CHAPTER 11
SEISMIC DESIGN CRITERIA

11.1 GENERAL

11.1.1 Purpose Chapter 11 presents criteria for the design and construction of buildings and other structures subject to earthquake ground motions. The specified earthquake loads are based on postelastic energy dissipation in the structure. Because of this fact, the requirements for design, detailing, and construction shall be satisfied, even for structures and members for which load combinations that do not include earthquake loads indicate larger demands than combinations that include earthquake loads.

11.1.2 Scope Every structure and portion thereof, including nonstructural components, shall be designed and constructed to resist the effects of earthquake motions as prescribed by the seismic requirements of this standard. Certain nonbuilding structures, as described in Chapter 15, are also within the scope and shall be designed and constructed in accordance with the requirements of Chapter 15. The following structures are exempt from the seismic requirements of this standard:

1. Detached one- and two-family dwellings that are located where the mapped, short period, spectral response acceleration parameter, S_S, is less than 0.4 or where the seismic design category determined in accordance with Section 11.6 is A, B, or C.
2. Detached one- and two-family wood-frame dwellings not included in Exemption 1 with not more than two stories above grade plane, satisfying the limitations of and constructed in accordance with the IRC.
3. Agricultural storage structures that are intended only for incidental human occupancy.
4. Nonbuilding structures that require special consideration of their response characteristics and environment that are not addressed in Chapter 15 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances, and nuclear reactors.
5. Piers and wharves that are not accessible to the general public.

11.1.3 Applicability Structures and their nonstructural components shall be designed and constructed in accordance with the requirements of the following chapters based on the type of structure or component:

Buildings: Chapter 12
Nonbuilding structures: Chapter 15
Nonstructural components: Chapter 13
Seismically isolated structures: Chapter 17
Structures with damping systems: Chapter 18

Buildings whose purpose is to enclose equipment or machinery and whose occupants are engaged in maintenance or monitoring of that equipment, machinery, or their associated processes shall be permitted to be classified as nonbuilding structures designed and detailed in accordance with Section 15.5 of this standard.

11.1.4 Alternate Materials and Methods of Construction Alternate materials and methods of construction to those prescribed in the seismic requirements of this standard shall not be used unless approved by the Authority Having Jurisdiction. Substantiating evidence shall be submitted demonstrating that the proposed alternate will be at least equal in strength, durability, and seismic resistance for the purpose intended.

11.1.5 Quality Assurance Quality assurance for seismic force-resisting systems and other designated seismic systems defined in Section 13.2.2 shall be provided in accordance with the requirements of the Authority Having Jurisdiction.

Where the Authority Having Jurisdiction has not adopted quality assurance requirements, or where the adopted requirements are not applicable to the seismic force-resisting system or designated seismic systems as described in Section 13.2.2, the registered design professional in responsible charge of designing the seismic force-resisting system or other designated seismic systems shall submit a quality assurance plan to the Authority Having Jurisdiction for approval. The quality assurance plan shall specify the quality assurance program elements to be implemented.

11.2 DEFINITIONS

The following definitions apply only to the seismic provisions of Chapters 11 through 22 of this standard.

ACTIVE FAULT: A fault determined to be active by the Authority Having Jurisdiction from properly substantiated data (e.g., most recent mapping of active faults by the US Geological Survey).

ADDITION: An increase in building area, aggregate floor area, height, or number of stories of a structure.

ALTERATION: Any construction or renovation to an existing structure other than an addition.

APPENDAGE: An architectural component such as a canopy, marquee, ornamental balcony, or statuary.

APPROVAL: The written acceptance by the Authority Having Jurisdiction of documentation that establishes the qualification of a material, system, component, procedure, or person to fulfill the requirements of this standard for the intended use.

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

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

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

BOUNDARY ELEMENTS: Portions along wall and diaphragm edges for transferring or resisting forces. Boundary elements include chords and collectors at diaphragm and shear wall perimeters, edges of openings, discontinuities, and reentrant corners.

BUILDING: Any structure whose intended use includes shelter of human occupants.

CANTILEVERED COLUMN SYSTEM: A seismic force-resisting system in which lateral forces are resisted entirely by columns acting as cantilevers from the base.

CHARACTERISTIC EARTHQUAKE: An earthquake assessed for an active fault having a magnitude equal to the best estimate of the maximum magnitude capable of occurring on the fault but not less than the largest magnitude that has occurred historically on the fault.

COLLECTOR (DRAG STRUT, TIE, DIAPHRAGM STRUT): A diaphragm or shear wall boundary element parallel to the applied load that collects and transfers diaphragm shear forces to the vertical elements of the seismic force-resisting system or distributes forces within the diaphragm or shear wall.

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

Component, Flexible: Nonstructural component that has a fundamental period greater than 0.06 s.

Component, Nonstructural: A part of an architectural, mechanical, or electrical system within or without a building or nonbuilding structure.

Component, Rigid: Nonstructural component that has a fundamental period less than or equal to 0.06 s.

Component, Rugged: A nonstructural component that has been shown to consistently function after design earthquake level or greater seismic events (based on past earthquake experience data or past seismic testing) when adequately anchored or supported. The classification of a nonstructural component as rugged shall be based on a comparison of the specific component with components of similar strength and stiffness. Common examples of rugged components are AC motors, compressors, and base-mounted horizontal pumps.

CONCRETE:

Plain Concrete: Concrete that is either unreinforced or contains less reinforcement than the minimum amount specified in ACI 318 for reinforced concrete.

Reinforced Concrete: Prestressed or nonprestressed concrete reinforced with no less reinforcement than the minimum amount required by ACI 318 and designed on the assumption that the two materials act together in resisting forces.

CONSTRUCTION DOCUMENTS: The written, graphic, electronic, and pictorial documents describing the design, locations, and physical characteristics of the project required to verify compliance with this standard.

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

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

High-Deformability Element: An element whose deformability is not less than 3.5 where subjected to four fully reversed cycles at the limit deformation.

Limited-Deformability Element: An element that is neither a low-deformability nor a high-deformability element.

Low-Deformability Element: An element whose deformability is 1.5 or less.

DEFORMATION:

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

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

DESIGN EARTHQUAKE: The earthquake effects that are two-thirds of the corresponding risk-targeted maximum considered earthquake (MCE_R).

DESIGN EARTHQUAKE DISPLACEMENT: *See* DISPLACEMENT AND DRIFT. *See* DISPLACEMENT AND DRIFT. *See* **DESIGN STORY DRIFT**.

DESIGN EARTHQUAKE GROUND MOTION: The earthquake ground motions that are two-thirds of the corresponding MCE_R ground motions.

DESIGNATED SEISMIC SYSTEMS: Those nonstructural components that require design in accordance with Chapter 13 and for which the component Importance Factor, I_p, is greater than 1.0.

DIAPHRAGM: Roof, floor, or other membrane or bracing system acting to transfer the lateral forces to the vertical resisting elements.

Flexure-Controlled Diaphragm: Diaphragm with a flexural yielding mechanism, which limits the maximum forces that develop in the diaphragm, and a design shear strength or factored nominal shear capacity greater than the shear corresponding to the nominal flexural strength.

Shear-Controlled Diaphragm: Diaphragm that does not meet the requirements of a flexure-controlled diaphragm.

Transfer Forces, Diaphragm: Forces that occur in a diaphragm caused by transfer of seismic forces from the vertical seismic force-resisting elements above the diaphragm to other vertical seismic force-resisting elements below the diaphragm because of offsets in the placement of the vertical elements or changes in relative lateral stiffnesses of the vertical elements.

Vertical Diaphragm: *See* WALL: Shear Wall.

DIAPHRAGM BOUNDARY: A location where shear is transferred into or out of the diaphragm element. Transfer is either to a boundary element or to another force-resisting element.

DIAPHRAGM CHORD: A diaphragm boundary element perpendicular to the applied load that is assumed to take axial stresses caused by the diaphragm moment.

DIAPHRAGM DEFORMATION: The relative horizontal displacement of portions of a diaphragm due to strain in the diaphragm elements and not due to deformations of the vertical elements of the seismic-force resisting system or to diaphragm-rotation effects.

DIAPHRAGM-ROTATION EFFECTS: Relative horizontal displacement of portions of a diaphragm due to unequal deformations of the vertical elements of the seismic-force resisting system.

DISPLACEMENT AND DRIFT:

Design Earthquake Displacement: The displacement at a given location of the structure corresponding to the Design Earthquake.

Design Story Drift: The story drift corresponding to the Design Earthquake, taken at a representative plan location (center of mass or building perimeter, as required by Section 12.8.6).

Maximum Considered Earthquake Displacement: The displacement at a given location of the structure corresponding to the Risk-Targeted Maximum Considered Earthquake (MCE_R).

Story Drift: The horizontal displacement at the top of the story relative to the bottom of the story at vertically aligned points corresponding to the given loading.

Story Drift Ratio: The story drift divided by the story height, h_{sx}.

DISTRIBUTION SYSTEM: An interconnected system composed primarily of linear components including piping, tubing, conduit, raceways, or ducts. Distribution systems include in-line components such as valves, in-line suspended pumps, and mixing boxes.

ELEMENT ACTION: Element axial, shear, or flexural behavior.

Critical Action: An action, failure of which would result in the collapse of multiple bays or multiple stories of the building or would result in a significant reduction in the structure's seismic resistance.

Deformation-Controlled Action: Element actions for which reliable inelastic deformation capacity is achievable without critical strength decay.

Force-Controlled Action: Any element actions modeled with linear properties and element actions not classified as deformation-controlled.

Noncritical Actions: An action, failure of which would not result in either collapse or significant loss of the structure's seismic resistance.

Ordinary Action: An action, failure of which would result in only local collapse, comprising not more than one bay in a single story, and would not result in a significant reduction of the structure's seismic resistance.

ENCLOSURE: An interior space surrounded by walls.

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

Equipment Support Structures and Platforms: Assemblies of members or manufactured elements other than integral supports, including, but not limited to, moment frames, braced frames, skids, legs longer than 24 in. (600 mm), or walls that support one or more nonstructural components or systems.

Distribution System Support: Members that provide vertical or lateral seismic resistance for distribution systems, including but not limited to, hangers, braces, pipe racks, and trapeze assemblies.

Equipment Support, Integral: Assemblies of members or manufactured elements and their associated attachments and base plates that provide vertical or lateral support for nonstructural components, are directly connected to both the nonstructural component and the attachment to the structure or foundation, and where the nonstructural component acts as part of the lateral force resisting system of the equipment support. Integral equipment supports include but are not limited to legs less than or equal to 24 in. (600 mm) in length, lugs, skirts, and saddles.

FLEXIBLE CONNECTIONS: Those connections between equipment components that permit rotational and/or translational movement without degradation of performance. Examples include universal joints, bellows expansion joints, and flexible metal hose.

FOUNDATION GEOTECHNICAL CAPACITY: The maximum pressure or strength design capacity of a foundation based on the supporting soil, rock, or controlled low-strength material.

FOUNDATION STRUCTURAL CAPACITY: The design strength of foundations or foundation components as provided by adopted material standards and as altered by the requirements of this standard.

FRAME:

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

Concentrically Braced Frame (CBF): A braced frame in which the members are subjected primarily to axial forces. CBFs are categorized as ordinary concentrically braced frames (OCBFs) or special concentrically braced frames (SCBFs).

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

Moment Frame: A frame in which members and joints resist lateral forces by flexure and along the axis of the members. Moment frames are categorized as intermediate moment frames (IMFs), ordinary moment frames (OMFs), and special moment frames (SMFs).

STRUCTURAL SYSTEM:

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

Dual System: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by moment-resisting frames and shear walls or braced frames as prescribed in Section 12.2.5.1.

Shear Wall–Frame Interactive System: A structural system that uses combinations of ordinary reinforced concrete shear walls and ordinary reinforced concrete moment frames designed to resist lateral forces in proportion to their rigidities considering interaction between shear walls and frames on all levels.

Space Frame System: A 3-D structural system composed of interconnected members, other than bearing walls, that is capable of supporting vertical loads and, where designed for such an application, is capable of providing resistance to seismic forces.

FRICTION CLIP: A device that relies on friction to resist applied loads in one or more directions to anchor a nonstructural component. Friction is provided mechanically and is not due to gravity loads.

GLAZED CURTAIN WALL: A nonbearing wall that extends beyond the edges of building floor slabs and includes a glazing material installed in the curtain wall framing.

GLAZED STOREFRONT: A nonbearing wall that is installed between floor slabs, typically including entrances, and includes a glazing material installed in the storefront framing.

GRADE PLANE: A horizontal reference plane representing the average of finished ground level adjoining the structure at all exterior walls. Where the finished ground level slopes away from the exterior walls, the grade plane is established by the lowest points within the area between the structure and the property line or, where the property line is more than 6 ft (1,829 mm) from the structure, between the structure and points 6 ft (1,829 mm) from the structure.

HEATING, VENTILATING, AIR-CONDITIONING, AND REFRIGERATION (HVACR): The equipment, distribution systems, and terminals, excluding interconnecting piping and ductwork that provide, either collectively or individually, the processes of heating, ventilating, air-conditioning, or refrigeration to a building or portion of a building.

INSPECTION, SPECIAL: The observation of the work by a special inspector to determine compliance with the approved construction documents and these standards in accordance with the quality assurance plan.

Continuous Special Inspection: The full-time observation of the work by a special inspector who is present in the area where work is being performed.

Periodic Special Inspection: The part-time or intermittent observation of the work by a special inspector who is present in the area where work has been or is being performed.

INSPECTOR, SPECIAL: A person approved by the Authority Having Jurisdiction to perform special inspection, and who shall be identified as the owner's inspector.

INVERTED PENDULUM-TYPE STRUCTURES: Structures in which more than 50% of the structure's mass is concentrated at the top of a slender, cantilevered structure and in which the stability of the mass at the top of the structure relies on rotational restraint to the top of the cantilevered element.

JOINT: The geometric volume common to intersecting members.

LIGHT-FRAME CONSTRUCTION: A method of construction where the structural assemblies (e.g., walls, floors, ceilings, and roofs) are primarily formed by a system of repetitive wood or cold-formed steel framing members or subassemblies of these members (e.g., trusses).

LONGITUDINAL REINFORCEMENT RATIO: Area of longitudinal reinforcement divided by the cross-sectional area of the concrete.

MAXIMUM CONSIDERED EARTHQUAKE DISPLACEMENT: *See* DISPLACEMENT AND DRIFT

MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION: The most severe earthquake effects considered by this standard, more specifically defined in the following two terms:

Maximum Considered Earthquake Geometric Mean (MCE_G) **Peak Ground Acceleration:** The most severe earthquake effects considered by this standard determined for geometric mean peak ground acceleration and without adjustment for targeted risk. The MCE_G peak ground acceleration adjusted for site effects (PGA_M) is used in this standard for evaluation of liquefaction, lateral spreading, seismic settlements, and other soil-related issues. In this standard, general procedures for determining PGA_M are provided in Section 11.8.3; site-specific procedures are provided in Section 21.5.

Risk-Targeted Maximum Considered Earthquake (MCE_R) **Ground Motion Response Acceleration:** The most severe earthquake effects considered by this standard determined for the orientation that results in the largest maximum response to horizontal ground motions and with adjustment for targeted risk. In this standard, general procedures for determining the MCE_R ground motion values are provided in Section 11.4.4; site-specific procedures are provided in Sections 21.1 and 21.2.

MECHANICALLY ANCHORED TANKS OR VESSELS: Tanks or vessels provided with mechanical anchors to resist overturning moments.

NONBUILDING STRUCTURE: A structure, other than a building, constructed of a type included in Chapter 15 and within the limits of Section 15.1.1.

NONBUILDING STRUCTURE SIMILAR TO A BUILDING: A nonbuilding structure that is designed and constructed in a manner similar to buildings, responds to strong ground motion in a fashion similar to buildings, and has a basic lateral and vertical seismic force-resisting system conforming to one of the types indicated in Table 12.2-1 or 15.4-1.

OPEN-TOP TANK: A tank without a fixed roof or cover, floating cover, gas holder cover, or dome.

ORTHOGONAL: In two horizontal directions, at 90 degrees to each other.

OWNER: Any person, agent, firm, or corporation that has a legal or equitable interest in a property.

P-DELTA EFFECT: The secondary effect on shears and moments of structural members caused by the action of the vertical loads induced by horizontal displacement of the structure resulting from various loading conditions.

PARTITION: A nonstructural interior wall that spans horizontally or vertically from support to support. The supports may be the basic building frame, subsidiary structural members, or other portions of the partition system.

PILE: Deep foundation element, which includes piers, caissons, and piles.

PILE CAP: Foundation elements to which piles are connected, including grade beams and mats.

