentrically braced frames, moment-resisting connections at columns away from links	(15) ^j	8	2	4	NL	NL	160
B. Steel eccentrically braced frames, non-moment-resisting connections at columns away from links	(15) ^j	7	2	4	NL	NL	160
C. Special steel concentrically braced frames	(13) ^j	6	2	5	NL	NL	160
D. Ordinary steel concentrically braced frames	(14) ^j	¾	2	¾	NL	NL	35 ⁿ
E. Special reinforced concrete shear walls	1910.2.4	6	2½	5	NL	NL	160
F. Ordinary reinforced concrete shear walls	1910.2.3	5	2½	4½	NL	NL	NP
G. Detailed plain concrete shear walls	NP	NP	NP	NP	NP	NP	NP
H. Ordinary plain concrete shear walls	NP	NP	NP	NP	NP	NP	NP
I. Composite eccentrically braced frames	(14) ^k	8	2	4	NL	NL	160
J. Composite concentrically braced frames	(13) ^k	5	2	4½	NL	NL	160
K. Ordinary composite braced frames	(12) ^k	3	2	3	NL	NL	NP

THE MASSACHUSETTS STATE BUILDING CODE

BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R^a	SYSTEM OVER-STRENGTH FACTOR, $-\Omega_o^c$	DEFLECTION AMPLIFICATION FACTOR, C_d^b	STRUCTURAL SYSTEM LIMITATIONS AND BUILDING HEIGHT (ft) LIMITATIONS BY SEISMIC DESIGN CATEGORY ^c		
					B	C	D ^d
L. Composite steel plate shear walls	(17) ^k	6½	2½	5½	NL	NL	160
M. Special composite reinforced concrete shear walls with steel elements	(16) ^k	6	2½	5	NL	NL	160
N. Ordinary composite reinforced concrete shear walls with steel elements	(15) ^k	5	2½	4½	NL	NL	NP
O. Special reinforced masonry shear walls	2106	5½	2½	4	NL	NL	160
P. Intermediate reinforced masonry shear walls	2106	4	2½	4	NL	NL	NP
Q. Ordinary reinforced masonry shear walls	NP	NP	NP	NP	NP	NP	NP
R. Detailed plain masonry shear walls	NP	NP	NP	NP	NP	NP	NP
S. Ordinary plain masonry shear walls	NP	NP	NP	NP	NP	NP	NP
T. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	2211/2305/2306	6½	2½	4¼	NL	NL	65
U. Light-framed walls with shear panels of all other materials	2211/2305/2306	2½	2½	2½	35	35	NP
V. Ordinary plain prestressed masonry shear walls	NP	NP	NP	NP	NP	NP	NP
W. Intermediate Prestressed masonry shear walls	2106	3	2½	2½	NL	NP	NP
X. Special prestressed masonry shear walls	NP	NP	NP	NP	NP	NP	NP
3. Moment-Resisting Frame Systems							
A. Special steel moment frames	(9) ^j	8	3	5½	NL	NL	NL
B. Special steel truss moment frames	(12) ^j	7	3	5½	NL	NL	160
C. Intermediate steel moment frames	(10) ^j	4½	3	4	NL	NL	35 ^h
D. Ordinary steel moment frames	(11) ^j	3½	3	3	NL	NL	NP ^{hi}
E. Special reinforced concrete moment frames	(21.1) ^l	8	3	5½	NL	NL	NL
F. Intermediate reinforced concrete moment frames	(21.1) ^l	5	3	4½	NL	NL	NP
G. Ordinary reinforced concrete moment frames	NP	NP	NP	NP	NP	NP	NP
H. Special composite moment frames	(9) ^k	8	3	5½	NL	NL	NL
I. Intermediate composite moment frames	(10) ^k	5	3	4½	NL	NL	NP
J. Composite partially restrained moment frames	(8) ^k	6	3	5½	160	160	100

BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R^a	SYSTEM OVER-STRENGTH FACTOR, $-\Omega_o^c$	DEFLECTION AMPLIFICATION FACTOR, C_d^b	STRUCTURAL SYSTEM LIMITATIONS AND BUILDING HEIGHT (ft) LIMITATIONS BY SEISMIC DESIGN CATEGORY ^c		
					B	C	D ^d
K. Ordinary composite moment frames	(11) ^k	3	3	2½	NL	NP	NP
L. Special masonry moment frames	NP	NP	NP	NP	NP	NP	NP
4. Dual Systems with Special Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces							
A. Steel eccentrically braced frames, moment-resisting connections at columns away from links	(15) ^j	8	2½	4	NL	NL	NL
B. Steel eccentrically braced frames, non-moment-resisting connections at columns away from links	(15) ^j	7	2½	4	NL	NL	NL
C. Special steel concentrically braced frames	(13) ^j	8	2½	6½	NL	NL	NL
D. Special reinforced concrete shear walls	1910.2.4	8	2½	6½	NL	NL	NL
E. Ordinary reinforced concrete shear walls	1910.2.3	7	2½	6	NL	NL	NP
F. Composite eccentrically braced frames	(14) ^k	8	2½	4	NL	NL	NL
G. Composite concentrically braced frames	(13) ^k	6	2½	5	NL	NL	NL
H. Composite steel plate shear walls	(17) ^k	8	2½	6½	NL	NL	NL
I. Special composite reinforced concrete shear walls with steel elements	(16) ^k	8	2½	6½	NL	NL	NL
J. Ordinary composite reinforced concrete shear walls with steel elements	(15) ^k	7	2½	6	NL	NL	NP
K. Special reinforced masonry shear walls	2106	7	3	6½	NL	NL	NL
L. Intermediate reinforced masonry shear walls	2106	6	3	5	NL	NL	NP
M. Ordinary steel concentrically braced frames	NP	NP	NP	NP	NP	NP	NP
5. Dual Systems with Intermediate Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces^m							
A. Special steel concentrically braced frames ^f	(13) ^j	4½	2½	4½	NL	NL	35 ^h
B. Special reinforced concrete shear walls	1910.2.4	6	2½	5	NL	NL	160
C. Ordinary reinforced concrete shear walls	1910.2.3	5½	2½	4½	NL	NL	NP
D. Ordinary reinforced masonry shear walls	NP	NP	NP	NP	NP	NP	NP
E. Intermediate reinforced masonry shear walls	2106	5	3	4½	NL	NL	NP
F. Composite concentrically braced frames	(13) ^k	5	2½	4½	NL	NL	160
G. Ordinary composite braced frames	(12) ^k	4	2½	3	NL	NL	NP

THE MASSACHUSETTS STATE BUILDING CODE

BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R^a	SYSTEM OVER-STRENGTH FACTOR, $-\Omega_o^c$	DEFLECTION AMPLIFICATION FACTOR, C_d^b	STRUCTURAL SYSTEM LIMITATIONS AND BUILDING HEIGHT (ft) LIMITATIONS BY SEISMIC DESIGN CATEGORY ^c		
					B	C	D ^d
H. Ordinary composite reinforced concrete shear walls with steel elements	(15) ^k	5	3	4½	NL	NL	NP
I. Ordinary steel concentrically braced frames	NP	NP	NP	NP	NP	NP	NP
6. Shear Wall-Frame Interactive System with Ordinary Reinforced Concrete Moment Frames and Ordinary Reinforced Concrete Shear Walls	NP	NP	NP	NP	NP	NP	NP
7. Inverted Pendulum Systems							
A. Cantilevered column systems		2½	2	2½	NL	NL	35
B. Special steel moment frames	(9) ^j	2½	2	2½	NL	NL	NL
C. Ordinary steel moment frames	(11) ^j	1¼	2	2½	NL	NL	NP
D. Special reinforced concrete moment frames	21.1 ^l	2½	2	1¼	NL	NL	NL
8. Structural Steel Systems Not Specifically Detailed For Seismic Resistance	AISC--335 AISC--LRFD AISI AISC--HSS 2205.2.1 <i>See footnote p</i>	3	3	3	100	65	NP

For SI: 1 foot = 304.8 mm, 1 pound per square foot = 0.0479 kN/m².

- 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, Cd, for use in Sections 9.5.3.7.1 and 9.5.3.7.2.
- c. NL = Not Limited and NP = Not Permitted. For metric units use 30 m for 100 ft and use 50 m for 160 ft. Heights are measured from the base of the structure as defined in Section 9.2.1.
- d. See Section 9.5.2.2.4.1 for a description of building systems limited to buildings with a height of 240 ft (75 m) or less.
- e. **Note deleted.**
- f. Ordinary moment frame is permitted to be used in lieu of intermediate moment frames in Seismic Design Categories B and C.
- g. The tabulated value of overstrength factor, Ω , may be reduced by subtracting 1/2 for structures with flexible diaphragms, but shall not be taken as less than 2.0 for any structure.
- h. Steel ordinary moment frames and intermediate moment frames are permitted in single story buildings up to a height of 60 ft, when the moment joints of field connections are constructed of bolted end plates and the dead load of the roof does not exceed 15 psf. The dead weight of the portion of walls more than 35 feet above the base shall not exceed 15 psf.
- i. Steel ordinary moment frames are permitted in buildings up to a height of 35 feet, where the dead load of the walls, floors and roof does not exceed 15 psf.
- j. AISC 341 Seismic Part I or Part III, Section number.
- k. AISC 341 Seismic Part II, Section number.
- l. ACI 318, Section number.
- m. Steel intermediate moment resisting frames as part of a dual system are not permitted in Seismic Design Category D.
- n. Steel ordinary concentrically braced frames are permitted in penthouse structures and in single-story buildings up to a height of 60 feet when the dead load of the roof does not exceed 15 pounds per square foot.
- o. **Note deleted.**
- p. K-Braced Frames are not permitted. For V-Type and Inverted V-Type Bracing, the frames shall comply with AISC 341 Section 13.4a, as modified by this Code, except for clause (3).

