SECTION 9.0
EARTHQUAKE LOADS

[Note to user: This Section is based on the 2000 NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings.]

SECTION 9.1
GENERAL PROVISIONS

9.1.1 Purpose. Section 9 presents criteria for the design and construction of buildings and similar structures subject to earthquake ground motions. The specified earthquake loads are based on post-elastic energy dissipation in the structure, and because of this fact, the provisions for design, detailing, and construction shall be satisfied even for structures and members for which load combinations that do not contain the earthquake effect indicate larger demands than combinations including earthquake.

9.1.2 Scope and Application.

9.1.2.1 Scope. Every building, and portion thereof, shall be designed and constructed to resist the effects of earthquake motions as prescribed by these provisions. Certain nonbuilding structures, as described in Section 9.14, are within the scope and shall be designed and constructed as required for buildings. Additions to existing structures also shall be designed and constructed to resist the effects of earthquake motions as prescribed by these provisions. Existing structures and alterations to existing structures need only comply with these provisions when required by Sections 9.1.2.2 and 9.1.2.3.

Exceptions:

1. Structures located where the mapped spectral response acceleration at 1-sec period, S_1, is less than or equal to 0.04 g and the mapped short period spectral response acceleration, S_S, is less than or equal to 0.15 g shall only be required to comply with Section 9.5.2.6.1.
2. Detached one- and two-family dwellings that are located where the mapped, short period, spectral response acceleration, S_S, is less than 0.4 g or where the Seismic Design Category determined in accordance with Section 9.4.2 is A, B, or C are exempt from the requirements of these provisions.
3. Detached one- and two-family wood frame dwellings not included in Exception 2 with not more than 2 stories and satisfying the limitations of International Code Council (ICC) [Ref. 9.12-10] are only required to be constructed in accordance with Ref. 9.12-10.
4. Agricultural storage structures that are intended only for incidental human occupancy are exempt from the requirements of these provisions.

Special structures including, but not limited to, vehicular bridges, transmission towers, piers and wharves, hydraulic structures, and nuclear reactors require special consideration of their response characteristics and environment that is beyond the scope of these provisions.

9.1.2.2 Additions to Existing Structures. Additions shall be made to existing structures only as follows:

9.1.2.2.1 An addition that is structurally independent from an existing structure shall be designed and constructed in accordance with the seismic requirements for new structures.

9.1.2.2.2 An addition that is not structurally independent from an existing structure shall be designed and constructed such that the entire structure conforms to the seismic force-resistance requirements for new structures unless the following three conditions are complied with:

1. The addition shall comply with the requirements for new structures.
2. The addition shall not increase the seismic forces in any structural element of the existing structure by more than 5% unless the capacity of the element subject to the increased forces is still in compliance with these provisions.
3. The addition shall not decrease the seismic resistance of any structural element of the existing structure unless the reduced resistance is equal to or greater than that required for new structures.

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

Exceptions:

1. When a change of use results in a structure being reclassified from Seismic Use Group I to Seismic Use Group II and the structure is located in a seismic map area where $S_{DS} < 0.33$, compliance with these provisions is not required.
2. Specific seismic detailing provisions of Appendix A required for a new structure are not required to be met when it can be shown that the level of performance and seismic safety is equivalent to that of a new structure. Such analysis shall consider the regularity, overstrength, redundancy, and ductility of the structure within the context of the existing and retrofit (if any) detailing provided.

9.1.2.4 Application of Provisions. Buildings and structures within the scope of these provisions shall be designed and constructed as required by this Section. When required by the authority having jurisdiction, design documents shall be submitted to determine compliance with these provisions.

9.1.2.4.1 New Buildings. New buildings and structures shall be designed and constructed in accordance with the quality assurance requirements of Section 9.2.1. The analysis and design of structural systems and components, including foundations, frames, walls, floors, and roofs shall be in accordance with the applicable requirements of Sections 9.5 and 9.7. Materials used in construction and components made of these materials shall be designed and constructed to meet the requirements of Sections 9.8 through 9.12. Architectural, electrical, and mechanical systems and components including tenant improvements shall be designed in accordance with Section 9.6.