PREMANUFACTURED MODULAR MECHANICAL AND ELECTRICAL SYSTEM: A prebuilt, fully or partially enclosed assembly of mechanical and electrical components.

REGISTERED DESIGN PROFESSIONAL: An architect or engineer registered or licensed to practice professional architecture or engineering, as defined by the statutory requirements of the professional registration laws of the state in which the project is to be constructed.

REINFORCED CONCRETE DUCTILE COUPLED WALL: A seismic force-resisting system as defined in ACI 318 Section 2.3 and complying with ACI 318 Section 18.10.9.

SEISMIC DESIGN CATEGORY: A classification assigned to a structure based on its Risk Category and the severity of the design earthquake ground motion at the site, as defined in Section 11.4.

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

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

SELF-ANCHORED TANKS OR VESSELS: Tanks or vessels that are stable under design overturning moment without the need for mechanical anchors to resist uplift.

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

SITE CLASS: A classification assigned to a site based on the types of soils present and their engineering properties, as defined in Chapter 20.

STORAGE RACKS, STEEL: A framework or assemblage, comprised of cold-formed or hot-rolled steel structural members, intended for storage of materials, including, but not limited to, pallet storage racks, selective racks, movable-shelf racks, rack-supported systems, automated storage and retrieval systems (stacker racks), push-back racks, pallet-flow racks, case-flow racks, pick modules, and rack-supported platforms. Other types of racks, such as drive-in or drive-through racks, cantilever racks, portable racks, or racks made of materials other than steel, are not considered steel storage racks for the purpose of this standard.

STORAGE RACKS, STEEL CANTILEVERED: A framework or assemblage comprised of cold-formed or hot-rolled steel structural members, primarily in the form of vertical columns, extended bases, horizontal arms projecting from the faces of the columns, and longitudinal (down-aisle) bracing between columns. There may be shelf beams between the arms, depending on the products being stored; this definition does not include other types of racks such as pallet storage racks, drive-in racks, drive-through racks, or racks made of materials other than steel.

STORY: The portion of a structure between the tops of two successive floor surfaces and, for the topmost story, from the top of the floor surface to the top of the roof surface.

STORY ABOVE GRADE PLANE: A story in which the floor or roof surface at the top of the story is more than 6 ft (1,829 mm) above grade plane or is more than 12 ft (3,658 mm) above the finished ground level at any point on the perimeter of the structure.

STORY DRIFT: *See* DISPLACEMENT AND DRIFT

STORY SHEAR: The summation of design lateral seismic forces at levels above the story under consideration.

STRENGTH:

Design Strength: Nominal strength multiplied by a strength reduction factor, ϕ.

Nominal Strength: Strength of a member or cross section calculated in accordance with the requirements and assumptions of the strength design methods of this standard (or the reference documents) before application of any strength-reduction factors.

Required Strength: 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 this standard.

STRUCTURAL HEIGHT: The vertical distance from the base to the highest level of the seismic force-resisting system of the structure. For pitched or sloped roofs, the structural height is from the base to the average height of the roof.

STRUCTURAL OBSERVATIONS: The visual observations to determine that the seismic force-resisting system is constructed in general conformance with the construction documents.

STRUCTURE: That which is built or constructed and limited to buildings and nonbuilding structures as defined herein.

SUBDIAPHRAGM: A portion of a diaphragm used to transfer wall anchorage forces to diaphragm crossties.

SUPPORTS: Those members, assemblies of members, or manufactured elements, including braces, frames, legs, lugs, snubbers, hangers, saddles, or struts, and associated fasteners that transmit loads between nonstructural components and their attachments to the structure.

TESTING AGENCY: A company or corporation that provides testing and/or inspection services.

TRUSSED TOWER: A lattice-type structure, freestanding or guyed, which supports static equipment such as chimneys, stacks, lights, or other lightweight components.

USGS SEISMIC DESIGN GEODATABASE: A US Geological Survey (USGS) database of geocoded values of seismic design parameters S_S, S_1, S_{MS}, S_{M1}, and PGA_M and geocoded sets of multi-period 5%-damped risk-targeted maximum considered earthquake (MCE_R) response spectra.

> **User Note:** The USGS Seismic Design Geodatabase is intended to be accessed through a USGS Seismic Design Web Service that allows the user to specify the site location, by latitude and longitude, and the site class to obtain the seismic design data. The USGS web service spatially interpolates between the gridded data of the USGS geodatabase. Both the USGS geodatabase and the USGS web service can be accessed at https://doi.org/10.5066/F7NK3C76. The USGS Seismic Design Geodatabase is available at the ASCE 7 Hazard Tool https://asce7hazardtool.online/ or an approved equivalent.

VENEERS: Facings or ornamentation of brick, concrete, stone, tile, or similar materials attached to a backing.

WALL: A component that has a slope of 60 degrees or greater with the horizontal plane used to enclose or divide space.

Bearing Wall: Any wall meeting either of the following classifications:

1. Any metal or wood stud wall that supports more than 100 lb per linear ft (1,459 N/m) of vertical load in addition to its own weight.
2. Any concrete or masonry wall that supports more than 200 lb per linear ft (2,919 N/m) of vertical load in addition to its own weight.

Light Frame Wall: A wall with wood or steel studs.

Light Frame Wood Shear Wall: A wall constructed with wood studs and sheathed with material rated for shear resistance.

Nonbearing Wall: Any wall that is not a bearing wall.

Nonstructural Wall: A wall other than a bearing wall or shear wall.

Shear Wall (Vertical Diaphragm): A wall, bearing or nonbearing, designed to resist lateral forces acting in the plane of the wall (sometimes referred to as a "vertical diaphragm").

Structural Wall: A wall that meets the definition for bearing wall or shear wall.

WALL SYSTEM:

BEARING: A structural system with bearing walls providing support for all or major portions of the vertical loads. Shear walls or braced frames provide seismic force resistance.

WOOD STRUCTURAL PANEL: A wood-based panel product that meets the requirements of DOC PS1 or DOC PS2 and is bonded with a waterproof adhesive. Included under this designation are plywood, oriented strand board, and composite panels.

11.3 SYMBOLS

The unit dimensions used with the items covered by the symbols shall be consistent throughout except where specifically noted. Symbols presented in this section apply only to the seismic provisions of Chapters 11 through 22 in this standard.

A_0 = Area of the load-carrying foundation, ft^2 (m^2)

A_{ch} = Cross-sectional area, in.^{2} (mm^2), of a structural member measured out-to-out of transverse reinforcement

A_{sh} = Total cross-sectional area of hoop reinforcement, in.^{2} (mm^2), including supplementary crossties, having a spacing of s_h and crossing a section with a core dimension of h_c

A_{vd} = Required area of leg of diagonal reinforcement, in.^{2} (mm^2)

A_x = Torsional amplification factor (Section 12.8.4.3)

a_i = Acceleration at level i obtained from a modal analysis (Section 13.3.1)

a_i = the maximum acceleration at level i obtained from the nonlinear response history analysis at the Design Earthquake ground motion

b_p = Width of the rectangular glass panel

C_{AR} = Component resonance ductility factor that converts the peak floor or ground acceleration into the peak component acceleration as determined in Section 13.3.1.3

C_d = Deflection amplification factor as given in Table 12.2-1, 15.4-1, or 15.4-2

C_{dX} = Deflection amplification factor in the X direction (Section 12.9.2.5)

C_{dY} = Deflection amplification factor in the Y direction (Section 12.9.2.5)

$C_{s\text{-diaph}}$ = Seismic response coefficient for design of diaphragms using the alternative diaphragm design method of Section 12.10.4

$C_{d\text{-}diaph}$ = Deflection amplification factor for diaphragm deflection (12.10.4)
C_{p0} = Diaphragm design acceleration coefficient at the structure base (Section 12.10.3.2.1)
C_{pi} = Diaphragm design acceleration coefficient at 80% of the structural height above the base, h_n (Section 12.10.3.2.1)
C_{pn} = Diaphragm design acceleration coefficient at the structural height, h_n (Section 12.10.3.2.1)
C_{px} = Diaphragm design acceleration coefficient at level x (Section 12.10.3.2.1)
C_s = Seismic response coefficient determined in Section 12.8.1.1 or 19.3.1 (dimensionless)
C_{s2} = Higher mode seismic response coefficient (Section 12.10.3.2.1)
C_t = Building period coefficient (Section 12.8.2.1)
C_v = Vertical response spectral coefficient as given in Table 11.9-1
C_{vs} = Coefficient of variation of soil shear modulus, defined as the standard deviation divided by the mean (Section 12.13.3)
C_{vx} = Vertical distribution factor as determined in Section 12.8.3
c = Distance from the neutral axis of a flexural member to the fiber of maximum compressive strain, in. (mm)
D = Effect of dead load
D_{clear} = Relative horizontal (drift) displacement, measured over the height of the glass panel under consideration, which causes initial glass-to-frame contact. For rectangular glass panels within a rectangular wall frame, D_{clear} is set forth in Section 13.5.9.1
D_{pI} = Seismic relative displacement (Section 13.3.2)
D_s = Total depth of stratum in Equation (19.3-4), ft (m)
d_c = Total thickness of cohesive soil layers in the top 100 ft (30 m), ft (m) (Section 20.4.3)
d_i = Thickness of any soil or rock layer i [between 0 and 100 ft (0 and 30 m)]; see Section 20.4.1
d_S = Total thickness of cohesionless soil layers in the top 100 ft (30 m), ft (m) (Section 20.4.2)
E = Effect of horizontal and vertical earthquake-induced forces (Section 12.4)
E_{cl} = Capacity-limited horizontal seismic load effect, equal to the maximum force that can develop in the element as determined by a rational, plastic mechanism analysis
F_i, F_n, F_x = Portion of the seismic base shear, V, induced at level i, n, or x, respectively, as determined in Section 12.8.3
F_{md} = Factor to convert the geometric mean spectral ordinate to a maximum direction spectral ordinate
F_p = Seismic force acting on a component of a structure as determined in Sections 12.11.1 and 13.3.1
F_{px} = Diaphragm seismic design force at level x
f'_c = Specified compressive strength of concrete used in design
f'_s = Ultimate tensile strength of the bolt, stud, or insert leg wires, psi (MPa). For ASTM A307 bolts or ASTM A108 studs, it is permitted to be assumed to be 60,000 psi (415 MPa)
f_y = Specified yield strength of reinforcement, psi (MPa)
f_{yh} = Specified yield strength of the special lateral reinforcement, psi (kPa)
$G = \gamma v_s^2/g$ = Average shear modulus for the soils beneath the foundation at large strain levels, lb/ft² (Pa)
$G_0 = \gamma v_{s0}^2/g$ = Average shear modulus for the soils beneath the foundation at small strain levels, lb/ft² (Pa)
g = Acceleration due to gravity
H = Thickness of soil
H_f = Factor for force amplification as a function of height in the structure as determined in Section 13.3.1.1
h = Height of a shear wall measured as the maximum clear height from top of foundation to bottom of diaphragm framing above, or the maximum clear height from top of diaphragm to bottom of diaphragm framing above
h = Average roof height of structure with respect to the base (Chapter 13)
h^* = Effective height of the building, ft (m), as determined in Chapter 19
h_c = Core dimension of a component measured to the outside of the special lateral reinforcement, in. (mm)
h_i, h_x = Height above the base to level i or x, respectively
h_n = Structural height as defined in Section 11.2
h_p = Height of the rectangular glass panel
h_{sx} = Story height below level $x = (h_x - h_{x-1})$
I_e = Importance Factor as prescribed in Section 11.5.1
I_p = Component Importance Factor as prescribed in Section 13.3.1
i = Building level referred to by a subscript i; $i = 1$ designates the first level above the base
K_p = Stiffness of the component or attachment (Section 13.3.3)
K_{xx}, K_{rr} = Rotational foundation stiffness in Equations (19.3-9) and (19.3-19), ft-lb/degree (N-m/rad)
K_y, K_r = Translational foundational stiffness in Equations (19.3-8) and (19.3-18), lb/in. (N/m)
KL/r = Lateral slenderness ratio of a compression member measured in terms of its effective length, KL, and the least radius of gyration of the member cross section, r
k = Distribution exponent, given in Section 12.8.3
k_a = Coefficient for amplification factor for diaphragm flexibility defined in Sections 12.11.2.1 and 12.14.7.5
L = Overall length of the building at the base in the direction being analyzed, ft (m)
L_{diaph} = Span in feet of the horizontal diaphragm or diaphragm segment being considered, measured between vertical elements or collectors that provide support to the diaphragm or diaphragm segment (Section 12.10.4)
M_t = Torsional moment resulting from eccentricity between the locations of the center of mass and the center of rigidity (Section 12.8.4.1)
M_{ta} = Accidental torsional moment as determined in Section 12.8.4.2
m = Subscript denoting the mode of vibration under consideration; $m = 1$ for the fundamental mode
N = Standard penetration resistance per ASTM D1586
N = Number of stories above the base (Section 12.8.2.1)
$\overline{N}$ = Average field standard penetration resistance for the top 100 ft (30 m) (Sections 20.3.3 and 20.4.2)
$\overline{N}_{ch}$ = Average standard penetration resistance for cohesionless soil layers for the top 100 ft (30 m) (Sections 20.3.3 and 20.4.2)
N_i = Standard penetration resistance of any soil or rock layer i [between 0 and 100 ft (0 and 30 m)]; see Section 20.4.2
n = Designation for the level that is uppermost in the main portion of the building
PGA_G = Lower-bound limit on deterministic maximum considered earthquake geometric mean peak ground acceleration (Table 21.2-1)

PGA_M = Mapped MCE_G peak ground acceleration as defined in Section 11.8.3

PI = Plasticity index per ASTM D4318

P_x = Total unfactored vertical design load at and above level x, for use in Section 12.8.7

Q_E = Effect of horizontal seismic (earthquake-induced) forces

R = Response modification coefficient as given in Tables 12.2-1, 12.14-1, 15.4-1, and 15.4-2

R_{diaph} = Response modification coefficient for design of diaphragms using the alternative diaphragm design method of Section 12.10.4

R_{po} = Component strength factor as determined in Section 13.3.1.4

R_μ = Structure ductility reduction factor as determined in Section 13.3.1.2

R_s = Diaphragm design force reduction factor (Section 12.10.3.5)

R_X = Response modification coefficient in the X direction (Section 12.9.2.5)

R_Y = Response modification coefficient in the Y direction (Section 12.9.2.5)

S_1 = MCE_R, 5% damped, spectral response acceleration parameter at a period of 1 s for Site Class BC site conditions as determined in accordance with Section 11.4.3

S_a = 5% damped design spectral response acceleration parameter at any period as defined in Section 11.4.5

S_{aM} = Site-specific, 5% damped, MCE_R spectral response acceleration parameter at any period

S_{av} = 5% damped, design vertical response spectral acceleration parameter at any period as determined in accordance with Section 11.9.3

S_{aMv} = 5% damped, MCE_R vertical response spectral acceleration parameter at any period as determined in accordance with Section 11.9.2

S_{D1} = Design, 5% damped, spectral response acceleration parameter at a period of 1 s as defined in Section 11.4.4

S_{DpS} = Design, 5% damped, spectral response acceleration parameter at short periods as defined in Section 11.4.4

S_{M1} = MCE_R, 5% damped, spectral response acceleration parameter at a period of 1 s adjusted for site class effects as defined in determined in accordance with Section 11.4.3

S_{MS} = MCE_R, 5% damped, spectral response acceleration parameter at short periods adjusted for site class effects as determined in accordance with Section 11.4.3

S_S = MCE_R, 5% damped, spectral response acceleration parameter at a period of 0.2 s for Site Class BC site conditions as determined in accordance with Section 11.4.3

s_h = Spacing of special lateral reinforcement, in. (mm)

s_u = Undrained shear strength (Section 20.4.3)

$\bar{s}_u$ = Average undrained shear strength in top 100 ft (30 m); see Sections 20.3.3 and 20.4.3, ASTM D2166, or ASTM D2850

s_{ui} = Undrained shear strength of any cohesive soil layer i [between 0 and 100 ft (0 and 30 m)]; see Section 20.4.3

T = Fundamental period of the building

$T_0 = 0.2S_{D1}/S_{DS}$

$\tilde{T}$ = Fundamental period as determined in Chapter 19

T_a = Approximate fundamental period of the building as determined in Section 12.8.2

T_{diaph} = Period of diaphragm for design of diaphragm using the alternative diaphragm design method of Section 12.10.4

T_L = Long-period transition period(s) shown in Figures 22-14 through 22-17

T_{lower} = Period of vibration at which 90% of the actual mass has been recovered in each of the two orthogonal directions of response (Section 12.9.2). The mathematical model used to compute T_{lower} shall not include accidental torsion and shall include P-delta effects.