9.5.2.6 Design and Detailing Requirements. The design and detailing of the components of the seismic force-resisting system shall comply with the requirements of this Section. Foundation design shall conform to the applicable requirements of 780 CMR 18.00. The materials and the systems composed of those materials shall conform to the requirements of 780 CMR 19.00 through 23.00 for the applicable category.

9.5.2.6.1.2 Anchorage of Concrete or Masonry Walls. Concrete and masonry walls shall be anchored to the roof and all floors and members that provide lateral support for the wall or which are supported by the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be capable of resisting the horizontal forces specified in Section 9.5.2.6.1.1 but not less than a minimum strength level, horizontal force of 280 lbs/ linear ft (4.09 kN/m) of wall substituted for E in the load combinations of 780 CMR 1605.2.1. and 1605.3. Also see 780 CMR 1604.8.

9.5.2.6.2.5 Nonredundant Systems. The design of a structure shall consider the potentially adverse effect that the failure of a single member, connection, or component of the seismic force-resisting system will have on the stability of the structure; see 780 CMR 1604.11.

9.5.2.6.2.8 Design of Concrete or Masonry Walls and Their Anchorage for Out-of-plane Forces. Concrete or masonry walls and their anchorage shall be designed for a force normal to the surface equal to 40% of the short period spectral response acceleration, S_{DS} , times the occupancy importance factor, I , multiplied by the weight of wall, with a minimum force of 10 % of the weight of the wall. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds four feet. Inter-connection of wall elements and connections to supporting framing systems shall have sufficient ductility, rotational capacity, or sufficient strength to resist shrinkage, thermal changes, and differential foundation settlement when combined with seismic forces.

9.5.2.6.2.8.1 Anchorage of Concrete or Masonry Walls. The anchorage of concrete or masonry walls to floors, roofs and other structural elements that provide lateral support to the wall

or are supported by the wall shall provide a positive direct connection capable of resisting the greater of the following:

1. A force of $0.4 S_{DS} I W_c$
2. A force of 400 $S_{DS} I$ lbs/linear foot (5.84 $S_{DS} I$ kN/m) of wall, except that a force of 800 $S_{DS}IW_c$ lbs/linear foot of wall shall apply for anchorage to flexible diaphragms
3. 280 lbs/linear ft of wall

where I = the occupancy importance factor, per Section 9.1.4. and W_c = the weight of wall tributary to the anchor.

9.5.2.6.3.2 Design and Anchorage of Concrete or Masonry Walls. Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragms. Added chords may be used to form subdiaphragms to transmit the anchorage forces to the main continuous cross ties. The maximum length-to-width ratio of the structural subdiaphragm shall be 2½ to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components. Connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm.

The strength design forces for steel elements of the wall anchorage system, other than anchor bolts and reinforcing steel, shall be 1.4 times the forces otherwise required by this Section.

In wood diaphragms, the continuous ties shall be in addition to the diaphragm sheathing. Anchorage shall not be accomplished by use of toe nails or nails subject to withdrawal nor shall wood ledgers of framing be used in cross-grain bending or cross-grain tension. The diaphragm sheathing shall not be considered effective as providing the ties or struts required by this Section.

In metal deck diaphragms, the metal deck shall not be used as the continuous ties required by this Section in the direction perpendicular to the deck span.

Diaphragm to wall anchorage using embedded straps shall be attached to, or hooked around, the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

When elements of the wall anchorage system are loaded eccentrically or are not perpendicular to the wall, the system shall be designed to resist all components

THE MASSACHUSETTS STATE BUILDING CODE

of the forces induced by the eccentricity.

When pilasters are present in the wall, the anchorage force at the pilasters shall be calculated considering the additional load transferred from the wall panels to the pilasters. However, the minimum anchorage force at a floor or roof shall not be reduced.

9.5.2.6.4.2 Plan or Vertical Irregularities. For structures having a plan structural irregularity of Type 1, 2, 3, or 4 in Table 9.5.2.3.2 or a vertical structural irregularity of Type 4 in Table 9.5.2.3.3, the design forces determined from Section 9.5.3.2 shall be increased 25% for connections of diaphragms to vertical elements and to collectors and for connections of collectors to the vertical elements. Collectors and their connections also shall be designed for these increased forces unless they are designed for the special seismic loads of Section 9.5.2.7.1, in accordance with Section 9.5.2.6.3.1.