9.1.2.5 Alternate Materials and Methods of Construction. Alternate materials and methods of construction to those prescribed in these provisions shall not be used unless approved by the authority having jurisdiction. Substantiating evidence shall be submitted demonstrating that the proposed alternate, for the purpose intended, will be at least equal in strength, durability, and seismic resistance.

9.1.3 Seismic Use Groups. All structures shall be assigned to Seismic Use Group: I, II, or III as specified in Table 9.1.3 corresponding to its Occupancy Category determined from Table 1-1:

9.1.3.1 High Hazard Exposure Structures. All buildings and structures assigned to Seismic Use Group III shall meet the following requirements:

TABLE 9.1.3
SEISMIC USE GROUP

		Seismic Use Group		
		I	II	III
Occupancy Category (Table 1-1)	I	X		
	II	X		
	III		X	
	IV			X

9.1.3.1.1 Seismic Use Group III Structure Protected Access. Where operational access to a Seismic Use Group III structure is required through an adjacent structure, the adjacent structure shall conform to the requirements for Seismic Use Group III structures. Where operational access is less than 10 ft from the 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 Seismic Use Group III structure.

9.1.3.1.2 Seismic Use Group III Function. Designated seismic systems in Seismic Use Group III structures shall be provided with the capacity to function, in so far as practical, during and after an earthquake. Site-specific conditions as specified in Section 9.6.3.8 that could result in the interruption of utility services shall be considered when providing the capacity to continue to function.

9.1.3.2 This Section is intentionally left blank.

9.1.3.3 This Section is intentionally left blank.

9.1.3.4 Multiple Use. Structures having multiple uses shall be assigned the classification of the use having the highest Seismic Use Group except in structures having two or more portions which are structurally separated in accordance with Section 9.5.2.8, each portion shall be separately classified. Where a structurally separated portion of a structure provides access to, egress from, or shares life safety components with another portion having a higher Seismic Use Group, both portions shall be assigned the higher Seismic Use Group.

9.1.4 Occupancy Importance Factor. An occupancy importance factor, I, shall be assigned to each structure in accordance with Table 9.1.4.

9.2.1 Definitions. The definitions presented in this Section provide the meaning of the terms used in these provisions. Definitions of terms that have a specific meaning relative to

TABLE 9.1.4
OCCUPANCY IMPORTANCE FACTORS

Seismic Use Group	*I*
I	1.0
II	1.25
III	1.5

the use of wood, steel, concrete, or masonry are presented in the section devoted to the material (Sections A.9.8 through A.9.12, respectively).

ACTIVE FAULT. A fault determined to be active by the authority having jurisdiction from properly substantiated geotechnical data (e.g., most recent mapping of active faults by the U.S. Geological Survey).

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

ADJUSTED RESISTANCE (*D'*). The reference resistance adjusted to include the effects of all applicable adjustment factors resulting from end-use and other modifying factors. Time effect factor (λ) adjustments are not included.

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 these provisions for the intended use.

ARCHITECTURAL COMPONENT SUPPORT. Those structural members or assemblies of members, including braces, frames, struts, and attachments that transmit all loads and forces between architectural systems, components, or elements and the structure.

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

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

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

BASEMENT. A basement is any story below the lowest story above grade.

BOUNDARY ELEMENTS. Diaphragm and shear wall boundary members to which the diaphragm transfers forces. Boundary members include chords and drag struts at diaphragm and shear wall perimeters, interior openings, discontinuities, and re-entrant corners.

BOUNDARY MEMBERS. Portions along wall and diaphragm edges strengthened by longitudinal and transverse reinforcement. Boundary members include chords and drag struts at diaphragm and shear wall perimeters, interior openings, discontinuities, and re-entrant corners.

BUILDING. Any structure whose use could include 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 foundation.

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

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

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

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

CONCRETE, PLAIN. Concrete that is either unreinforced or contains less reinforcement than the minimum amount specified in Ref. 9.9-1 for reinforced concrete.