T_p = Fundamental period of the component and its attachment (Section 13.3.3)

$T_S = S_{D1}/S_{DS}$

T_{upper} = Larger of the two orthogonal fundamental periods of vibration (Section 12.9.2). The mathematical model used to compute T_{upper} shall not include accidental torsion and shall include P-delta effects

TIR = Torsional Irregularity Ratio defined in Section 12.3.2.1.1

T_v = Vertical period of vibration

V = Total design lateral force or shear at the base

V_{EX} = Maximum absolute value of elastic base shear computed in the X direction among all three analyses performed in that direction (Section 12.9.2.5)

V_{EY} = Maximum absolute value of elastic base shear computed in the Y direction among all three analyses performed in that direction (Section 12.9.2.5)

V_{IX} = Inelastic base shear in the X direction (Section 12.9.2.5)

V_{IY} = Inelastic base shear in the Y direction (Section 12.9.2.5)

V_t = Design value of the seismic base shear as determined in Section 12.9.1.4.1

V_X = Equivalent lateral force (ELF) base shear in the X direction (Section 12.9.2.5)

V_x = Seismic design shear in story x as determined in Section 12.8.4

V_Y = Equivalent lateral force (ELF) base shear in the Y direction (Section 12.9.2.5)

$\tilde{V}$ = Reduced base shear accounting for the effects of soil structure interaction as determined in Section 19.3.1

$\tilde{V}_1$ = Portion of the reduced base shear, $\tilde{V}_1$, contributed by the fundamental mode, kip (kN) (Section 19.3)

ΔV = Reduction in V as determined in Section 19.3.1, kip (kN)

ΔV_1 = Reduction in V_1 as determined in Section 19.3.1, kip (kN)

v_s = Shear wave velocity, in ft/s (m/s), at small shear strains (greater than 10^{-3}% strain); see Section 19.2.1

$\bar{v}_s$ = Average shear wave velocity at small shear strains in top 100 ft (30 m) (Sections 20.3.3 and 20.4.1)

v_{si} = Shear wave velocity of any soil or rock layer i [between 0 and 100 ft (0 and 30 m)] (Section 20.4.1)

v_{so} = Average shear wave velocity, ft/s (m/s), for the soils beneath the foundation at small strain levels (Section 19.2.1.1)

W = Effective seismic weight of the building as defined in Section 12.7.2. For calculation of seismic-isolated building period, W is the total effective seismic weight of the building, in kip (kN), as defined in Sections 19.2 and 19.3

W = Effective seismic weight of the building, in kip (kN), as defined in Sections 19.2 and 19.3
W_c = Gravity load of a component of the building
W_P = Component operating weight, lb (N)
w_{px} = Weight tributary to the diaphragm at level x
w = Moisture content, in percent, per ASTM D2216
w_i, w_n, w_x = Portion of W that is located at or assigned to level i, n, or x, respectively
x = Level under consideration; $x = 1$ designates the first level above the base
z = Height in structure of point of attachment of component with respect to the base (Section 13.3.1)
z_s = Mode shape factor (Section 12.10.3.2.1)
β = Ratio of shear demand to shear capacity for the story between levels x and $x-1$
$\bar{\beta}$ = Fraction of critical damping for the coupled structure–foundation system, determined in Section 19.2.1
β_0 = Foundation damping factor as specified in Section 19.2.1.2
Γ_{m1}, Γ_{m2} = First and higher modal contribution factors, respectively (Section 12.10.3.2.1)
γ = Average unit weight of soil, lb/ft^3 (N/m^3)
Δ = Design story drift as determined in Section 12.8.6
Δ_{avg} = Average of the story drifts at the two opposing edges of the building, as defined in Section 12.3.2.1.1
Δ_{max} = Maximum story drift at the building's edge subjected to lateral forces, as defined in Section 12.3.2.1.1
$\Delta_{fallout}$ = Relative seismic displacement (drift) at which glass fallout from the curtain wall, storefront, or partition occurs
Δ_a = Allowable story drift as specified in Section 12.12.1
Δ_{ADVE} = Average drift of adjoining vertical elements of the seismic force-resisting system over the story below the diaphragm under consideration, under tributary lateral load equivalent to that used in the computation of δ_{MDD} in Figure 12.3-1, in. (mm)
δ_{DE} = Design earthquake displacement as determined in Section 12.8.6
δ_{di} = Displacement due to diaphragm deformation corresponding to the design earthquake including Section 12.10 diaphragm forces (Section 12.8.6)
δ_e = Elastic displacement computed under design earthquake forces (Section 12.8.6)
δ_{MCE} = Maximum Considered Earthquake Displacement as determined in Section 12.8.6
δ_{MDD} = Computed maximum in-plane deflection of the diaphragm under lateral load, in. (mm) (Figure 12.3-1)
δ_{max} = Maximum displacement at level x considering torsion (Section 12.8.4.3)
δ_M = Maximum inelastic response displacement considering torsion (Section 12.12.3)
δ_{MT} = Total separation distance between adjacent structures on the same property (Section 12.12.3)
δ_{avg} = Average of the displacements at the extreme points of the structure at level x (Section 12.8.4.3)
δ_{xm} = Modal deflection of level x at the center of the mass at and above level x as determined by Section 19.3.2
$\bar{\delta}_x, \bar{\delta}_{x1}$ = Deflection of level x at the center of the mass at and above level x in Equations (19.2-13) and (19.3-3), in. (mm)
θ = Stability coefficient for P-delta effects as determined in Section 12.8.7
ηx = Force scale factor in the X direction (Section 12.9.2.5)
ηy = Force scale factor in the Y direction (Section 12.9.2.5)
ρ = Redundancy factor based on the extent of structural redundancy present in a building, as defined in Section 12.3.4
λ = Time effect factor
Ω_0 = Overstrength factor as defined in Tables 12.2-1, 15.4.-1, and 15.4-2
$\Omega_{0\text{-diaph}}$ = Diaphragm overstrength factor (Section 12.10.4)
Ω_{0p} = The anchorage overstrength factor given in Tables 13.5-1 and 13.6-1

11.4 SEISMIC GROUND MOTION VALUES

11.4.1 Near-Fault Sites Sites satisfying either of the following conditions shall be classified as near fault:

- 9.5 mi (15 km) or less from the surface projection of a known active fault capable of producing M_w7 or larger events, or
- 6.25 mi (10 km) or less from the surface projection of a known active fault capable of producing events M_w6 or larger, but smaller than M_w7.

EXCEPTIONS:

1. Faults with estimated slip rate less than 0.04 in. (1 mm) per year shall not be used to determine whether a site is a near-fault site.
2. The surface projection used in the determination of near-fault site classification shall not include portions of the fault at depths of 6.25 mi (10 km) or greater.

11.4.2 Site Class The site shall be classified as Site Class A, B, BC, C, CD, D, DE, E, or F in accordance with Chapter 20.

11.4.2.1 Default Site Conditions Where the soil properties are not known in sufficient detail to determine the site class, risk-targeted maximum considered earthquake (MCE_R) spectral response accelerations shall be based on the most critical spectral response acceleration at each period of Site Class C, Site Class CD, and Site Class D, unless the Authority Having Jurisdiction determines, based on geotechnical data, that Site Class DE, E, or F soils are present at the site.

11.4.3 Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters Risk-targeted maximum considered earthquake (MCE_R) spectral response acceleration parameters S_S, S_1, S_{MS}, and S_{M1} shall be obtained from the USGS Seismic Design Geodatabase for the applicable site class.

EXCEPTION: Where a site-specific ground motion analysis is performed in accordance with Section 11.4.7, risk-targeted maximum considered earthquake (MCE_R) spectral response acceleration parameters S_{MS} and S_{M1} shall be determined in accordance with Section 21.4 and risk-targeted maximum considered earthquake (MCE_R) spectral response acceleration parameters S_S and S_1 shall be either (1) determined from the site-specific MCE_R response spectrum calculated in accordance with the requirements of Section 21.2.3 assuming Site Class BC site condition or (2) obtained from the USGS Seismic Design Geodatabase.

11.4.4 Design Spectral Acceleration Parameters Design earthquake spectral response acceleration parameters at short periods, S_{DS}, and at 1-s periods, S_{D1}, shall be determined from Equations (11.4-1) and (11.4-2), respectively. Where the simplified alternative design procedure of Section 12.14 is used, the value of S_{DS} shall be determined in accordance with Section 12.14.8.1, and the value for S_{D1} need not be determined.

$$S_{DS} = \frac{2}{3} S_{MS} \quad (11.4\text{-}1)$$

$$S_{D1} = \frac{2}{3} S_{M1} \quad (11.4\text{-}2)$$

where

S_{MS} = MCE_R, 5%-damped, spectral response acceleration parameter at short periods adjusted for site effects as determined in accordance with Section 11.4.3, and

S_{M1} = MCE_R, 5%-damped, spectral response acceleration parameter at a period of 1 s adjusted for site effects as determined in accordance with Section 11.4.3.

11.4.5 Design Response Spectrum Where a design response spectrum is required by this standard, the design response spectrum shall be determined in accordance with the requirements of Section 11.4.5.1.

EXCEPTIONS:

1. Where a site-specific ground motion analysis is performed in accordance with Section 11.4.7, the design response spectrum shall be determined in accordance with Section 21.3.
2. Where values of the multi-period 5%-damped MCE_R response spectrum are not available from the USGS Seismic Design Geodatabase, the design response spectrum shall be permitted to be determined in accordance with Section 11.4.5.2.

11.4.5.1 Multi-Period Design Response Spectrum The multi-period design response spectrum shall be developed as follows:

1. At discrete values of period, T, equal to 0.0 s, 0.01 s, 0.02 s, 0.03 s, 0.05 s, 0.075 s, 0.1 s, 0.15 s, 0.2 s, 0.25 s, 0.3 s, 0.4 s, 0.5 s, 0.75 s, 1.0 s, 1.5 s, 2.0 s, 3.0 s, 4.0 s, 5.0 s, 7.5 s, and 10 s, the 5%-damped design spectral response acceleration parameter, S_a, shall be taken as 2/3 of the multi-period 5%-damped MCE_R response spectrum from the USGS Seismic Design Geodatabase for the applicable site class.
2. At each response period, T, less than 10 s and not equal to one of the discrete values of period, T, listed in Item 1 above, S_a, shall be determined by linear interpolation between values of S_a, of Item 1 above.
3. At each response period, T, greater than 10 s, S_a, shall be taken as the value of S_a at the period of 10 s of Item 1 above, factored by $10/T$, where the value of T is less than or equal to that of the long-period transition period, T_L, and shall be taken as the value of S_a at the period of 10 s factored by $10T_L/T^2$, where the value of T is greater than that of the long-period transition period, T_L.

11.4.5.2 Two-Period Design Response Spectrum The two-period design response spectrum shall be developed as indicated in Figure 11.4-1 and as follows:

1. For periods less than T_0, the design spectral response acceleration parameter, S_a, shall be taken as given in Equation (11.4-3):

$$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right) \quad (11.4\text{-}3)$$

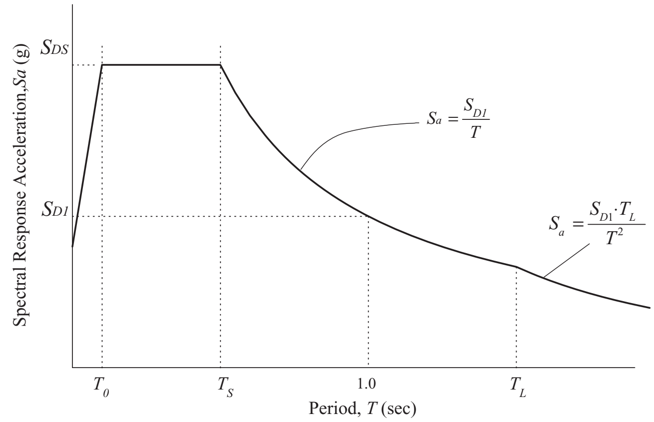

Figure 11.4-1. Two-period design response spectrum.

2. For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response acceleration parameter, S_a, shall be taken as equal to S_{DS}.
3. For periods greater than T_S and less than or equal to T_L, the design spectral response acceleration parameter, S_a, shall be taken as given in Equation (11.4-4):

$$S_a = \frac{S_{D1}}{T} \quad (11.4\text{-}4)$$

4. For periods greater than T_L, S_a shall be taken as given in Equation (11.4-5):

$$S_a = \frac{S_{D1} T_L}{T^2} \quad (11.4\text{-}5)$$

where

S_{DS} = Design spectral response acceleration parameter at short periods;

S_{D1} = Design spectral response acceleration parameter at a 1 s period;

T = Fundamental period of the structure, S;

$T_0 = 0.2(S_{D1}/S_{DS})$;

$T_S = S_{D1}/S_{DS}$; and

T_L = Long-period transition period(s) shown in Figures 22-14 through 22-17.

11.4.6 Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum Where an MCE_R response spectrum is required, it shall be determined by multiplying the design response spectrum by 1.5.

11.4.7 Site-Specific Ground Motion Procedures A site response analysis shall be performed in accordance with Section 21.1 for structures on Site Class F sites, unless exempted in accordance with Section 20.3.1.

It shall be permitted to perform a site response analysis in accordance with Section 21.1 and/or a ground motion hazard analysis in accordance with Section 21.2 to determine ground motions for any structure.

When the procedures of either Section 21.1 or 21.2 are used, the design response spectrum shall be determined in accordance with Section 21.3, the design acceleration parameters shall be determined in accordance with Section 21.4, and, if required, the MCE_G peak ground acceleration parameter PGA_M shall be determined in accordance with Section 21.5.

11.5 IMPORTANCE FACTOR AND RISK CATEGORY

11.5.1 Importance Factor An Importance Factor, I_e, shall be assigned to each structure in accordance with Table 1.5-2.

11.5.2 Protected Access for Risk Category IV Where operational access to a Risk Category IV structure is required through an adjacent structure, the adjacent structure shall conform to the requirements for Risk Category IV structures. Where operational access is less than 10 ft (3.048 m) from an interior lot line or another structure on the same lot, protection from potential falling debris from adjacent structures shall be provided by the owner of the Risk Category IV structure.

11.6 SEISMIC DESIGN CATEGORY

Structures shall be assigned a seismic design category in accordance with this section.

Risk Category I, II, or III structures located where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category E. Risk Category IV structures located where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category F. All other structures shall be assigned to a seismic design category based on their risk category and the design spectral response acceleration parameters, S_{DS} and S_{D1}, determined in accordance with Section 11.4.4. Each building and structure shall be assigned to the more severe seismic design category in accordance with Table 11.6-1 or 11.6-2, irrespective of the fundamental period of vibration of the structure, T. The provisions in Chapter 19 shall not be used to modify the spectral response acceleration parameters for determining seismic design category.

Where S_1 is less than 0.75, the seismic design category is permitted to be determined from Table 11.6-1 alone where 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 12.8.2.1 is less than $0.8T_s$, where T_s is determined in accordance with Section 11.4.5.
2. In each of two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.
3. Equation (12.8-2) is used to determine the seismic response coefficient, C_s.
4. The diaphragms are rigid in accordance with Section 12.3; or, for diaphragms that are not rigid, the horizontal distance between vertical elements of the seismic force-resisting system does not exceed 40 ft (12.192 m).

Table 11.6-1. Seismic Design Category Based on Short-Period Response Acceleration Parameter.

Value of S_{DS}	Risk Category I or II or III	Risk Category IV
$S_{DS} < 0.167$	A	A
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	D	D

Table 11.6-2. Seismic Design Category Based on 1 s Period Response Acceleration Parameter.

Value of S_{D1}	Risk Category I or II or III	Risk Category IV
$S_{D1} < 0.067$	A	A
$0.067 \le S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$.	D	

Where the alternate simplified design procedure of Section 12.14 is used, the seismic design category is permitted to be determined from Table 11.6-1 alone, using the value of S_{DS} determined in Section 12.14.8.1, except that where S_1 is greater than or equal to 0.75, the seismic design category shall be E.

11.7 DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORY A

Buildings and other structures assigned to Seismic Design Category A need only comply with the requirements of Section 1.4. Nonstructural components in SDC A are exempt from seismic design requirements. In addition, tanks assigned to Risk Category IV shall satisfy the freeboard requirement in Section 15.6.5.1.

11.8 GEOLOGIC HAZARDS AND GEOTECHNICAL INVESTIGATION

11.8.1 Site Limitation for Seismic Design Categories E and F A structure assigned to Seismic Design Category E or F shall not be located where a known potential exists for an active fault to cause rupture of the ground surface at the structure.