9.5.2.7 Combination of Load Effects. The effects on the structure and its components due to seismic forces shall be combined with the effects of other loads in accordance with the combinations of load effects given in 780 CMR 1605.0. For use with those combinations, the earthquake-induced force effect shall include vertical and horizontal effects as given by Equation 9.5.2.7-1 or 9.5.2.7-2, as applicable. The vertical seismic effect term $0.2S_{DS}D$ need not be included where S_{DS} is equal to or less than 0.125 in Equations 9.5.2.7-1, 9.5.2.7-2, 9.5.2.7.1-1, and 9.5.2.7.1-2. The vertical seismic effect term $0.2S_{DS}D$ need not be included in Equation 9.5.2.7-2 when considering foundation overturning.

For Equation 16-5 in 780 CMR 1605.2.1 or 16-10 in 780 CMR 1605.3.1:

EQUATION 9.5.2.7-1

$$E = \rho Q_E + 0.2 S_{DS} D$$

For Equation 16-6 in 780 CMR 1605.2.1 or 16-2) in 780 CMR 1605.3.1:

EQUATION 9.5.2.7-2

$$E = \rho Q_E - 0.2 S_{DS} D$$

Where:

E = the effect of horizontal and vertical earthquake-induced forces

S_{DS} = the design spectral response acceleration at short periods obtained from Section 9.4.1.2.5

D = the effect of dead load, D

Q_E = the effect of horizontal seismic (earthquake-induced) forces

ρ = the reliability factor obtained from Section 9.5.2.4.

9.5.2.7.1 Special Seismic Load. Where specifically required by this Code, the special seismic load of Equation 9.5.2.7.1-1 shall be used to compute E for use in Equation 16-5 in 780 CMR 1605.2.1 or Equation 16-10 in 780 CMR 1605.3.1 and the special seismic load of Equation 9.5.2.7.1-2 shall be used to compute E in Equation 16-6 in 780 CMR 1605.2.1 or Equation 16-12 in 780 CMR 1605.3.1:

EQUATION 9.5.2.7.1-1

$$E = \Omega_o Q_E + 0.2 S_{DS} D$$

EQUATION 9.5.2.7.1-2

$$E = \Omega_o Q_E - 0.2 S_{DS} D$$

The value of the quantity $\Omega_o Q_E$ in Equations 9.5.2.7.1-1 and 9.5.2.7.1-2 need not be taken greater than the capacity of other elements of the structure to transfer force to the component under consideration.

Where allowable stress methodologies are used with the special load of this Section applied in Equation 16-10 or 16-12 of 780 CMR 1605.3.1, allowable stresses are permitted to be determined using an allowable stress increase of 1.1. This increase shall not be combined with increases in allowable stresses or load combination reductions otherwise permitted by this standard or the material reference standard except that combination with the duration of load increases permitted in 780 CMR 23.00 is permitted.

9.5.2.8 Deflection, Drift Limits, and Building Separation. The design story drift (Δ) as determined in Section 9.5.5.7 or 9.5.6.6, shall not exceed the allowable story drift (Δ_a) as obtained from Table 9.5.2.8 for any story. For structures with significant torsional deflections, the maximum drift shall include torsional effects. All portions of the structure shall be designed and constructed to act as an integral unit in resisting seismic forces unless separated by a distance sufficient to avoid damaging contact under total deflection δ_x as determined in Section 9.5.5.7 and below. All structures shall be separated from adjoining structures. Separations shall allow for the displacement δ_M . Adjacent structures on the same property shall be separated by at least δ_{MT} , at all levels, where

EQUATION 9.5.2.8-1

$$\delta_{MT} = \sqrt{[(\delta_{M1})^2 + (\delta_{M2})^2]}$$

and δ_{M1} and δ_{M2} are the displacements of the adjacent buildings. When a structure adjoins a property line not common to a public way, that structure shall also be set back from the property line by at least the displacement, δ_M , of that structure.

Exception. Smaller separations or property line setbacks shall be permitted when justified by engineering analyses based on maximum expected ground motions.

9.5.2.9 Foundation Walls and Retaining Walls. Exterior foundation walls and retaining walls shall be designed to resist at least the superimposed effects of the total static lateral soil and ground water pressures, excluding the pressure caused by any temporary surcharge, plus an earthquake force, F_w , for horizontal backfill surface, equal to:

EQUATION 9.5.2.9

$$F_w = 0.375(S_1)(F_v)(Y_t)(H)^2$$

where S_1 is the maximum considered earthquake spectral response acceleration at a period of 1-second from Table 1604.10 of the Massachusetts State Building Code, F_v is the soil amplification factor from Table 9.4.1.2.4b, Y_t is the total unit weight of the soil, and H is the height of the wall measured as the difference in elevation of finished ground surface or floor in front of and behind the wall.