CONCRETE, REINFORCED. Concrete reinforced with no less than the minimum amount required by Ref. 9.9-1, prestressed or nonprestressed, and designed on the assumption that the two materials act together in resisting forces.

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

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.

CONTAINER. A large-scale independent component used as a receptacle or vessel to accommodate plants, refuse, or similar uses, not including liquids.

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 when 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 which shall be deemed to occur if the sustainable load reduces to 80% or less of the maximum strength.

DESIGN EARTHQUAKE GROUND MOTION. The earthquake effects that buildings and structures are specifically proportioned to resist as defined in Section 9.4.1.

DESIGN EARTHQUAKE. The earthquake effects that are two-thirds of the corresponding maximum considered earthquake.

DESIGNATED SEISMIC SYSTEMS. The seismic force-resisting system and those architectural, electrical, and mechanical systems or their components that require design in accordance with Section 9.6.1 and for which the component importance factor, I_p, is 1.0.

DIAPHRAGM. Roof, floor, or other membrane or bracing system acting to transfer the lateral forces to the vertical resisting elements. Diaphragms are classified as either flexible or rigid according to the requirements of Section 9.5.2.3.1.

DIAPHRAGM, BLOCKED. A diaphragm in which all sheathing edges not occurring on a framing member are supported on and fastened to blocking.

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 due to the diaphragm moment in a manner analogous to the flanges of a beam. Also applies to shear walls.

DISPLACEMENT.

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

Total design displacement. The design earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for design of the isolation system or an element thereof.

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

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

DRAG STRUT (COLLECTOR, 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-force-resisting elements or distributes forces within the diaphragm or shear wall. A drag strut often is an extension of a boundary element that transfers forces into the diaphragm or shear wall. See Sections 9.5.2.6.3.1 and 9.5.2.6.4.1.

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

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

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.

ESSENTIAL FACILITY. A structure required for post-earthquake recovery.

FACTORED RESISTANCE ($\lambda\phi D$). Reference resistance multiplied by the time effect and resistance factors. This value must be adjusted for other factors such as size effects, moisture conditions, and other end-use factors.

FLEXIBLE EQUIPMENT 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.

FRAME.

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

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

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

Ordinary concentrically braced frame (OCBF). A steel concentrically braced frame in which members and connections are designed in accordance with the provisions of Ref. 9.8-3 without modification.

Special concentrically braced frame (SCBF). A steel or composite steel and concrete concentrically braced frame in which members and connections are designed for ductile behavior. Special concentrically braced frames shall conform to Section A.9.8.1.3.1.

Moment frame.

Intermediate moment frame (IMF). A moment frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members. Intermediate moment frames of reinforced concrete shall conform to Ref. 9.9-1. Intermediate moment frames of structural steel construction shall conform to Section 9.8-3 or 10. Intermediate moment frames of composite construction shall conform to Ref. 9.10-3, Part II, Section 6.4b, 7, 8, and 10.

Ordinary moment frame (OMF). A moment frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members. Ordinary moment frames shall conform to Ref. 9.9-1, exclusive of Chapter 21, and Ref. 9.8-3, Section 12 or A.9.9.3.1.

Special moment frame (SMF). A moment frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members. Special moment frames shall conform to Ref. 9.8-3 or Ref. 9.9-1.

FRAME 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 frame 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 9.5.2.2.1.

Space frame system. A structural system composed of interconnected members, other than bearing walls, that is capable of supporting vertical loads and, when designed for such an application, is capable of providing resistance to seismic forces.

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 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 reference plane shall be established by the lowest points within the area between the buildings and the lot line or, where the lot line is more than 6 ft (1829 mm) from the structure, between the structure and a point 6 ft (1829 mm) from the structure.

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

HIGH TEMPERATURE ENERGY SOURCE. A fluid, gas, or vapor whose temperature exceeds 220 °F.