11.8.2 Geotechnical Investigation Report Requirements for Seismic Design Categories C through F A geotechnical investigation report shall be provided for a structure assigned to Seismic Design Category C, D, E, or F in accordance with this section. An investigation shall be conducted, and a report shall be submitted that includes an evaluation of the following potential geologic and seismic hazards:

(a) Slope instability,
(b) Liquefaction,
(c) Total and differential settlement, and
(d) Surface displacement caused by faulting or seismically induced lateral spreading or lateral flow.

The report shall contain recommendations for foundation designs or other measures to mitigate the effects of the previously mentioned hazards.

EXCEPTION: Where approved by the Authority Having Jurisdiction, a site-specific geotechnical report is not required where prior evaluations of nearby sites with similar soil conditions provide direction relative to the proposed construction.

11.8.3 Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F The geotechnical investigation report for a structure assigned to Seismic Design Category D, E, or F shall include all of the following, as applicable:

1. The determination of dynamic seismic lateral earth pressures on basement and retaining walls caused by design earthquake ground motions.
2. The potential for liquefaction, seismically-induced permanent ground displacement, and soil strength loss evaluated for site peak ground acceleration, earthquake magnitude, and source characteristics consistent with the MCE_G peak ground acceleration. Peak ground acceleration shall be determined based on either (1) a site-specific study taking into account soil amplification effects as specified in Section 11.4.7 or (2) the value of the MCE_G peak ground acceleration parameter PGA_M from the USGS Seismic Design Geodatabase for the applicable site class.
3. Assessment of potential consequences of liquefaction, seismically-induced permanent ground displacement, and soil strength loss, including, but not limited to, estimation of total and differential settlement, lateral soil movement, lateral soil loads on foundations, reduction in foundation soil-bearing capacity and lateral soil reaction, soil downdrag and reduction in axial and lateral soil reaction for pile foundations, increases in soil lateral pressures on retaining walls, and flotation of buried structures.
4. Discussion of mitigation measures such as, but not limited to, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements and forces, ground stabilization, or any combination of these measures and how they shall be considered in the design of the structure.

11.9 VERTICAL GROUND MOTIONS FOR SEISMIC DESIGN

11.9.1 General If the option to incorporate the effects of vertical seismic ground motions is exercised in lieu of the requirements of Section 12.4.2.2, the requirements of this section are permitted to be used in the determination of the vertical design earthquake ground motions. The requirements of Section 11.9.2 shall only apply to structures in Seismic Design Categories C, D, E, and F located in the conterminous United States at or west of −105 degrees longitude, Alaska, Hawaii, Puerto Rico, US Virgin Islands, Guam and Northern Mariana Islands, and American Samoa. For structures in Seismic Design Categories C, D, E, and F located in the conterminous United States east of −105 degrees longitude, the value of S_{aMv} shall be taken as two-thirds of the value of S_{aM}. The requirements of Section 11.9.3 shall apply to all structures in the Seismic Design Categories C, D, E, and F.

11.9.2 MCE_R **Vertical Response Spectrum** Where a vertical response spectrum is required by this standard and site-specific procedures are not used, the MCE_R vertical response spectral acceleration, S_{aMv}, shall be developed as follows:

1. For vertical periods (T_v) less than or equal to 0.025 s, S_{aMv} shall be determined in accordance with Equation (11.9-1) as follows:

$$S_{aMv} = 0.65C_v(S_{aM}/F_{md}) \quad (11.9\text{-}1)$$

2. For vertical periods greater than 0.025 s and less than or equal to 0.05 s, S_{aMv} shall be determined in accordance with Equation (11.9-2) as follows:

$$S_{aMv} = 16_{Cv}(S_{aM}/F_{md})(T_v - 0.025) + 0.65C_v(S_{aM}/F_{md}) \quad (11.9\text{-}2)$$

3. For vertical periods greater than 0.05 s and less than or equal to 0.1 s, S_{aMv} shall be determined in accordance with Equation (11.9-3) as follows:

$$S_{aMv} = 1.05C_v(S_{aM}/F_{md}) \quad (11.9\text{-}3)$$

4. For vertical periods greater than 0.1 s and less than or equal to 2.0 s, S_{aMv} shall be determined in accordance with Equation (11.9-4) as follows:

$$S_{aMv} = 1.05C_v\left(S_{aM}/F_{md}\right)\left(\frac{0.1}{T_v}\right)^{0.5} \quad (11.9\text{-}4)$$

The value of S_{aMv} shall not be less than 0.5 (S_{aM}/F_{md}).

5. For vertical periods greater than 2.0 s, S_{aMv} shall be determined in accordance with Equation (11.9-5) as follows:

$$S_{aMv} = 0.5(S_{aM}/F_{md}) \quad (11.9\text{-}5)$$

where

Cv = Is defined in terms of SS in Table 11.9-1;
S_{aM} = MCE_R spectral response acceleration parameter at the same period as S_{aMv};
F_{md} = Factor to convert the geometric mean spectral ordinate to a maximum direction spectral ordinate; and
T_v = Vertical period of vibration.

The maximum-component factor, F_{md}, shall be taken as follows:

$$T_v \le 0.2\,\text{s} : F_{md} = 1.2 \quad (11.9\text{-}6)$$

$$0.2 < T_v \le 1.0\,\text{s} : F_{md} = 1.2 + 0.0625(T_v - 0.2) \quad (11.9\text{-}7)$$

$$1 < T_v \le 1.0\,\text{s} : F_{md} = 1.25 + 0.05(T_v - 1.0)/9 \quad (11.9\text{-}8)$$

In lieu of using the above procedure, a site-specific study is permitted to be performed to obtain S_{aMv}, but the value so

Table 11.9-1. Values of Vertical Coefficient, C_v.

MCE_R Spectral Response Parameter at Short Periods*	Site Classes A, B	Site Class BC	Site Class C	Site Class CD	Site Classes D, DE, E, F
$S_{MS} \ge 2.0$	0.9	1.1	1.3	1.4	1.5
$S_{MS} = 12.0$	0.9	1.0	1.1	1.2	1.3
$S_{MS} = 0.6$	0.9	0.95	1.0	1.05	1.1
$S_{MS} = 0.3$	0.8	0.8	0.8	0.85	0.9
$S_{MS} \le 0.2$	0.7	0.7	0.7	0.7	0.7

*Use straight-line interpolation for intermediate values of S_{MS}.

determined shall not be less than 80% of the S_{aMv} value determined from Equations (11.9-1) through (11.9-5).

11.9.3 Design Vertical Response Spectrum The design vertical response spectral acceleration, S_{av}, shall be taken as two-thirds of the value of S_{aMv} determined in Sections 11.9.1 or 11.9.2.

11.10 CONSENSUS STANDARDS AND OTHER REFERENCED DOCUMENTS

See Chapter 23 for the list of consensus standards and other documents that shall be considered part of this standard to the extent referenced in this chapter.

CHAPTER C11
SEISMIC DESIGN CRITERIA

C11.1 GENERAL

Many of the technical changes made to the seismic provisions of the 2010 edition of this standard are primarily based on Part 1 of the 2009 edition of the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* (FEMA 2009), which was prepared by the Building Seismic Safety Council (BSSC) under sponsorship of the Federal Emergency Management Agency (FEMA) as part of its contribution to the National Earthquake Hazards Reduction Program (NEHRP). The National Institute of Standards and Technology (NIST) is the lead agency for NEHRP, the federal government's long-term program to reduce the risks to life and property posed by earthquakes in the United States. Since 1985, the NEHRP provisions have been updated every three to five years. The efforts by the BSSC to produce the NEHRP provisions were preceded by work performed by the Applied Technology Council (ATC) under sponsorship of the National Bureau of Standards (NBS)—now NIST—which originated after the 1971 San Fernando Valley earthquake. These early efforts demonstrated the design rules of that time for seismic resistance but had some serious shortcomings. Each subsequent major earthquake has taught new lessons. The NEHRP agencies [FEMA, NIST, the National Science Foundation (NSF), and the US Geological Survey (USGS)], ATC, BSSC, ASCE, and others have endeavored to work individually and collectively to improve each succeeding document to provide state-of-the-art earthquake engineering design and construction provisions and to ensure that the provisions have nationwide applicability.

Content of Commentary. This commentary is updated from the commentary to ASCE/SEI 7-16 and includes updates identified in Part 2, Commentary, of the 2020 *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* (FEMA 2020). For additional background on the earthquake provisions contained in Chapters 11 through 23 of this standard, the reader is referred to *Recommended Lateral Force Requirements and Commentary* (SEAOC 1999).

Nature of Earthquake "Loads." Earthquakes load structures indirectly through ground motion. As the ground shakes, a structure responds. The response vibration produces structural deformations with associated strains and stresses. The computation of dynamic response to earthquake ground shaking is complex. The design forces prescribed in this standard are intended only as approximations to generate internal forces suitable for proportioning the strength and stiffness of structural elements and for estimating the deformations (when multiplied by the deflection amplification factor, C_d) that would occur in the same structure in the event of the design-level earthquake ground motion (not MCE_R).

The basic methods of analysis in the standard use the common simplification of a response spectrum. A response spectrum for a specific earthquake ground motion provides the maximum value of response for elastic single-degree-of-freedom oscillators as a function of period without the need to reflect the total response history for every period of interest. The design response spectrum specified in Section 11.4 and used in the basic methods of analysis in Chapter 12 is a smoothed and normalized approximation for many different recorded ground motions.

The design limit state for resistance to an earthquake is unlike that for any other load within the scope of ASCE 7. The earthquake limit state is based on system performance, not member performance, and considerable energy dissipation through repeated cycles of inelastic straining is assumed. The reason is the large demand exerted by the earthquake and the associated high cost of providing enough strength to maintain linear elastic response in ordinary buildings. This unusual limit state means that several conveniences of elastic behavior, such as the principle of superposition, are not applicable and make it difficult to separate design provisions for loads from those for resistance. This difficulty is the reason Chapter 14 of the standard contains so many provisions that modify customary requirements for proportioning and detailing structural members and systems. It is also the reason for the construction quality assurance requirements.

Use of Allowable Stress Design Standards. The conventional design of almost all masonry structures and many wood and steel structures has been accomplished using allowable stress design (ASD). Although the fundamental basis for the earthquake loads in Chapters 11 through 23 is a strength limit state beyond the first yield of the structure, the provisions are written such that conventional ASD methods can be used by the design engineer. Conventional ASD methods may be used in one of two ways:

1. The earthquake load as defined in Chapters 11 through 23 may be used directly in allowable stress load combinations of Section 2.4, and the resulting stresses may be compared directly with conventional allowable stresses.
2. The earthquake load may be used in strength design load combinations, and resulting stresses may be compared with amplified allowable stresses (for those materials for which the design standard gives the amplified allowable stresses, e.g., masonry).

Executive Order (EO) 13717: *Establishing a Federal Earthquake Risk Management Standard*, issued February 2016, establishes a minimum level of seismic safety compliance for new buildings that will be constructed, financed, or regulated by the federal government. NIST Technical Note 1922 (NIST 2017) provides guidance on agency compliance with the EO.

C11.1.1 Purpose The purpose of Section 11.1.1 is to clarify that the detailing requirements and limitations prescribed in this section and referenced standards are still required even when the design load combinations involving the wind forces of Chapters 26 through 29 produce greater effects than the design load combinations involving the earthquake forces of Chapters 11 through 23. This detailing is required so that the structure resists, in a ductile manner, potential seismic loads in excess of the prescribed wind loads. A proper, continuous load path is an obvious design requirement, but experience has shown that it is often overlooked and that significant damage and collapse can result. The basis for this design requirement is twofold:

1. To ensure that the design has fully identified the seismic force-resisting system and its appropriate design level, and
2. To ensure that the design basis is fully identified for the purpose of future modifications or changes in the structure.

Detailed requirements for analyzing and designing this load path are given in the appropriate design and materials chapters.

C11.1.2 Scope Certain structures are exempt for the following reasons:

Exemption 1. Detached wood-frame dwellings not exceeding two stories above grade plane constructed in accordance with the prescriptive provisions of the International Residential Code (IRC) for light-frame wood construction, including all applicable IRC seismic provisions and limitations, are deemed capable of resisting the anticipated seismic forces. Detached one- and two-story wood-frame dwellings generally have performed well even in regions of higher seismicity. Therefore, within its scope, the IRC adequately provides the level of safety required for buildings. The structures that do not meet the prescriptive limitations of the IRC are required to be designed and constructed in accordance with the International Building Code (IBC) and the ASCE 7 provisions adopted therein.

Exemption 2. Agricultural storage structures generally are exempt from most code requirements because such structures are intended only for incidental human occupancy and represent an exceptionally low risk to human life.

Exemption 3. Bridges, transmission towers, nuclear reactors, and other structures with special configurations and uses are not covered. The regulations for buildings and buildinglike structures presented in this document do not adequately address the design and performance of such special structures.

ASCE 7 is not retroactive and usually applies to existing structures only when there is an addition, change of use, or alteration. Minimum acceptable seismic resistance of existing buildings is a policy issue normally set by the Authority Having Jurisdiction. ASCE 41 (2014) provides technical guidance but does not contain policy recommendations. A chapter in the International Building Code (IBC) applies to alteration, repair, addition, and change of occupancy of existing buildings, and the International Code Council maintains the International Existing Building Code (IEBC) and associated commentary.

C11.1.3 Applicability Industrial buildings may be classified as nonbuilding structures in certain situations for the purposes of determining seismic design coefficients and factors, system limitations, height limits, and associated detailing requirements. Many industrial building structures have geometries and/or framing systems that are different from the broader class of occupied structures addressed by Chapter 12, and the limited nature of the occupancy associated with these buildings reduces the hazard associated with their performance in earthquakes. Therefore, when the occupancy is limited primarily to maintenance and monitoring operations, these structures may be designed in accordance with the provisions of Section 15.5 for nonbuilding structures similar to buildings. Examples of such structures include, but are not limited to, boiler buildings, aircraft hangars, steel mills, aluminum smelting facilities, and other automated manufacturing facilities, whereby the occupancy restrictions for such facilities should be uniquely reviewed in each case. These structures may be clad or open structures.

C11.1.4 Alternate Materials and Methods of Construction It is not possible for a design standard to provide criteria for the use of all possible materials and their combinations and methods of construction, either existing or anticipated. This section serves to emphasize that the evaluation and approval of alternate materials and methods require a recognized and accepted approval system. The requirements for materials and methods of construction contained within the document represent the judgment of the best use of the materials and methods based on well-established expertise and historical seismic performance. It is important that any replacement or substitute be evaluated with an understanding of all the ramifications of performance, strength, and durability implied by the standard.

Until needed standards and agencies are created, authorities that have jurisdiction need to operate on the basis of the best evidence available to substantiate any application for alternates. If accepted standards are lacking, applications for alternative materials or methods should be supported by test data obtained from test data requirements in Section 1.3.1. The tests should simulate expected load and deformation conditions to which the system, component, or assembly may be subjected during the service life of the structure. These conditions, when applicable, should include several cycles of full reversals of loads and deformations in the inelastic range.

C11.1.5 Quality Assurance Quality assurance (QA) requirements are essential for satisfactory performance of structures in earthquakes. QA requirements are usually incorporated in building codes as special inspections and tests or as structural observation, and they are enforced through the Authorities Having Jurisdiction. Many building code requirements parallel or reference the requirements found in standards adopted by ASCE 7. Where special inspections and testing, or structural observations are not specifically required by the building code, a level of QA is usually provided by inspectors employed by the Authority Having Jurisdiction.

Where building codes are not in force or where code requirements do not apply to or are inadequate for a unique structure or system, the registered design professional for the structure or system should develop a QA program to verify that the structure or system is constructed as designed. A QA program could be modeled on similar provisions in the building code or applicable standards.

The quality assurance plan is used to describe the QA program to the owner, the Authority Having Jurisdiction, and to all other participants in the QA program. As such, the QA plan should include definitions of the roles and responsibilities of the participants. It is anticipated that in most cases the owner of the project would be responsible for implementing the QA plan.