Surcharges which are applied over extended periods of time shall be included in the total static lateral soil pressure and their earthquake lateral force shall be computed and added to the force determined above. The earthquake force from the backfill shall be distributed as an inverse triangle over the height of the wall.

The point of application of the earthquake force from an extended duration surcharge shall be determined on an individual case basis. If the backfill or the existing soil behind the backfill consists of loose saturated granular soil, the potential for liquefaction of the backfill or existing soil adjacent to the wall during seismic loading shall be evaluated in accordance with the requirements of 780 CMR 1804.6. If the backfill or existing soil beyond the backfill is potentially subject to liquefaction, the increase in design lateral load on the foundation wall or retaining wall shall be determined by a registered professional engineer, registered in the Commonwealth of Massachusetts.

For use in wall strength design, a load factor of 1.43 times the earthquake force calculated above shall be applied.

9.5.3 Effective Seismic Weight. The effective seismic weight, W , of the structure shall include the total dead load and other loads listed below:

1. In areas used for storage, a minimum of 25% of the floor live load (floor live load in public garages and open parking structures need not be included).
2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 psf (0.48kN/m²) of floor area, whichever is greater.
3. Total operating weight of permanent equipment.
4. 50 % of the roof snow load.

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

EQUATION 9.5.5.2-1

$$V = C_s W$$

where

C_s = the seismic response coefficient determined in accordance with Section 9.5.5.2.1

W = the effective seismic weight of the structure as defined in Section 9.5.3.

9.6 Architectural, Mechanical, and Electrical Components and Systems.

9.6.1 General. Section 9.6 establishes minimum design criteria for architectural, mechanical, electrical, and non-structural systems components, and elements permanently attached to structures including supporting structures and attachments (hereinafter referred to as "components") The design criteria establish minimum equivalent static force levels and relative displacement demands for the design of components and their attachments to the structure, recognizing ground motion and structural amplification, component toughness and weight, and performance expectations. Seismic Design Categories for structures are defined in Section 9.4.2. For the purposes of this Section, components shall be considered to have the same Seismic Design Category as that of the structure that they occupy or to which they are attached unless otherwise noted.

This Section also establishes minimum seismic design force requirements for nonbuilding structures that are supported by other structures where the weight of the nonbuilding structure is less than 25% of the combined weight of the nonbuilding structure and the supporting structure. Seismic design requirements for nonbuilding structures that are supported by other structures where the weight

THE MASSACHUSETTS STATE BUILDING CODE

of the nonbuilding structure is 25% or more of the combined weight of the nonbuilding structure and supporting structure are prescribed in Section 9.14. Seismic design requirements for nonbuilding structures that are supported at grade are prescribed in Section 9.14; however, the minimum seismic design forces for nonbuilding structures that are supported by another structure shall be determined in accordance with the requirements of Section 9.6.1.3 with R_p equal to the value of R specified in Section 9.14 and $a_p = 2.5$ for nonbuilding structures with flexible dynamic characteristics and $a_p = 1.0$ for nonbuilding structures with rigid dynamic characteristics. The distribution of lateral forces for the supported nonbuilding structure and all nonforce requirements specified in Section 9.14 shall apply to supported nonbuilding structures.

In addition, all components are assigned a component importance factor (I_p) in this chapter. The default value for I_p is 1.00 for typical components in normal service. Higher values for I_p are assigned for components, which contain hazardous substances, must have a higher level of assurance of function, or otherwise require additional attention because of their life safety characteristics. Component importance factors are prescribed in Section 9.6.1.5.

All architectural, mechanical, electrical, and other non-structural components in structures shall be designed and constructed to resist the equivalent static forces and displacements determined in accordance with this Section. The design and evaluation of support structures and architectural components and equipment shall consider their flexibility as well as their strength.

Exceptions: The following components are exempt from the requirements of this Section:

1. Not Used
2. Non-fire-resistance rated ceilings and access floors in Seismic Design Category B and C structures which are in Seismic Use Group I.
3. Mechanical and Electrical components in structures assigned to Seismic Design Category B provided the Importance Factor $I = 1.0$.
4. Mechanical and Electrical components in structures assigned to Seismic Design Category C provided the Importance factor $I = 1.0$ and having $a_p = 1.0$, other than elevator components and systems in buildings more than 70 feet in height.
5. Mechanical and electrical components in Seismic Design Category D, where $I_p = 1.0$ and flexible connections between the

components and associated ductwork, piping, and conduit are provided and that are mounted at 4 ft (1.22 m) or less above a floor level and weigh 400 lb (1780 N) or less.