INSPECTION, SPECIAL. The observation of the work by the special inspector to determine compliance with the approved construction documents and these standards.

Continuous special inspection. The full-time observation of the work by an approved 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 an approved special inspector who is present in the area where work has been or is being performed.

INSPECTOR, SPECIAL (WHO SHALL BE IDENTIFIED AS THE OWNER'S INSPECTOR). A person approved by the authority having jurisdiction to perform special inspection. The authority having jurisdiction shall have the option to approve the quality assurance personnel of a fabricator as a special inspector.

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

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

ISOLATION SYSTEM. The collection of structural elements that includes all individual isolator units, all structural elements that transfer force between elements of the isolation system, and all connections to other structural elements. The isolation system also includes the wind-restraint system, energy-dissipation devices, and/or the displacement restraint system if such systems and devices are used to meet the design requirements of Section 9.13.

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

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).

LOAD.

Dead load. The gravity load due to the weight of all permanent structural and nonstructural components of a building such as walls, floors, roofs, and the operating weight of fixed service equipment.

Gravity load (W). The total dead load and applicable portions of other loads as defined in Section 9.5.3.2.

Live load. The load superimposed by the use and occupancy of the building not including the wind load, earthquake load, or dead load, see Section 9.5.3.2.

MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION. The most severe earthquake effects considered by these standards as defined in Section 9.4.1.

NONBUILDING STRUCTURE. A structure, other than a building, constructed of a type included in Section 9.14 and within the limits of Section 9.14.1.1.

OCCUPANCY IMPORTANCE FACTOR. A factor assigned to each structure according to its Seismic Use Group as prescribed in Section 9.1.4.

OWNER. Any person, agent, firm, or corporation having a legal or equitable interest in the property.

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.

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

QUALITY ASSURANCE PLAN. A detailed written procedure that establishes the systems and components subject to special inspection and testing. The type and frequency of testing and the extent and duration of special inspection are given in the quality assurance plan.

REFERENCE RESISTANCE (D). The resistance (force or moment as appropriate) of a member or connection computed at the reference end-use conditions.

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 registrations laws of the state in which the project is to be constructed.

ROOFING UNIT. A unit of roofing tile or similar material weighing more than 1 pound.

SEISMIC DESIGN CATEGORY. A classification assigned to a structure based on its Seismic Use Group and the severity of the design earthquake ground motion at the site as defined in Section 9.1.2.5.

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.

SEISMIC RESPONSE COEFFICIENT. Coefficient C_s as determined from Section 9.5.3.2.1.

SEISMIC USE GROUP. A classification assigned to a structure based on its use as defined in Section 9.1.3.

SHALLOW ANCHOR. Anchors with embedment length-to-diameter ratios of less than 8.

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

SITE CLASS. A classification assigned to a site based on the types of soils present and their engineering properties as defined in Section 9.4.1.2.

SITE COEFFICIENTS. The values of F_a and F_v as indicated in Tables 9.4.1.2.4a and 9.4.1.2.4b, respectively.

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

STORAGE RACKS. Include industrial pallet racks, moveable shelf racks, and stacker racks made of cold-formed or hot-rolled structural members. Does not include other types of racks such as drive-in and drive-through racks, cantilever racks, portable racks, or racks made of materials other than steel.

STORY. The portion of a structure between the top of two successive, finished floor surfaces and, for the topmost story, from the top of the floor finish to the top of the roof structural element.

STORY ABOVE GRADE. Any story having its finished floor surface entirely above grade, except that a story shall be considered as a story above grade where the finished floor surface of the story immediately above is more than 6 ft (1829 mm) above the grade plane, more than 6 ft (1829 mm) above the finished ground level for more than 40% of the total structure perimeter, or more than 12 ft (3658 mm) above the finished ground level at any point.

STORY DRIFT. The difference of horizontal deflections at the top and bottom of the story as determined in Section 9.5.3.7.1.

STORY DRIFT RATIO. The story drift, as determined in Section 9.5.3.7.1, divided by the story height.

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 referenced standards) 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.