C11.2 DEFINITIONS

ATTACHMENTS, COMPONENTS, AND SUPPORTS: The distinction among attachments, components, and supports is

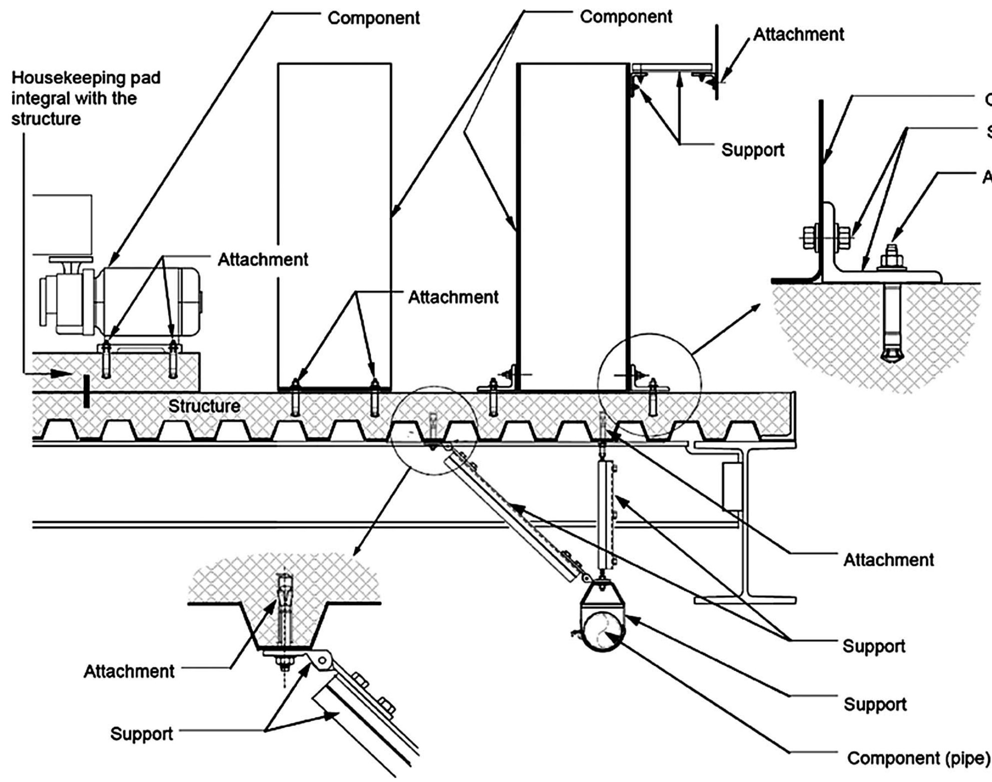

Figure C11.2-1. Examples of attachments, components, and supports.

necessary to the understanding of the requirements for nonstructural components and nonbuilding structures. Common cases associated with nonstructural elements are illustrated in Figure C11.2-1. The definitions of attachments, components, and supports are generally applicable to components with a defined envelope in the as-manufactured condition and for which additional supports and attachments are required to provide support in the as-built condition. This distinction may not always be clear, particularly when the component is equipped with prefabricated supports; therefore, judgment must be used in the assignment of forces to specific elements in accordance with the provisions of Chapter 13.

BASE: The following factors affect the location of the seismic base:

- Location of the grade relative to floor levels,
- Soil conditions adjacent to the building,
- Openings in the basement walls,
- Location and stiffness of vertical elements of the seismic force-resisting system,
- Location and extent of seismic separations,
- Depth of basement,
- Manner in which basement walls are supported,
- Proximity to adjacent buildings, and
- Slope of grade.

For typical buildings on level sites with competent soils, the base is generally close to the grade plane. For a building without a basement, the base is generally established near the ground-level slab elevation, as shown in Figure C11.2-2. Where the vertical elements of the seismic force-resisting system are supported on interior footings or pile caps, the base is the top of these elements. Where the vertical elements of the seismic force-resisting system are supported on top of perimeter foundation walls, the base is typically established at the top of the foundation walls. Often vertical elements are supported at various elevations on the top of footings, pile caps, and perimeter foundation walls. Where this occurs, the base is generally established as the lowest elevation of the tops of elements supporting the vertical elements of the seismic force-resisting system.

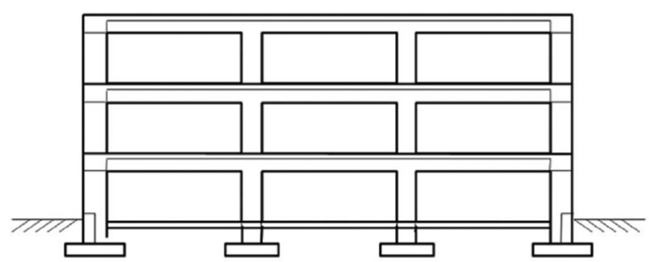

Figure C11.2-2. Base for a level site.

For a building with a basement located on a level site, it is often appropriate to locate the base at the floor closest to grade, as shown in Figure C11.2-3. If the base is to be established at the level located closest to grade, the soil profile over the depth of the basement should not be liquefiable in the MCE_G ground motion. The soil profile over the depth of the basement also should not include quick and highly sensitive clays or weakly cemented soils prone to collapse in the MCE_G ground motion. Where liquefiable soils or soils susceptible to failure or collapse in an MCE_G ground motion are located within the depth of the basement, the base may need to be located below these soils rather than close to grade. Stiff soils are required over the depth of the basement because seismic forces are transmitted to and from the building at this level and over the height of the basement

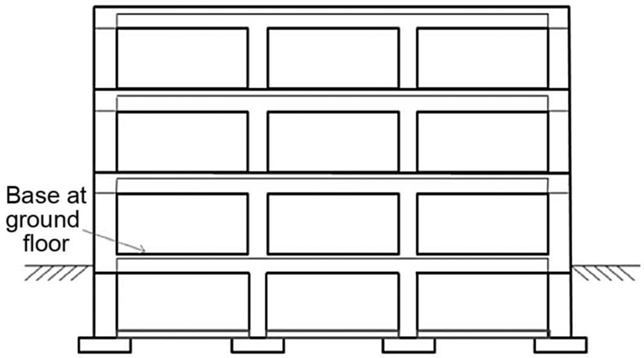

Figure C11.2-3. Base at ground floor level.

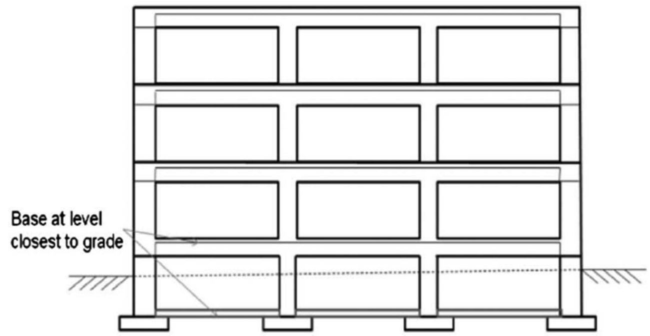

Figure C11.2-4. Base at level closest to grade elevation.

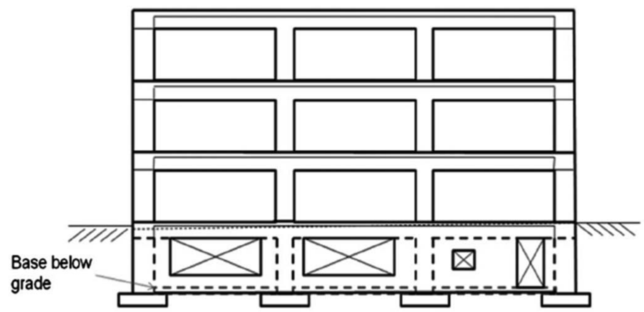

Figure C11.2-5. Base below substantial openings in basement wall.

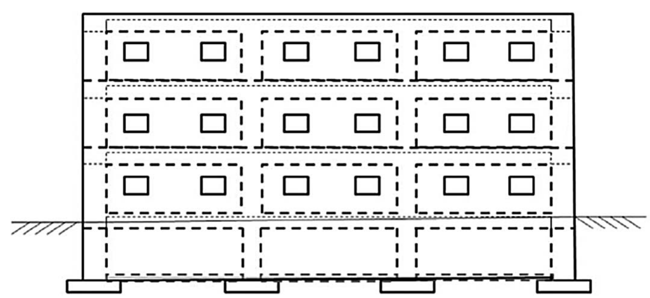

Figure C11.2-6. Base at foundation level where there are full-length exterior shear walls.

walls. The engineer of record is responsible for establishing whether the soils are stiff enough to transmit seismic forces near grade. For tall or heavy buildings or where soft soils are present within the depth of the basement, the soils may compress laterally too much during an earthquake to transmit seismic forces near grade. For these cases, the base should be located at a level below grade.

In some cases, the base may be at a floor level above grade. For the base to be located at a floor level above grade, stiff foundation walls on all sides of the building should extend to the underside of the elevated level considered the base. Locating the base above grade is based on the principles for the two-stage equivalent lateral force procedure for a flexible upper portion of a building with one-tenth the stiffness of the lower portion of the building, as permitted in Section 12.2.3.2. For a floor level above grade to be considered the base, it generally should not be above grade more than one-half the height of the basement story, as shown in Figure C11.2-4. Figure C11.2-4 illustrates the concept of the base level located at the top of a floor level above grade, which also includes light-frame floor systems that rest on top of stiff basement walls or stiff crawl space stem walls of concrete or masonry construction.

A condition where the basement walls that extend above grade on a level site may not provide adequate stiffness is where the basement walls have many openings for items such as light wells, areaways, windows, and doors, as shown in Figure C11.2-5. Where the basement wall stiffness is inadequate, the base should be taken as the level close to but below grade. If all of the vertical elements of the seismic force-resisting system are located on top of basement walls and there are many openings in the basement walls, it may be appropriate to establish the base at the bottom of the openings. Another condition where the basement walls may not be stiff enough is where the vertical elements of the seismic force-resisting system are long concrete shear walls extending over the full height and length of the building, as shown in Figure C11.2-6. For this case, the appropriate location for the base is the foundation level of the basement walls.

Where the base is established below grade, the weight of the portion of the story above the base that is partially above and below grade must be included as part of the effective seismic weight. If the equivalent lateral force procedure is used, this procedure can result in disproportionately high forces in upper levels because of a large mass at this lowest level above the base. The magnitude of these forces can often be mitigated by using the two-stage equivalent lateral force procedure where it is allowed or by using dynamic analysis to determine force distribution over the height of the building. If dynamic analysis is used, it may be necessary to include multiple modes to capture the required mass participation, unless soil springs are incorporated into the model. Incorporation of soil springs into the model generally reduces seismic forces in the upper levels. With one or more stiff stories below more flexible stories, the dynamic behavior of the structure may result in the portion of the base shear from the first mode being less than the portion of base shear from higher modes.

Other conditions may also necessitate establishing the base below grade for a building with a basement that is located on a level site. Such conditions include those where seismic separations extend through all floors, including those located close to and below grade; those where the floor diaphragms close to and below grade are not tied to the foundation wall; those where the floor diaphragms, including the diaphragm for the floor close to grade, are flexible; and those where other buildings are located nearby.

For a building with seismic separations extending through the height of the building including levels close to and below grade, the separate structures are not supported by the soil against a basement wall on all sides in all directions. If there is only one joint through the building, assigning the base to the level close to grade may still be appropriate if the soils over the depth of the basement walls are stiff and the diaphragm is rigid. Stiff soils are required so that the seismic forces can be transferred between the soils and basement walls in both bearing and side friction. If the soils are not stiff, adequate side friction may not develop for movement in the direction perpendicular to the joint.

For large footprint buildings, seismic separation joints may extend through the building in two directions and there may be multiple parallel joints in a given direction. For individual structures within these buildings, substantial differences in the location of the center of rigidity for the levels below grade relative to levels above grade can lead to torsional response. For such buildings, the base should usually be at the foundation elements below the basement or the highest basement slab level where the separations are no longer provided.

Where floor levels are not tied to foundation walls, the base may need to be located well below grade at the foundation level. An example is a building with tieback walls and posttensioned floor slabs. For such a structure, the slabs may not be tied to the wall to allow relative movement between them. In other cases, a soft joint may be provided. If shear forces cannot be transferred between the wall and a ground level or basement floor, the location of the base depends on whether forces can be transferred through bearings between the floor diaphragm and basement wall and between the basement wall and the surrounding soils. Floor diaphragms bearing against the basement walls must resist the compressive stress from earthquake forces without buckling. If a seismic or expansion joint is provided in one of these buildings, the base almost certainly needs to be located at the foundation level or a level below grade where the joint no longer exists.

If the diaphragm at grade is flexible and does not have substantial compressive strength, the base of the building may need to be located below grade. This condition is more common with existing buildings. Newer buildings with flexible diaphragms should be designed for compression to avoid the damage that can otherwise occur.

Proximity to other structures can also affect where the base should be located. If other buildings with basements are located adjacent to one or more sides of a building, it may be appropriate to locate the base at the bottom of the basement. The closer the adjacent building is to the building, the more likely it is that the base should be below grade.

For sites with sloping grade, many of the same considerations for a level site are applicable. For example, on steeply sloped sites, the earth may be retained by a tieback wall so that the building does not have to resist the lateral soil pressures. For such a case, the building is independent of the wall, so the base should be located at a level close to the elevation of grade on the side of the building where it is lowest, as shown in Figure C11.2-7. Where the building's vertical elements of the seismic force-resisting system also resist lateral soil pressures, as shown in Figure C11.2-8, the base should also be located at a level close to the elevation of grade on the side of the building where grade is low. For these buildings, the seismic force-resisting system below highest grade is often much stiffer than the system used above it, as shown in Figure C11.2-9, and the seismic weights for levels close to and below highest grade are greater than for levels above highest grade. Use of a two-stage equivalent lateral force procedure can be useful for these buildings.

Where the site is moderately sloped such that it does not vary in height by more than a story, stiff walls often extend to the underside of the level close to the elevation of high grade, and the seismic force-resisting system above grade is much more flexible

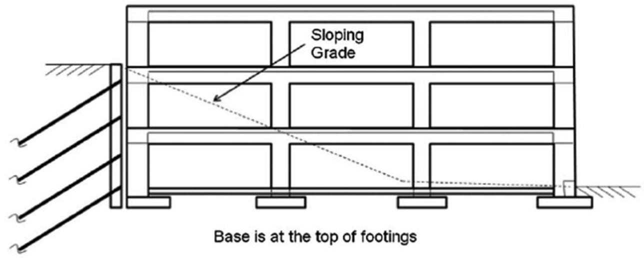

Figure C11.2-7. Building with tie-back or cantilevered retaining wall that is separate from the building.

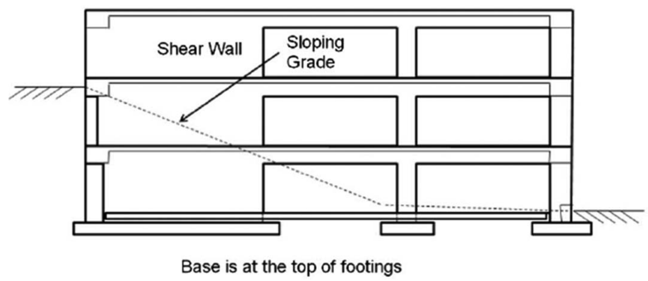

Figure C11.2-8. Building with vertical elements of the seismic force-resisting system supporting lateral earth pressures.

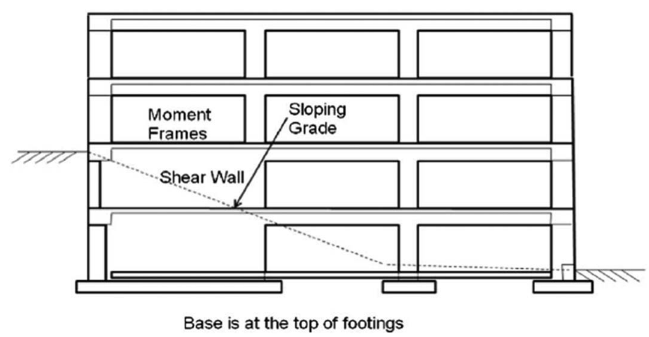

Figure C11.2-9. Building with vertical elements of the seismic force-resisting system supporting lateral earth pressures.

above grade than it is below grade. If the stiff walls extend to the underside of the level close to high grade on all sides of the building, locating the base at the level closest to high grade may be appropriate. If the stiff lower walls do not extend to the underside of the level located closest to high grade on all sides of the building, the base should be assigned to the level closest to low grade. If there is doubt as to where to locate the base, it should conservatively be taken at the lower elevation.

DISTRIBUTION SYSTEM: For the purposes of determining the anchorage of components in Chapter 13, a distribution system is characterized as a series of individual in-line mechanical or electrical components that have been physically attached together to function as an interconnected system. In general, the individual in-line components of a distribution system are comparable to those of the pipe, duct, or electrical raceway, so that the overall seismic behavior of the system is relatively uniform along its length. For example, a damper in a duct or a valve in a pipe is sufficiently similar to the weight of the duct or pipe itself, as opposed to a large fan or large heat exchanger. If a component is large enough to require support that is independent of the piping, duct, or conduit to which it is attached, it should likely be treated as a discrete component with regard to both exemptions and general design requirements. Representative distribution systems are listed in Table 13.6-1.