6. Mechanical and electrical components in Seismic Design Category D, weighing 20 lbs (95 N) or less where $I_p = 1.0$ and flexible connections between the components and associated ductwork, piping, and conduit are provided, or for distribution systems, weighing 5 lb/ft (7 N/m) or less.

The functional and physical interrelationship of components and their effect on each other shall be designed so that the failure of an essential or nonessential architectural, mechanical, or electrical component shall not cause the failure of a nearby essential architectural, mechanical, or electrical component.

9.6.1.5 Component Importance Factor. The component importance factor (I_p) shall be selected as follows:

$I_p = 1.5$ for life safety component required to function after an earthquake (*e.g.*, fire protection sprinkler system)

$I_p = 1.5$ for stair and elevator enclosures and for non-loadbearing walls and/or partitions that form the exit from the facility and are required by other provisions of the code to provide a fire resistance rating for protection from smoke and fire for the means of egress path leading from the interior of the building to an exit discharge

$I_p = 1.5$ for component that contains hazardous content

$I_p = 1.5$ for storage racks in structures open to the public (*e.g.*, warehouse retail stores)

$I_p = 1.0$ for all other components

In addition, for structures in Seismic Use Group III:

$I_p = 1.5$ for all components needed for continued operation of the facility or whose failure could impair the continued operation of the facility

9.6.1.7 Construction Documents. Construction documents shall show the design and anchorage of all architectural, mechanical and electrical components and systems to the structure, except that the construction documents may alternatively include performance specifications for the design and detailing of components and systems and their anchorage. The performance specifications shall require that the design and detailing of components and systems be prepared by a professional engineer(s) registered in the Commonwealth of Massachusetts.

Table 9.6.1.7
CONSTRUCTION DOCUMENTS
DELETED

9.6.2.2 Architectural Component Forces and Displacements. Architectural components shall meet the force requirements of Section 9.6.1.3 and Table 9.6.2.2.

Components supported by chains or otherwise suspended from the structural system above are not required to meet the lateral

seismic force requirements and seismic relative displacement requirements of this Section provided that they cannot be damaged to become a hazard or cannot damage any other component when subject to seismic motion and they have ductile or articulating connections to the structure at the point of attachment. The gravity design load for these items shall be three times their operating load. Anchorage forces for concrete or masonry walls shall not be less than required by Section 9.5.2.6.

TABLE 9.6.2.2
ARCHITECTURAL COMPONENT COEFFICIENTS

Architectural Component or Element	a_p^a	R_p^b
Interior Nonstructural Walls and Partitions: Plain (unreinforced) masonry walls are not permitted. All other walls and partitions	NP ^c 1	NP 2.5
Cantilever Elements (Unbraced or Braced to Structural Frame Below Its Center of Mass) Parapets and cantilever exterior nonstructural walls. Chimneys and stacks when laterally braced or supported by the structural frame.	2.5 2.5	2.5 2.5
Cantilever Elements (Braced to Structural Frame Above Its Center of Mass) Parapets Chimneys and stacks Exterior nonstructural walls	1.0 1.0 1.0 ^b	2.5 2.5 2.5
Exterior Nonstructural Wall Elements and Connections Wall element Body of wall panel connections Fasteners of the connecting system	1.0 1.0 1.25	2.5 2.5 1
Veneer Limited deformability elements and attachments Low deformability elements and attachments	1.0 1.0	2.5 2.5
Penthouses (Except when Framed by an Extension of the Building Frame)	2.5	3.5
Ceilings All	1.0	2.5
Cabinets Storage cabinets and laboratory equipment	1.0	2.5
Access Floors Special access floors (designed in accordance with Section 9.6.2.7.2) All other	1.0 1.0	2.5 1.5
Appendages and Ornamentations	2.5	2.5
Signs and Billboards	2.5	2.5
Other Rigid Components High deformability elements and attachments Limited deformability elements and attachments Low deformability materials and attachments Other Flexible Components High deformability elements and attachments Limited deformability elements and attachments Low deformability materials and attachments	1.0 1.0 1.0 2.5 2.5 2.5	3.5 2.5 1.5 3.5 2.5 1.5

a. A lower value for a_p shall not be used unless justified by detailed dynamic analysis. The value for a_p shall not be less than 1.00. The value of $a_p = 1$ is for equipment generally regarded as rigid and rigidly attached. The value of $a_p = 2.5$ is for equipment generally regarded as flexible or flexibly attached. See Section 9.2.1 for definitions of rigid and flexible.

b. Where flexible diaphragms provide lateral support for walls and partitions, the design forces for anchorage to the diaphragm shall be as specified in Section 9.5.2.6.

c. NP = Not Permitted.