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

STRUCTURAL OBSERVATIONS. The visual observations performed by the registered design professional in responsible charge (or another registered design professional) to determine that the seismic force-resisting system is constructed in general conformance with the construction documents.

STRUCTURAL-USE PANEL. A wood-based panel product that meets the requirements of Ref. 9.12-3 or Ref. 9.12-5 and is bonded with a waterproof adhesive. Included under this designation are plywood, oriented strand board, and composite panels.

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

TESTING AGENCY. A company or corporation that provides testing and/or inspection services. The person in charge of the special inspector(s) and the testing services shall be a registered design professional.

TIE-DOWN (HOLD-DOWN). A device used to resist uplift of the boundary elements of shear walls. These devices are intended to resist load without significant slip between the device and the shear wall boundary element or be shown with cyclic testing to not reduce the wall capacity or ductility.

TIME EFFECT FACTOR (λ). A factor applied to the adjusted resistance to account for effects of duration of load.

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

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

UTILITY OR SERVICE INTERFACE. The connection of the structure's mechanical and electrical distribution systems to the utility or service company's distribution system.

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

Cripple wall. Short stud wall, less than 8 ft (2400 mm) in height, between the foundation and the lowest framed floors with studs not less than 14 inches long—also known as a knee wall. Cripple walls can occur in both engineered structures and conventional construction.

Light-framed wall. A wall with wood or steel studs.

Light-framed 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. All walls other than bearing walls or shear walls.

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

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.

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

9.2.2 Symbols. The unit dimensions used with the items covered by the symbols shall be consistent throughout except where specifically noted. The symbols and definitions presented in this Section apply to these provisions as indicated.

A, B, C, D, E, F = the Seismic Performance Categories as defined in Tables 9.4.2.1a and 9.4.2.1b

A, B, C, D, E, F = the Site Classes as defined in Section 9.4.1.2

A_{ch} = cross-sectional area (in.^{2} or mm^2) of a component measured to the outside of the special lateral reinforcement

A_0 = the area of the load-carrying foundation (ft^2 or m^2)

A_{sh} = total cross-sectional area of hoop reinforcement (in.^{2} or 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 (in.^{2} or mm^2) of diagonal reinforcement

A_x = the torsional amplification factor, Section 9.5.3.5.2

a_d = the incremental factor related to P-delta effects in Section 9.5.3.7.2

a_p = the amplification factor related to the response of a system or component as affected by the type of seismic attachment, determined in Section 9.6.1.3

B_D = numerical coefficient as set forth in Table 9.13.3.3.1 for effective damping equal to β_D

B_M = numerical coefficient as set forth in Table 9.13.3.3.1 for effective damping equal to β_M

b = the shortest plan dimension of the structure, in ft (mm) measured perpendicular to d

C_d = the deflection amplification factor as given in Table 9.5.2.2

C_s = the seismic response coefficient determined in Section 9.5.3.2 (dimensionless)

C_s = the seismic response coefficient determined in Sections 9.5.5.2.1 and 9.5.5.3.1 (dimensionless)

C_{sm} = the modal seismic response coefficient determined in Section 9.5.4.5 (dimensionless)

C_T = the building period coefficient in Section 9.5.3.3

C_{vx} = the vertical distribution factor as determined in Section 9.5.3.4

c = distance from the neutral axis of a flexural member to the fiber of maximum compressive strain (in. or mm)

D = the effect of dead load

D_D = design displacement, in in. (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Eq. 9.13.3.3.1

D_D' = design displacement, in in. (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Eq. 9.13.4.2-1

D_M = maximum displacement, in in. (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Eq. 9.13.3.3.3

D_M' = maximum displacement, in in. (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Eq. 9.13.4.2-2

D_p = relative seismic displacement that the component must be designed to accommodate as defined in Section 9.6.1.4

D_s = the total depth of the stratum in Eq. 9.5.5.2.1.2-4 (ft or m)

D_{TD} = total design displacement, in in. (mm), of an element of the isolation system including both translational displacement at the center of rigidity and the component of torsional displacement in the direction under consideration as prescribed by Eq. 9.13.3.3.5-1