FLEXURE-CONTROLLED DIAPHRAGM: An example of a flexure-controlled diaphragm is a cast-in-place concrete diaphragm, where the flexural yielding mechanism would typically be yielding of the chord tension reinforcement.

SHEAR-CONTROLLED DIAPHRAGM: Shear-controlled diaphragms fall into two main categories. The first category is diaphragms that cannot develop a flexural mechanism because of aspect ratio, chord member strength, or other constraints. The second category is diaphragms that are intended to yield in shear rather than in flexure. Wood-sheathed diaphragms, for example, typically fall in the second category.

STORY ABOVE GRADE PLANE: Figure C11.2-10 illustrates this definition.

TRANSFER FORCES: Transfer forces are diaphragm forces that are not caused by the acceleration of the diaphragm inertial mass. Transfer forces occur because of discontinuities in the vertical elements of the seismic force-resisting system or because of changes in stiffness in these vertical elements from one story to the next, even if there is no discontinuity. In addition, buildings that combine frames and shear walls, which would have different deflected shapes under the same loading, also develop transfer forces in the diaphragms that constrain the frames and shear walls to deform together; this development is especially significant in dual systems.

C11.3 SYMBOLS

The provisions for precast concrete diaphragm design are intended to ensure that yielding, when it occurs, is ductile. Since yielding in shear is generally brittle at precast concrete connections, an additional overstrength factor, Ω_v, has been introduced; the required shear strength for a precast diaphragm is required to be amplified by this factor. This term is added to the symbols.

δ_{MDD}= This symbol refers to in-plane diaphragm deflection and is therefore designated with a lower-case delta. Note that the definition for δ_{MDD} refers to "lateral load" without any qualification, and the definition for Δ_{ADVE} refers to "tributary lateral load equivalent to that used in the computation of δ_{MDD}." This equivalency is an important concept that was part of the 1997 Uniform Building Code (UBC) (ICBO 1997) definition for a flexible diaphragm.

Ω_v= The provisions for precast concrete diaphragm design are intended to ensure that yielding, when it occurs, is ductile. Since yielding in shear is generally brittle at precast concrete connections, an additional overstrength factor, Ω_v, has been introduced; the required shear strength for a precast diaphragm is required to be amplified by this factor. This term is added to the symbols.

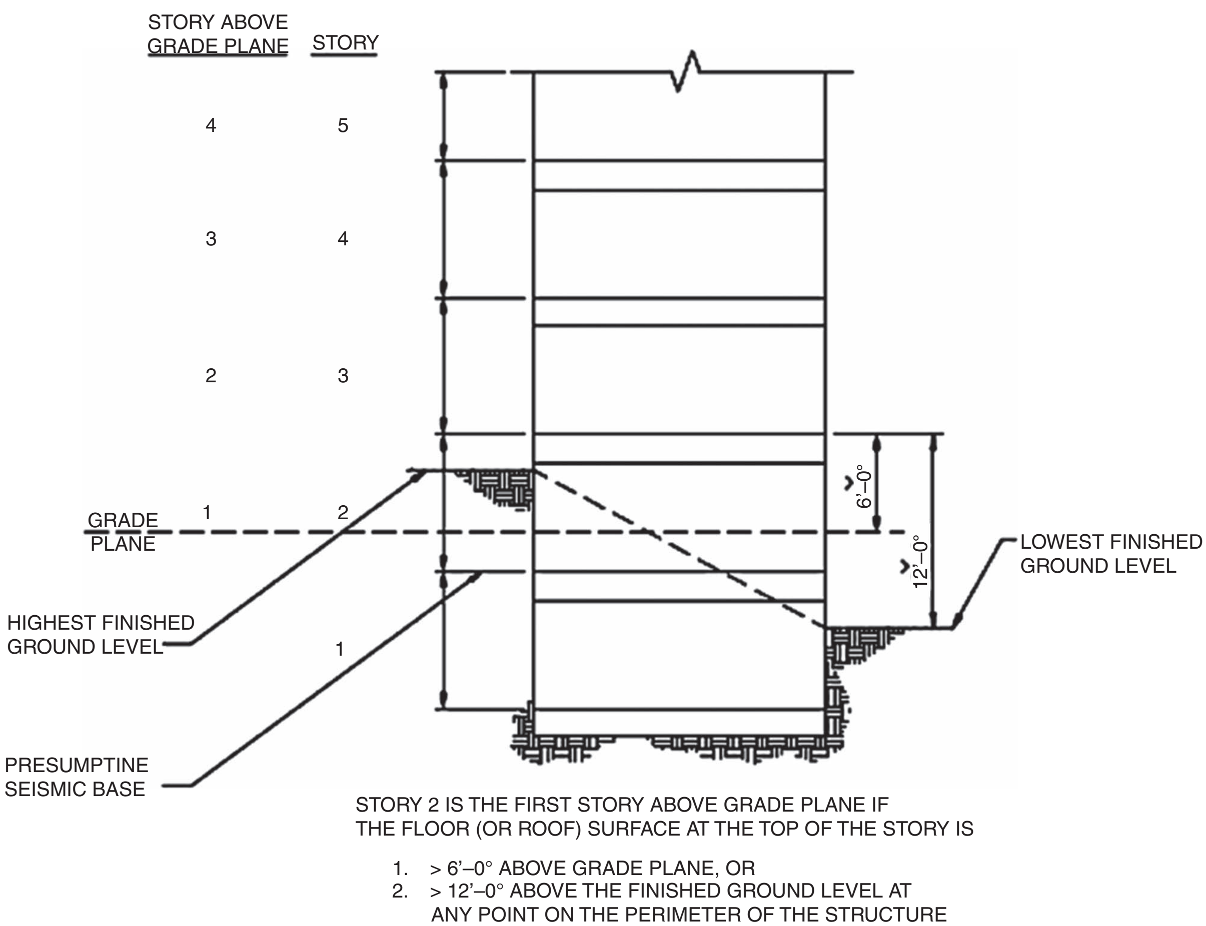

Figure C11.2-10. Illustration of definition of story above grade plane.

Note: To convert feet to millimeters, multiply by 304.8.

C11.4 SEISMIC GROUND MOTION VALUES

The theoretical basis for the mapped values of the MCE_R ground motions in the 2020 *NEHRP Recommended Provisions* (ASCE 7-22) is identical to that in the 2015 *NEHRP Recommended Provisions* (the basis of ASCE 7-16) and in the 2009 *NEHRP Recommended Provisions* (the basis of ASCE 7-10). ASCE 7-22 MCE_R ground motions (like those of ASCE 7-16 and ASCE 7-10) are significantly different from mapped values of maximum considered earthquake (MCE) ground motions in earlier editions of ASCE 7. These differences include use of (1) probabilistic ground motions that are based on uniform risk, rather than uniform hazard; (2) deterministic ground motions that are based on the 84th percentile (approximately 1.8 times median), rather than 1.5 times median response spectral acceleration for sites near active faults; and (3) ground motion intensity that is based on maximum rather than average (geometric mean) response spectral acceleration in the horizontal plane. These differences are explained in detail in the commentary of the 2009 *NEHRP Recommended Provisions*. Except for determining the MCE_G Peak Ground Acceleration (PGA) values in Chapters 11 and 21, the mapped values are given as MCE_R spectral values.

While the theoretical basis for MCE_R ground motions has not changed from ASCE 7-16 (and prior editions), ASCE 7-22 now uses a multi-period response spectra (MPRS) to improve the accuracy of the frequency content of earthquake design ground motions and to enhance the reliability of the seismic design parameters derived from these ground motions. These improvements make better use of the available earth science, which has, in general, sufficiently advanced to accurately define spectral response for different site conditions over a broad range of periods and eliminate the need for site-specific hazard analysis required by ASCE 7-16 for certain (soft soil) sites, as discussed following.

During the closing months of the 2015 cycle of the Provisions Update Committee (PUC) of the Building Seismic Safety Council, a study was undertaken of the compatibility of current Site Class coefficients F_a and F_v, with the ground motion relations used by USGS to produce the design maps (Kircher & Associates 2015). In the course of this study, it was discovered that the standard three-domain spectral shape defined by the short-period response spectral acceleration parameter, S_{DS}, the 1-second response spectral acceleration parameter, S_{D1}, and the long-period transition period, T_L, is not appropriate for soft soil sites (Site Class D or softer), in particular, where ground motion hazard is dominated by large magnitude events. Specifically, on such sites, the standard spectral shape substantially understates spectral demand for moderately long period structures. The PUC initiated a proposal to move to specification of spectral acceleration values over a range of periods, abandoning the present three-domain format, as this would provide better definition of likely ground motion demands. However, this proposal was ultimately not adopted due to both the complexity of implementing such a revision in the design procedure and time constraints. Instead, the PUC adopted a proposal prohibiting the general use of the three-parameter spectrum, and instead requiring site-specific hazard determination for longer period structures on soft soil sites.

Subsequently, Project 17 (NIBS 2019) was charged with re-evaluating the use of multiperiod response spectra as a replacement or supplement to the present three-domain spectral definition and to consider how the basic design procedures embedded in ASCE 7 should be modified for compatibility with the multi-period spectra. As a result, Project 17 developed (and unanimously approved) a comprehensive multi-period response spectra (MPRS) proposal that was subsequently adopted (with changes) in the 2020 *NEHRP Recommended Provisions* (FEMA P-2082, 2020a), which form the basis for related changes to ASCE 7-22. "Procedures for Developing Multi-Period Response Spectra of Non-Conterminous United States Sites," FEMA P-2078 (2020b), complements the changes to ASCE 7-22 by providing methods for developing MPRS of those regions (i.e., Alaska, Hawaii, Puerto Rico, Guam, and American Samoa) for which ground motion relations have not yet been used by the USGS to fully define all periods and site classes of interest.

C11.4.1 Near-Fault Sites In addition to very large accelerations, ground motions on sites located close to the zone of fault rupture of large-magnitude earthquakes can exhibit impulsive characteristics as well as unique directionality not typically recorded at sites located more distant from the zone of rupture. In past earthquakes, these characteristics have been observed to be particularly destructive. Accordingly, this standard establishes more restrictive design criteria for structures located on sites where such ground motions may occur. The standard also requires direct consideration of these unique characteristics in selection and scaling of ground motions used in nonlinear response history analysis and for the design of structures using seismic isolation or energy-dissipation devices when located on such sites.

The distance from the zone of fault rupture at which these effects can be experienced is dependent on a number of factors, including the rupture type, depth of fault, magnitude, and direction of fault rupture. Therefore, a precise definition of what constitutes a near-fault site is difficult to establish on a general basis. This standard uses two categorizations of near-fault conditions, both based on the distance of a site from a known active fault, capable of producing earthquakes of a defined magnitude or greater, and having average annual slip rates of nonnegligible amounts. These definitions were first introduced in the 1997 Uniform Building Code (ICBO 1997). Figure C11.4-1 illustrates the means of determining the distance of a site from a fault, where the fault plane dips at an angle relative to the ground surface.

C11.4.2 Site Class Site class is defined in terms of average shear wave velocity ($\overline{v}_s$) in accordance with Table 20.2-1 of Chapter 20. Table 0.2-1 includes the six site classes of ASCE 7-16 (A, B, C, D, E, and F) plus three new site classes (BC, CD, and DE) that provide better resolution of site shear wave velocity and associated site amplification for common site conditions. The new site classes allow for more accurate derivation of the amplitude and frequency content of earthquake ground motions, and their variation with shaking intensity (nonlinear effects). The additional site classes are of particular importance to the characterization of long period ground motions for softer sites.

C11.4.2.1 Default Site Conditions The "default" site condition is defined as the most critical response spectral acceleration at each period of typical soil site conditions (Site Classes C, CD, and D) to provide a conservative basis for design where site class is not known (e.g., due to insufficient geological investigation). Enveloping of Site Classes C, CD, and D is consistent with ASCE 7-16, which effectively requires the more critical of Site Class C and D to be used for design where site class is not known. Use of the default site conditions for design presumes that the site does not have soft soil site conditions (i.e., Site Class DE, E, or F site conditions) and should not be used for design where such site conditions are known to exist.

C11.4.3 Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters "Mapped" values of seismic parameters S_S, S_1, S_{MS}, and

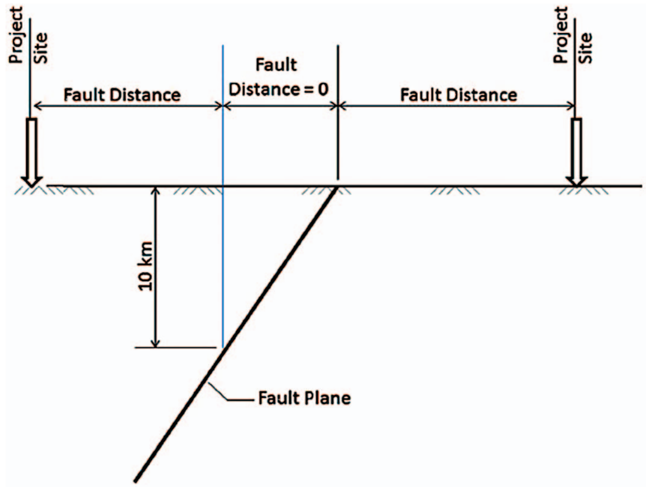

Figure C11.4-1. Fault distance for various project site locations.
Note: To convert to km to mi, multiply by 0.62.

S_{M1} are archived in the USGS Seismic Design Geodatabase at gridded locations across United States regions of interest and provided online by the USGS Seismic Design Web Service, for user-specified site location (latitude and longitude) and site class. The USGS web service spatially interpolates between the gridded values of these parameters based on site location (latitude and longitude). Chapter 22 provides print copies of seismic parameters S_{MS} and S_{M1} for default site conditions. Seismic parameters S_{MS} and S_{M1} (and S_{DS} and S_{D1}) incorporate site effects, eliminating the need for the tables of site factors F_a and F_v of ASCE 7-16.

C11.4.4 Design Spectral Acceleration Parameters Design in ASCE 7 (e.g., Chapter 12) is performed for earthquake demands that are 2/3 of MCE$_R$ ground motions. As set forth in Section 11.4.4, two additional parameters S_{DS} and S_{D1} are used to define design spectral accelerations.

Values of the seismic parameters S_{DS} and S_{D1} (2/3S_{MS} and 2/3S_{M1}) are provided online by the USGS Seismic Design Web Service for user-specified site location (latitude and longitude) and site class. Values of the seismic parameters S_{DS} and S_{D1} provided by the USGS are based on the multiperiod design response spectrum (Section 11.4.5.1) of the site of interest and the requirements of Section 21.4 (for determining values of S_{DS} and S_{D1} from a site-specific design response spectrum).

C11.4.5 Design Response Spectrum The design response spectrum, (and MCE$_R$ response spectrum of Section 11.4.6) are defined by either (1) a multi-period response spectrum (Section 11.4.5.1) or (2) a two-period response spectrum (Section 11.4.5.2), unless the design is based on site-specific ground motions (Section 21.3). The multi-period design response spectrum provides a more accurate representation of the frequency content of design ground motions and is the preferred characterization of spectral response. The shape of the two-period design response spectrum is the same as that of ASCE 7-16, which relies on a simpler characterization of the frequency content of design ground motions (Figure 11.4-1) based on the values of seismic parameters S_{DS} and S_{D1} (and T_L). The two-period design response spectrum is retained in ASCE 7-22 as an alternative characterization of ground motions for design where multi-period spectra are not available (e.g., from the USGS).

C11.4.5.1 Multi-Period Design Response Spectrum Sets of multi-period MCE$_R$ response spectra (5% damping) at 22 response periods (0.0 s, 0.01 s, 0.02 s, 0.03 s, 0.05 s, 0.075 s, 0.1 s, 0.15 s, 0.2 s, 0.25 s, 0.3 s, 0.4 s, 0.5 s, 0.75 s, 1.0 s, 1.5 s, 2.0 s, 3.0 s, 4.0 s, 5.0 s, 7.5 s, and 10 s) are archived in the USGS Seismic Design Geodatabase at gridded locations across United States regions of interest. A USGS Seismic Design Web Service, for the site location and site class of interest, spatially interpolates between the gridded sets of multi-period MCE$_R$ response spectra based on site location (latitude and longitude). Multi-period design response spectrum is constructed from two-thirds of these values by linear interpolation for response periods less than 10 s and by extrapolation for response periods greater than 10 s.

At response periods beyond 10 s, values of the multi-period design response spectrum are assumed to decrease from the value of the design response spectrum value at 10 s as the inverse of the period, T, where T is less than T_L and/or as inverse of the square of the period, T^2, where T is greater than T_L, essentially following the same approach as that used to construct the two-period spectrum (Section 11.4.5.2) at very long-periods.