THE MASSACHUSETTS STATE BUILDING CODE

9.6.2.6.2.1 Seismic Design Categories B and C. Suspended ceilings in Seismic Design Categories B and C shall be designed and installed in accordance with the CISCA recommendations for seismic Zones 0-2, (Ref. 9.6-16), except that seismic forces shall be determined in accordance with Sections 9.6.1.3 and 9.6.2.6.1. Note that design of non-fire-resistance rated ceilings in Seismic Design Categories B and C which are in Seismic Use Group I need not meet these requirements.

Sprinkler heads and other penetrations in Seismic Design Category C shall have a minimum of 1/4 in. (6 mm) clearance on all sides.

9.13.6.2.3 Fire Resistance. Fire resistance ratings of the isolation system shall comply with 780 CMR 714.7.

9.14.6.6.1 General. Piers and wharves are structures located in waterfront areas that project into a body of water or parallel the shoreline. A pier or wharf supporting a building or buildings shall be designed considering soil-structure interaction and the dynamic properties of the combined building/pier or building/wharf structure for earthquake loads specified in this Code.

780 CMR 1616.0 THROUGH 1623.0 RESERVED

780 CMR 1624.0 *IN-SITU* LOAD TESTS

1624.1 General. Whenever there is a reasonable doubt as to the stability or load-bearing capacity of a completed building, structure or portion thereof for the expected loads, an engineering assessment shall be required. The engineering assessment shall involve either a structural analysis or an in-situ load test, or both. The structural analysis shall be based on actual material properties and other as-built conditions that affect stability or load-bearing capacity, and shall be conducted in accordance with the applicable design standard. If the structural assessment determines that the load-bearing capacity is less than that required by the code, load tests shall be conducted in accordance with 780 CMR 1624.2. If the building, structure or portion thereof is found to have inadequate stability or load-bearing capacity for the expected loads, modifications to ensure structural adequacy or the removal of the inadequate construction shall be required.

1624.2 Test Standards. Structural components and assemblies shall be tested in accordance with the appropriate material standards listed in 780 CMR 35.00. In the absence of a standard that contains an applicable load test procedure, the test procedure shall be developed by a registered design

professional and approved. The test procedure shall simulate loads and conditions of application that the completed structure or portion thereof will be subjected to in normal use.

1624.3 *In-situ* Load Tests. In-situ load tests shall be conducted in accordance with 780 CMR 1624.3.1 or 1624.3.2 and shall be supervised by a registered design professional. The test shall simulate the applicable loading conditions specified in 780 CMR 16.00 as necessary to address the concerns regarding structural stability of the building, structure or portion thereof.

1624.3.1 Load Test Procedure Specified.

Where a standard listed in 780 CMR 35.00 contains an applicable load test procedure and acceptance criteria, the test procedure and acceptance criteria in the standard shall apply. In the absence of specific load factors or acceptance criteria, the load factors and acceptance criteria in 780 CMR 1624.3.2 shall apply.

1624.3.2 Load Test Procedure Not Specified.

In the absence of applicable load test procedures contained within a standard referenced by 780 CMR or acceptance criteria for a specific material or method of construction, such existing structure shall be subjected to a test procedure developed by a registered design professional that simulates applicable loading and deformation conditions. For components that are not a part of the seismic load-resisting system, the test load shall be equal to two times the unfactored design loads. The test load shall be left in place for a period of 24 hours. The structure shall be considered to have successfully met the test requirements where the following criteria are satisfied:

1. Under the design load, the deflection shall not exceed the limitations specified in 780 CMR 1604.3.
2. Within 24 hours after removal of the test load, the structure shall have recovered not less than 75% of the maximum deflection.
3. During and immediately after the test, the structure shall not show evidence of failure.

780 CMR 1625.0 ALTERNATIVE TEST PROCEDURE

1625.1 General. For materials and assemblies that are not specifically provided for in 780 CMR, the design strengths and permissible stresses shall be established by tests or a combination of tests and structural analysis.

1625.2 Tests by Approved Agencies. The building official shall accept duly authenticated reports from approved agencies in respect to the quality and manner of use of new materials or assemblies as provided for in 780 CMR 109.4.

1625.3 Cost of Tests. The cost of all tests and other investigations required under the provisions of

780 CMR shall be borne by the permit applicant.

780 CMR 1626.0 TEST SAFE LOAD

1626.1 Where Required. Where proposed construction is not capable of being designed by approved engineering analysis, or where proposed construction design method does not comply with the applicable material design standard, the system of construction or the structural unit and the connections shall be subjected to the tests prescribed in 780 CMR 1627.0. The building official shall accept certified reports of such tests conducted by an approved testing agency, provided that such tests meet the requirements of 780 CMR and approved procedures.

780 CMR 1627.0 PRECONSTRUCTION LOAD TESTS

1627.1 General. In evaluating the physical properties of materials and methods of construction that are not capable of being designed by approved engineering analysis or that do not comply with applicable material design standards listed in 780 CMR 35.00, the structural adequacy shall be predetermined based on the load test criteria established in 780 CMR 1627.0.

1627.2 Load Test Procedures Specified. Where specific load test procedures, load factors and acceptance criteria are included in the applicable design standards listed in 780 CMR 35.00, such test procedures, load factors and acceptance criteria shall apply. In the absence of specific test procedures, load factors or acceptance criteria, the corresponding provisions in 780 CMR 1627.3 shall apply.

1627.3 Load Test Procedures Not Specified. Where load test procedures are not specified in the applicable design standards listed in 780 CMR 35.00, the load-bearing and deformation capacity of structural components and assemblies shall be determined on the basis of a test procedure developed by a registered design professional that simulates applicable loading and deformation conditions. For components and assemblies that are not a part of the seismic load-resisting system, the test shall be as specified in 780 CMR 1627.3.1. Load tests shall simulate the applicable loading conditions specified in 780 CMR 16.00.

1627.3.1 Test Procedure. The test assembly shall be subjected to an increasing superimposed load equal to not less than two times the superimposed design load. The test load shall be left in place for a period of 24 hours. The tested assembly shall be considered to have successfully met the test requirements if the assembly recovers not less than 75% of the maximum deflection within 24 hours after the removal of the test load. The test assembly shall then be reloaded and subjected to an increasing superimposed load until

either structural failure occurs or the superimposed load is equal to two and one-half times the load at which the deflection limitations specified in 780 CMR 1627.3.2 were reached, or the load is equal to two and one-half times the superimposed design load. In the case of structural components and assemblies for which deflection limitations are not specified in 780 CMR 1627.3.2, the test specimen shall be subjected to an increasing superimposed load until structural failure occurs or the load is equal to two and one-half times the desired superimposed design load. The allowable superimposed design load shall be taken as the lesser of:

1. The load at the deflection limitation given in 780 CMR 1627.3.2.
2. The failure load divided by 2.5.
3. The maximum load applied divided by 2.5.

1627.3.2 Deflection. The deflection of structural members under the design load shall not exceed the limitations in 780 CMR 1604.3.

1627.4 Wall and Partition Assemblies. Load-bearing wall and partition assemblies shall sustain the test load both with and without window framing. The test load shall include all design load components. Wall and partition assemblies shall be tested both with and without door and window framing.

1627.5 Exterior Window and Door Assemblies. The design pressure rating of exterior windows and doors in buildings shall be determined in accordance with 780 CMR 1627.5.1 or 1627.5.2.

Exception. Structural wind load design pressures for window units smaller than the size tested in accordance with 780 CMR 1627.5.1 or 1627.5.2 shall be permitted to be higher than the design value of the tested unit provided such higher pressures are determined by accepted engineering analysis. All components of the small unit shall be the same as the tested unit. Where such calculated design pressures are used, they shall be validated by an additional test of the window unit having the highest allowable design pressure.

1627.5.1 Aluminum, Vinyl and Wood Exterior Windows and Glass Doors. Aluminum, vinyl and wood exterior windows and glass doors shall be labeled as conforming to AAMA/NWWDA 101/I.S.2 or 101/I.S.2/NAFS. The label shall state the name of the manufacturer, the approved labeling agency and the product designation as specified in AAMA/NWWDA 101/I.S.2 or 101/I.S.2/NAFS. Products tested and labeled as conforming to AAMA/NWWDA 101/I.S.2 or 101/I.S.2/NAFS shall not be subject to the requirements of 780 CMR 2403.2 and 2403.3.

1627.5.2 Exterior Windows and Door Assemblies Not Provided for in 780 CMR 1627.5.1. Exterior window and door assemblies shall be tested in accordance with ASTM E 330.

THE MASSACHUSETTS STATE BUILDING CODE

Exterior window and door assemblies containing glass shall comply with 780 CMR 2403. The design pressure for testing shall be calculated in accordance with 780 CMR 16.00. Each assembly shall be tested for ten seconds at a load equal to 1.5 times the design pressure.

1627.6 Test Specimens. Test specimens and construction shall be representative of the materials,

workmanship and details normally used in practice. The properties of the materials used to construct the test assembly shall be determined on the basis of tests on samples taken from the load assembly or on representative samples of the materials used to construct the load test assembly. Required tests shall be conducted or witnessed by an approved agency.