C11.4.5.2 Two-Period Design Response Spectrum The two-period design response spectrum (Figure 11.4-1) consists of several segments. The constant-acceleration segment covers the period band from T_0 to T_s; response accelerations in this band are constant and equal to S_{DS}. The constant-velocity segment covers the period band from T_s to T_L, and the response accelerations in this band are proportional to $1/T$ with the response acceleration at a 1-s period equal to S_{D1}. The long-period portion of the design response spectrum is defined on the basis of the parameter T_L, the period that marks the transition from the constant-velocity segment to the constant-displacement segment of the design response spectrum. Response accelerations in the constant-displacement segment, where $T \geq T_L$, are proportional to $1/T^2$. Values of T_L are provided on maps in Figures 22-14 through 22-17.

The T_L maps were prepared following a two-step procedure. First, a correlation between earthquake magnitude and T_L was established. Then, the modal magnitude from deaggregation of the ground-motion seismic hazard at a 2 s period (a 1 s period for Hawaii) was mapped. Details of the procedure and the rationale for it are found in Crouse et al. (2006).

C11.4.7 Site-Specific Ground Motion Procedures Site-specific ground motions are permitted for design of any structure and are required for design of certain structures and certain site soil conditions. The objective of a site-specific ground motion analysis is to determine ground motions for local seismic and site conditions with higher confidence than is possible using the general procedure of Section 11.4.

As noted earlier, the site-specific procedures of Chapter 21 are the same as those used by the US Geological Survey (USGS) to develop the mapped values of MCE_R ground motions. Unless significant differences in local seismic and site conditions are determined by a site-specific analysis of earthquake hazards, site-specific ground motions would not be expected to differ significantly from those of the mapped values of MCE_R ground motions prepared by the USGS.

C11.5 IMPORTANCE FACTOR AND RISK CATEGORY

Large earthquakes are rare events that include severe ground motions. Such events are expected to result in damage to structures even if they were designed and built in accordance with the minimum requirements of the standard. The consequence of structural damage or failure is not the same for the various types of structures located within a given community. Serious damage to certain classes of structures, such as critical facilities (e.g., hospitals), disproportionately affects a community. The fundamental purpose of this section and of subsequent requirements that depend on this section is to improve the ability of a community to recover from a damaging earthquake by tailoring the seismic protection requirements to the relative importance of a structure. That purpose is achieved by requiring improved performance for structures that

1. Are necessary to response and recovery efforts immediately after an earthquake,
2. Present the potential for catastrophic loss in the event of an earthquake, or
3. House a large number of occupants or occupants less able to care for themselves than the average.

The first basis for seismic design in the standard is that structures should have a suitably low likelihood of collapse in the rare events defined as the maximum considered earthquake (MCE) ground motion. A second basis is that life-threatening damage, primarily from failure of nonstructural components in and on structures, is unlikely in a design earthquake ground motion (defined as two-thirds of the MCE). Given the occurrence of ground motion equivalent to the MCE, a population of structures built to meet these design objectives probably still experiences substantial damage in many structures, rendering these structures unfit for occupancy or use. Experience in past earthquakes around the world has demonstrated that there is an immediate need to treat injured people, to extinguish fires and prevent conflagration, to rescue people from severely damaged or collapsed structures, and to provide shelter and sustenance to a population deprived of its normal means. These needs are best met when structures essential to response and recovery activities remain functional.

This standard addresses these objectives by requiring that each structure be assigned to one of the four risk categories presented in Chapter 1 and by assigning an Importance Factor, I_e, to the structure based on that risk category. (The two lowest categories, I and II, are combined for all purposes within the seismic provisions.) The risk category is then used as one of two components in determining the seismic design category (SDC, see Section C11.6) and is a primary factor in setting drift limits for building structures under the design earthquake ground motion (see Section C12.12).

Figure C11.5-1 shows the combined intent of these requirements for design. The vertical scale is the likelihood of the ground motion; the MCE is the rarest considered. The horizontal scale is the level of performance intended for the structure and attached nonstructural components, which range from Collapse Prevention to Operational. The basic objective of Collapse Prevention at the MCE for ordinary structures (Risk Category II) is shown at the lower right by the solid triangle; protection from life-threatening damage at the design earthquake ground motion (defined by the standard as two-thirds of the MCE) is

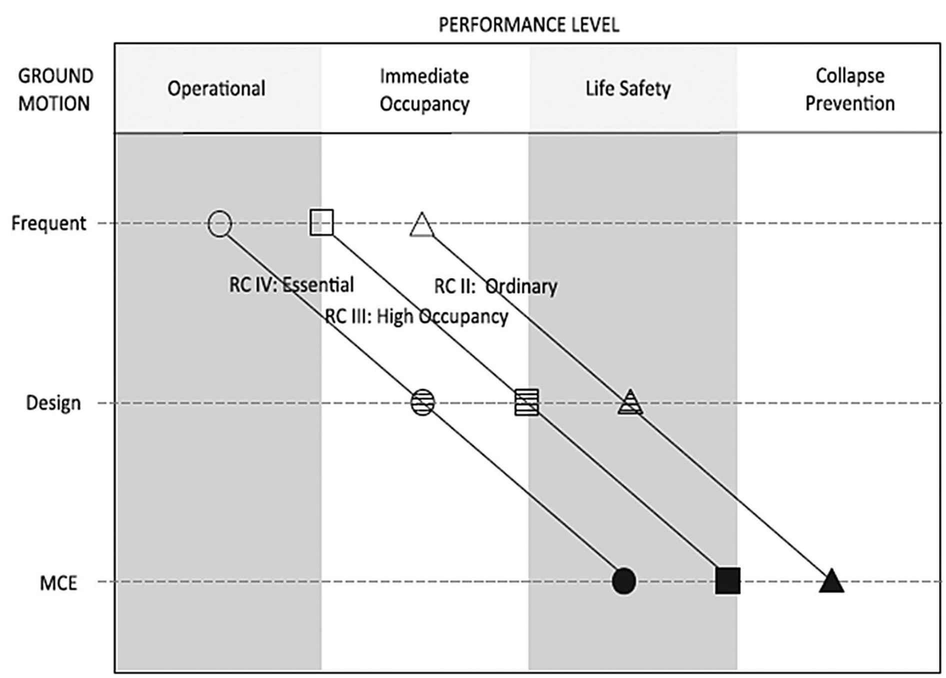

Figure C11.5-1. Expected performance as related to risk category and level of ground motion.

shown by the hatched triangle. The performance implied for the higher Risk Categories III and IV is shown by squares and circles, respectively. The performance anticipated for less severe ground motion is shown by open symbols.

C11.5.1 Importance Factor The Importance Factor, I_e, is used throughout the standard in quantitative criteria for strength. In most of those quantitative criteria, the Importance Factor is shown as a divisor on the factor R or R_p to reduce damage for important structures in addition to preventing collapse in larger ground motions. The R and R_p factors adjust the computed linear elastic response to a value appropriate for design; in many structures, the largest component of that adjustment is ductility (the ability of the structure to undergo repeated cycles of inelastic strain in opposing directions). For a given strength demand, reducing the effective R factor (by means of the Importance Factor) increases the required yield strength, thus reducing ductility demand and related damage.

C11.5.2 Protected Access for Risk Category IV Those structures considered Essential Facilities for response and recovery efforts must be accessible to carry out their purpose. For example, if the collapse of a simple canopy at a hospital could block ambulances from the emergency room admittance area, then the canopy must meet the same structural standard as the hospital. The protected access requirement must be considered in the siting of Essential Facilities in densely built urban areas.

C11.6 SEISMIC DESIGN CATEGORY

Seismic design categories (SDCs) provide a means to step progressively from simple, easily performed design and construction procedures and minimums to more sophisticated, detailed, and costly requirements as both the level of seismic hazard and the consequence of failure escalate. The SDCs are used to trigger requirements that are not scalable; such requirements are either on or off. For example, the basic amplitude of ground motion for design is scalable—the quantity simply increases in a continuous fashion as one moves from a low hazard area to a high hazard area. However, a requirement to avoid weak stories is not particularly scalable. Requirements such as this create step functions. There are many such requirements in the standard, and the SDCs are used systematically to group these step functions. (Further examples include whether seismic anchorage of nonstructural components is required or not, whether particular inspections will be required or not, and structural height limits applied to various seismic force-resisting systems.)

In this regard, SDCs perform one of the functions of the seismic zones used in earlier US building. However, SDCs also depend on a building's occupancy and, therefore, its desired performance. Furthermore, unlike the traditional implementation of seismic zones, the ground motions used to define the SDCs include the effects of individual site conditions on probable ground-shaking intensity.

In developing the ground-motion limits and design requirements for the various seismic design categories, the equivalent modified Mercalli intensity (MMI) scale was considered. There are now correlations of the qualitative MMI scale with quantitative characterizations of ground motions. The reader is encouraged to consult any of a great many sources that describe the MMIs. The following list is a coarse generalization:

MMI V: No real damage
MMI VI: Light nonstructural damage
MMI VII: Hazardous nonstructural damage
MMI VIII: Hazardous damage to susceptible structures
MMI IX: Hazardous damage to robust structures

When the current design philosophy was adopted from the 1997 NEHRP provisions and commentary (FEMA 1997a, b), the upper limit for SDC A was set at roughly one-half of the lower threshold for MMI VII, and the lower limit for SDC D was set at roughly the lower threshold for MMI VIII. However, the lower limit for SDC D was more consciously established by equating that design value (two-thirds of the MCE) to one-half of what had been the maximum design value in building codes over the period of 1975 to 1995. As more correlations between MMI and numerical representations of ground motion have been created, it is reasonable to make the following correlation between the MMI at MCE ground motion and the seismic design category (all this discussion is for ordinary occupancies):

MMI V: SDC A
MMI VI: SDC B
MMI VII: SDC C
MMI VIII: SDC D
MMI IX: SDC E

An important change was made to the determination of SDC when the current design philosophy was adopted. Earlier editions of the NEHRP provisions used the peak velocity-related acceleration, A_v, to determine a building's seismic performance category. However, this coefficient does not adequately represent the damage potential of earthquakes on sites with soil conditions other than rock. Consequently, the 1997 NEHRP provisions (FEMA 1997a) adopted the use of response spectral acceleration parameters S_{DS} and S_{D1}, which include site soil effects for this purpose.

Except for the lowest level of hazard (SDC A), the SDC also depends on the Risk Categories. For a given level of ground motion, the SDC is one category higher for Risk Category IV structures than for lower risk structures. This rating has the effect of increasing the confidence that the design and construction requirements can deliver the intended performance in the extreme event.

Note that the tables in the standard are at the design level, defined as two-thirds of the MCE level. Also recall that the MMIs are qualitative by their nature and that the above correlation will be more or less valid, depending on which numerical correlation for MMI is used. The numerical correlations for MMI roughly double with each step, so correlation between design earthquake ground motion and MMI is not as simple or convenient.

In sum, at the MCE level, SDC A structures should not see motions that are normally destructive to structural systems, whereas the MCE level motions for SDC D structures can destroy vulnerable structures. The grouping of step function requirements by SDC is such that there are a few basic structural integrity requirements imposed at SDC A, graduating to a suite of requirements at SDC D based on observed performance in past earthquakes, analysis, and laboratory research.

The nature of ground motions within a few kilometers of a fault can be different from more distant motions. For example, some near-fault motions have strong velocity pulses, associated with forward rupture directivity, that tend to be highly destructive to irregular structures, even if they are well detailed. For ordinary occupancies, the boundary between SDCs D and E is set to define sites likely to be close enough to a fault that these unusual ground motions may be present. Note that this boundary is defined in terms of mapped bedrock outcrop motions affecting response at 1 s, not site-adjusted values, to better discriminate between sites near and far from faults. Short-period response is not normally as affected as the longer period response. The additional design criteria imposed on structures in SDCs E and F specifically are intended to provide acceptable performance under these very intense near-fault ground motions.

For most buildings, the SDC is determined without consideration of the building's period. Structures are assigned to an SDC based on the more severe condition determined from 1 s acceleration and short-period acceleration. This assigning is done for several reasons. Perhaps the most important of these is that it is often difficult to estimate precisely the period of a structure using the default procedures contained in the standard. Consider, for example, the case of rigid wall–flexible diaphragm buildings, including low-rise reinforced masonry and concrete tilt-up buildings with either untopped metal deck or wood diaphragms. The formula in the standard for determining the period of vibration of such buildings is based solely on the structural height, h_n, and the length of wall present. These formulas typically indicate very short periods for such structures, often on the order of 0.2 s or less. However, the actual dynamic behavior of these buildings often is dominated by the flexibility of the diaphragm—a factor neglected by the formula for approximate fundamental period. Large buildings of this type can have actual periods on the order of 1 s or more. To avoid misclassifying a building's SDC by inaccurately estimating the fundamental period, the standard generally requires that the more severe SDC determined on the basis of short- and long-period shaking be used.

Another reason for this requirement is a desire to simplify building regulation by requiring all buildings on a given soil profile in a particular region to be assigned to the same SDC, regardless of the structural type. This assignment has the advantage of permitting uniform regulation in the selection of seismic force-resisting systems, inspection and testing requirements, seismic design requirements for nonstructural components, and similar aspects of the design process regulated on the basis of SDC, within a community.

Notwithstanding the above, it is recognized that classification of a building as SDC C instead of B or D can have a significant impact on the cost of construction. Therefore, the standard includes an exception permitting the classification of buildings that can reliably be classified as having short structural periods on the basis of short-period shaking alone.

Local or regional jurisdictions enforcing building regulations may desire to consider the effect of the maps, typical soil conditions, and seismic design categories on the practices in their jurisdictional areas. For reasons of uniformity of practice or reduction of potential errors, adopting ordinances could stipulate particular values of ground motion, particular site classes, or particular seismic design categories for all or part of the area of their jurisdiction. For example,

- An area with a historical practice of high seismic zone detailing might mandate a minimum SDC of D regardless of ground motion or site class.
- A jurisdiction with low variation in ground motion across the area might stipulate particular values of ground motion rather than requiring the use of maps.
- An area with unusual soils might require use of a particular site class unless a geotechnical investigation proves a better site class.

C11.7 DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORY A

The 2002 edition of the standard included a new provision of minimum lateral force for Seismic Design Category A structures. The minimum load is a structural integrity issue related to the load path. It is intended to specify design forces in excess of wind loads in heavy low-rise construction. The design calculation in Section 1.4.2 of the standard is simple and easily done to ascertain if the seismic load or the wind load governs. This provision requires a minimum lateral force of 1% of the total gravity load assigned to a story to ensure general structural integrity.

Seismic Design Category A is assigned when the MCE ground motions are below those normally associated with hazardous damage. Damaging earthquakes are not unknown or impossible in such regions, however, and ground motions close to such events may be large enough to produce serious damage. Providing a minimum level of resistance reduces both the radius over which the ground motion exceeds structural capacities and the resulting damage in such rare events. There are reasons beyond seismic risk for minimum levels of structural integrity.

The requirements for SDC A in Section 1.4 are all minimum strengths for structural elements stated as forces at the level appropriate for direct use in the strength design load combinations of Section 2.3. The two fundamental requirements are a minimum strength for a structural system to resist lateral forces (Section 1.4.2) and a minimum strength for connections of structural members (Section 1.4.3).

For many buildings, the wind force controls the strength of the lateral-force-resisting system, but for low-rise buildings of heavy construction with large plan aspect ratios, the minimum lateral force specified in Section 1.4.2 may control. Note that the requirement is for strength and not for toughness, energy-dissipation capacity, or some measure of ductility. The force level is not tied to any postulated seismic ground motion. The boundary between SDCs A and B is based on a spectral response acceleration of 25% of gravity (MCE level) for short-period structures; clearly the 1% acceleration level [from Equation (1.4-1)] is far smaller. For ground motions below the A/B boundary, the spectral displacements generally are on the order of a few inches or less depending on period. Experience has shown that even a minimal strength is beneficial in providing resistance to small ground motions, and it is an easy provision to implement in design. The low probability of motions greater than the MCE is a factor in taking the simple approach without requiring details that would produce a ductile response. Another factor is that larger design forces are specified in Section 1.4.3 for connections between main elements of the lateral force load path.

The minimum connection force is specified in three ways: a general minimum horizontal capacity for all connections; a special minimum for horizontal restraint of in-line beams and trusses, which also includes the live load on the member; and a special minimum for horizontal restraint of concrete and masonry walls perpendicular to their plane (Section 1.4.4). The 5% coefficient used for the first two is a simple and convenient value that provides some margin over the minimum strength of the system as a whole.

C11.8 GEOLOGIC HAZARDS AND GEOTECHNICAL INVESTIGATION

In addition to this commentary, Part 3 of the 2009 *NEHRP Recommended Provisions* (FEMA 2009) includes additional and more detailed discussion and guidance on evaluation of geologic hazards and determination of seismic lateral pressures.

C11.8.1 Site Limitation for Seismic Design Categories E and F Because of the difficulty of designing a structure for the direct shearing displacement of fault rupture and the relatively high seismic activity of SDCs E and F, locating a structure on an active fault that has the potential to cause rupture of the ground surface at the structure is prohibited.

C11.8.2 Geotechnical Investigation Report Requirements for Seismic Design Categories C through F Earthquake motion is only one factor in assessing potential for geologic and seismic hazards. All of the listed hazards can lead to surface ground displacements with potential adverse consequences to structures. Finally, hazard identification alone has little value unless mitigation options are also identified.

C11.8.3 Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F Provisions for computing peak ground acceleration (PGA) for soil liquefaction and stability evaluations were introduced in this section in ASCE 7-16. Of particular note in this section is the explicitly stated requirement that liquefaction must now be evaluated for maximum considered earthquake geometric mean (MCE_G) peak ground acceleration (PGA_M), where the parameter PGA_M includes site effects. Values of the parameter PGA_M are archived in the USGS Seismic Design Geodatabase at gridded locations across United States regions of interest. Values are provided online by the USGS Seismic Design Web Service for user-specified site location (latitude and longitude) and site class, by spatially interpolating between the gridded values of PGA_M based on site location. Mapped values of PGA_M are provided in Chapter 22 for default site conditions.

PGA Provisions. Item 2 of Section 11.8.3 states that Peak Ground Acceleration (PGA) shall be determined based on either a site-specific study, taking into account soil amplification effects, or from the USGS Seismic Design Geodatabase via the USGS Seismic Design Web Service, for the site location and site class of interest. This methodology for determining Peak Ground Acceleration for liquefaction provides an alternative to conducting site response analysis using rock PGA by providing a site-adjusted ground surface acceleration (PGA_M) that can directly be applied in the widely used empirical correlations for assessing liquefaction potential. Correlations for evaluating liquefaction potential are elaborated on in Resource Paper RP 12, "Evaluation of Geologic Hazards and Determination of Seismic Lateral Earth Pressures," published in the 2009 NEHRP provisions (FEMA 2009).

There is an important difference in the derivation of the PGA maps and the maps of S_s and S_1 in ASCE 7-10. Unlike previous editions of ASCE 7, the S_s and S_1 maps in ASCE 7-10 were derived for the "maximum direction shaking" and are risk based rather than hazard based. However, the PGA maps have been derived based on the geometric mean of the two horizontal components of motion. The geometric mean was used in the PGA maps rather than the PGA for the maximum direction shaking to ensure that there is consistency between the determination of PGA and the basis of the simplified empirical field procedure for estimating liquefaction potential based on results of standard penetration tests (SPTs), cone penetrometer tests (CPTs), and other similar field investigative methods. When these correlations were originally derived, the geomean (or a similar metric) of peak ground acceleration at the ground surface was used to identify the cyclic stress ratio for sites with or without liquefaction. The resulting envelopes of data define the liquefaction cyclic resistance ratio (CRR). Rather than reevaluating these case histories for the "maximum direction shaking," it was decided to develop maps of the geomean PGA and to continue using the existing empirical methods.

Liquefaction Evaluation Requirements. Beginning with ASCE 7-02, it has been the intent that liquefaction potential be evaluated at MCE ground motion levels. There was ambiguity in the previous requirement in ASCE 7-05 as to whether liquefaction potential should be evaluated for the MCE or for the design earthquake. Paragraph 2 of Section 11.8.3 of ASCE 7-05 stated that liquefaction potential would be evaluated for the design earthquake; it also stated that in the absence of a site-specific study, peak ground acceleration shall be assumed to be equal to $S_s/2.5$ (S_s is the MCE short-period response spectral acceleration on Site Class B rock). There has also been a difference in provisions between ASCE 7-05 and the 2006 edition of the International Building Code, in which Section 1802.2.7 stated that liquefaction shall be evaluated for the design earthquake ground motions, and the default value of peak ground acceleration in the absence of a site-specific study was given as $S_{DS}/2.5$ (S_{DS} is the short-period site-adjusted design response spectral acceleration). Item 2 of Section 11.8.3, require explicitly that liquefaction potential and other effects be evaluated based on the MCE_G peak ground acceleration.

The explicit requirement in ASCE 7-10 and ASCE 7-16 to evaluate liquefaction for MCE ground motion, rather than to design earthquake ground motion, ensures that the full potential for liquefaction is addressed during the evaluation of structure stability, rather than a lesser level when the design earthquake is used. This change also ensures that, for the MCE ground motion, the performance of the structure is considered under a consistent hazard level for the effects of liquefaction, such as Collapse Prevention or Life Safety, depending on the risk category for the structure (Figure C11.5-1). By evaluating liquefaction for the MCE rather than the design earthquake peak ground acceleration, the ground motion for the liquefaction assessment increases by a factor of 1.5. This increase in peak ground acceleration to the MCE level means that sites that previously were nonliquefiable could now be liquefiable, and sites where liquefaction occurred to a limited extent under the design earthquake could undergo more liquefaction, in terms of depth and lateral extent. Some mechanisms that are directly related to the development of liquefaction, such as lateral spreading and flow or ground settlement, could also increase in severity.

This change in peak ground acceleration level for the liquefaction evaluation addressed an issue that has existed and has periodically been discussed since the design earthquake concept was first suggested in the 1990s. The design earthquake ground motion was obtained by multiplying the MCE ground motion by a factor of $2/3$ to account for a margin in capacity in most buildings. Various calibration studies at the time of code development concluded that for the design earthquake, most buildings had a reserve capacity of more than 1.5 relative to collapse. This reserve capacity allowed the spectral accelerations for the MCE to be reduced using a factor of $2/3$, while still achieving safety from collapse. However, liquefaction potential is evaluated at the selected MCE_G peak ground acceleration and is typically determined to be acceptable if the factor of safety is greater than 1.0, meaning that there is no implicit safety margin on liquefaction potential. By multiplying peak ground acceleration by a factor of $2/3$, liquefaction would be assessed at an effective return period or probability of exceedance different than that for the MCE. However, ASCE 7-10 requires that liquefaction be evaluated for the MCE.

Item 3 of Section 11.8.3 lists the various potential consequences of liquefaction, seismically induced permanent ground displacement, and soil strength loss that must be assessed; soil downdrag and loss in lateral soil reaction for pile foundations are additional consequences that have been included in this paragraph. This section of the new provisions, as in previous editions, does not present specific seismic criteria for the design of the foundation or substructure, but item 4 does state that the geotechnical report must include discussion of possible measures to mitigate these consequences.

A liquefaction resource document has been prepared in support of these revisions to Section 11.8.3. The resource document "Evaluation of Geologic Hazards and Determination of Seismic Lateral Earth Pressures" includes a summary of methods that are currently being used to evaluate liquefaction potential and the limitations of these methods. This summary appears as Resource Paper RP 12 in the 2009 NEHRP provisions (FEMA 2009). The resource document summarizes alternatives for evaluating liquefaction potential, methods for evaluating the possible consequences of liquefaction (e.g., loss of ground support and increased lateral earth pressures), and methods of mitigating the liquefaction hazard. The resource document also identifies alternate methods of evaluating liquefaction hazards, such as analytical and physical modeling. Reference is made to the use of nonlinear effective stress methods for modeling the buildup in pore water pressure during seismic events at liquefiable sites.

Evaluation of Dynamic Seismic Lateral Earth Pressures. The dynamic lateral earth pressure on basement and retaining walls during earthquake ground shaking is considered to be an earthquake load, E, for use in design load combinations. This dynamic earth pressure is superimposed on the preexisting static lateral earth pressure during ground shaking. The preexisting static lateral earth pressure is considered to be an H load.

C11.9 VERTICAL GROUND MOTIONS FOR SEISMIC DESIGN

C11.9.1 General The provisions for developing vertical spectra apply in the western US because that is the main region for which models for vertical-component response spectra are available. The boundary line of -105 degrees longitude comes from the approximate eastern limit of ground motion models for active tectonic regions, as given in Figure 1.1 of Goulet et al. (2017).

ASCE 7-16 included recommendation for developing vertical spectra in the central and eastern United States as presented by EPRI (2015, Appendix A); those recommendations apply to relatively old versions of western United States models for applications in the east. Since the application of western models in the central and eastern United States has not been demonstrated, a simple two-thirds rule is suggested in lieu of the more complex model in these provisions.

C11.9.2 MCE$_R$ Vertical Response Spectrum Recent studies of horizontal and vertical ground motions (e.g., Bozorgnia and Campbell 2004, 2016a, 2016b; Gülerce et al. 2017; Stewart et al. 2016) have shown that vertical ground motion is different from horizontal ground motion in several important respects: (1) vertical ground motion has a larger proportion of short-period (high-frequency) spectral content than horizontal ground motion, and this difference increases with decreasing soil stiffness; (2) vertical ground motion attenuates at a higher rate than horizontal ground motion, and this difference increases with decreasing distance from the earthquake; and (3) the nonlinear component of site response is stronger in the horizontal component than in the vertical component, which causes the vertical/horizontal (*V/H*) spectral ratio to exceed unity for soil sites at close distance to large faults where nonlinear effects are significant. The observed differences in the spectral content and attenuation rate of vertical and horizontal ground motion lead to the following observations regarding the V/H spectral ratio:

1. The V/H spectral ratio is sensitive to spectral period, distance from the earthquake, local site conditions, and earthquake magnitude and is insensitive to earthquake mechanism and sediment depth;
2. The V/H spectral ratio has a distinct peak at short periods that generally exceeds $2/3$ in the near-source region of an earthquake; and
3. The V/H spectral ratio is generally less than $2/3$ at mid-to-long periods.

The procedure for defining the MCE$_R$ vertical response is keyed to the MCE$_R$ spectral response acceleration parameter, S_{aM}. The procedure is based on the studies of horizontal and vertical ground motions by Campbell and Bozorgnia (2003) and Bozorgnia and Campbell (2004), and a series of models generated in the NGA-West2 project (Bozorgnia and Campbell 2016a, b; Gülerce et al. 2017, Stewart et al. 2016).

The specification of vertical ground motions in Section 11.9.2 is based on the product of the multi-period risk-targeted maximum considered earthquake ground motion (S_{aM}) and a simplified representation of the *V/H* spectral ratio that has five regions defined by the vertical period of vibration (T_v). Based on the study of Bozorgnia and Campbell (2004), the periods that define these regions are approximately constant with respect to the magnitude of the earthquake, the distance from the earthquake, and the local site conditions.

The MCE$_R$ is based on maximum direction parameters, so it must be converted back to the geometric mean component to be consistent with the studies referenced above. The NGA-West2 models used geometric mean component horizontal spectral parameters to compute the ratio of vertical to horizontal. To reduce the S_{aM} from the maximum direction to the geometric mean component, S_{aM} is divided by a factor, F_{md} [Equations (11.9-1) to (11.9-5)]. Those F_{md} factors are consistent with the commentary of Chapter 21, Resource Paper 4 in the 2015 provisions, and Shahi and Baker (2014).

The equations in Section 11.9.2 that are used to define the design vertical response spectrum are based on four considerations (adapted from Bozorgnia and Campbell 2004):

1. The short-period part of the 5% damped vertical response spectrum is controlled by the spectral acceleration at $T_v = 0.1$ s;
2. The mid-period part of the vertical response spectrum is controlled by a spectral acceleration that decays as the inverse of a power of the vertical period of vibration. This was taken as $T_v^{-0.75}$ in the 2009 NEHPR provisions and has been updated to $T_v^{-0.5}$;
3. The short-period part of the V/H spectral ratio is a function of the local site conditions (i.e., V_{S30}) and the level of seismic demand (represented in Table 11.9-1 by parameter S_s); and
4. For vertical vibration periods $T_v >$ about 0.3 to 0.5 sec, V/H spectral ratios saturate to values typically less than 0.5 that are relatively consistent with respect to period across this period range. For simplicity, V/H spectral ratios are taken as 0.5 in this range of periods.

The following description of the detailed procedure listed in Section 11.9.2 refers to the illustrated MCE$_R$ vertical response spectrum in Figure C11.9-1.

Vertical Periods Less than or Equal to 0.025 s. Equation (11.9-1) defines that part of the MCE$_R$ vertical response spectrum that is controlled by the vertical peak ground acceleration. The f_1 factor (taken as 0.65) was selected to approximately match V/H ordinates from recent NGA-West2 models for soil

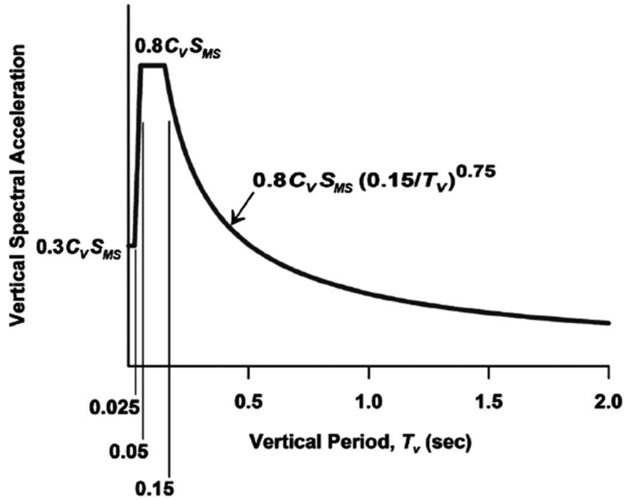

Figure C11.9-1. Illustrative example of the vertical response spectrum.

sites classes (it is somewhat unconservative in this period range for rock sites). The vertical coefficient, C_v, in Table 11.9-1 accounts for the site dependence of V/H ordinates.

Vertical Periods Greater than 0.025 s and Less than or Equal to 0.05 s. Equation (11.9-2) defines that part of the $\mathrm{MCE_R}$ vertical response spectrum for which the V/H ratio linearly transitions from the part of the spectrum that is controlled by the vertical peak ground acceleration to the part that is controlled by the dynamically amplified short-period spectral plateau. The factor of 16 is required to provide appropriate levels of amplification at the peak of the V/H spectral ratio plot.

Vertical Periods Greater than 0.05 s and Less than or Equal to 0.1 s. Equation (11.9-3) defines that part of the $\mathrm{MCE_R}$ vertical response spectrum for which the V/H spectral ratio is dynamically amplified to a short-period plateau at amplitude f_2 in Figure C11.9-2. The width of the peak from 0.05 to 0.1 s is best suited to soil sites (Classes C to DE), being conservative for rock sites (Classes BC to A).

Vertical Periods Greater than 0.1 s and Less than or Equal to 2.0 s. Equation (11.9-4) defines that part of the $\mathrm{MCE_R}$ vertical response spectrum for which the V/H spectral ratio decays with the inverse of the vertical period of vibration raised to the -f_3 power (currently taken as -0.5, formerly -0.75 in ASCE 7-16). This portion of the spectrum was constructed in a generally conservative manner, as two of the three NGA-West2 models suggest that the period range of post-peak decay is steeper than implied by the -0.5 power, with approximately flat V/H ratios at periods longer than about 0.3 to 0.5 s. The flat V/H ratios at periods beyond 0.3-0.5 s are typically less than 0.5, and a limiting value of 0.5 is suggested in the absence of site-specific analysis. This limit of 0.5 is considered a reasonable, but somewhat conservative, lower bound (Campbell and Bozorgnia 2003, Bozorgnia and Campbell 2004).

Vertical Periods Greater than 2.0 s and Less than or Equal to 10.0 s. Equation (11.9-5) defines that part of the $\mathrm{MCE_R}$ vertical response spectrum for which the V/H spectral ratio is roughly constant. A recommended lower limit of 0.5 is provided for this range.

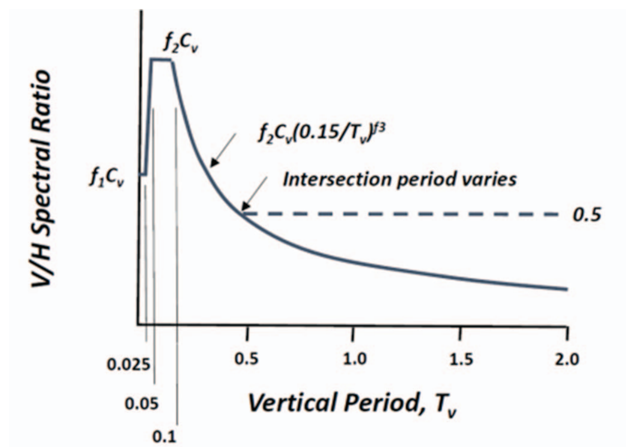

Figure C11.9-2. Illustrative example of the vertical/ horizontal spectral ratio.

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