D_{TM} = total maximum displacement, in in. (mm), of an element of the isolation system including both translational displacement at the center of rigidity and the component of torsional displacement in the direction under consideration as prescribed by Eq. 9.13.3.3.5-2

d = overall depth of member (in. or mm) in Section 9.5

d = the longest plan dimension of the structure, in ft (mm), in Section 9.13

d_p = the longest plan dimension of the structure, in ft (mm)

E = the effect of horizontal and vertical earthquake-induced forces, in Sections 9.5.2.7 and 9.13

E_{loop} = energy dissipated in kip-in. (kN-mm), in an isolator unit during a full cycle of reversible load over a test displacement range from Δ^+ to Δ^-, as measured by the area enclosed by the loop of the force-deflection curve

e = the actual eccentricity, in ft (mm), measured in plan between the center of mass of the structure above the isolation interface and the center of rigidity of the isolation system, plus accidental eccentricity, in ft (mm), taken as 5% of the maximum building dimension perpendicular to the direction of force under consideration

F_a = acceleration-based site coefficient (at 0.3-sec period)

F^- = maximum negative force in an isolator unit during a single cycle of prototype testing at a displacement amplitude of Δ^-

F^+ = positive force in kips (kN) in an isolator unit during a single cycle of prototype testing at a displacement amplitude of Δ^+

F_i, F_n, F_x = the portion of the seismic base shear, V, induced at Level i, n, or x, respectively, as determined in Section 9.5.3.4

F_p = the seismic force acting on a component of a structure as determined in Section 9.5.2.6.1.1, 9.5.2.5.1.3, 9.5.2.6.1.4, or 9.6.1.3

F_v = velocity-based site coefficient (at 1.0-sec period)

F_x = total force distributed over the height of the structure above the isolation interface as prescribed by Eq. 9.13.3.5

F_{xm} = the portion of the seismic base shear, V_m, induced at Level x as determined in Section 9.5.4.6

f_c' = specified compressive strength of concrete used in design

f_s' = ultimate tensile strength (psi or MPa) of the bolt, stud, or insert leg wires. For A307 bolts or A108 studs, it is permitted to be assumed to be 60,000 psi (415 MPa).

f_y = specified yield strength of reinforcement (psi or MPa)

f_{yh} = specified yield stress of the special lateral reinforcement (psi or kPa)

$G = \gamma v_s^2/g$ = the average shear modulus for the soils beneath the foundation at large strain levels (psf or Pa)

$G_0 = \gamma v_{s0}^2/g$ = the average shear modulus for the soils beneath the foundation at small strain levels (psf or Pa)

g = the acceleration due to gravity

H = thickness of soil

h = the 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 = the roof elevation of a structure in Section 9.6

h = the effective height of the building as determined in Section 9.5.5.2 or 9.5.5.3 (ft or m)

h_c = the core dimension of a component measured to the outside of the special lateral reinforcement (in. or mm)

h_i, h_n, h_x = the height above the base Level i, n, or x, respectively

h_{sx} = the story height below Level $x = (h_x - h_{x-1})$

I = the occupancy importance factor in Section 9.1.4

I_0 = the static moment of inertia of the load-carrying foundation, see Section 9.5.5.2.1 (in.^{4} or mm^4)

I_p = the component importance factor as prescribed in Section 9.6.1.5

i = the building level referred to by the subscript i; $i = 1$ designates the first level above the base

K_p = the stiffness of the component or attachment, Section 9.6.3.3

K_y = the lateral stiffness of the foundation as defined in Section 9.5.5.2.1.1 (lb/in. or N/m)

K_θ = the rocking stiffness of the foundation as defined in Section 9.5.5.2.1.1 (ft-lb/degree or N-m/rad)

KL/r = the lateral slenderness of a compression member measured in terms of its effective buckling length, KL, and the least radius of gyration of the member cross-section, r

K_{Dmax} = maximum effective stiffness, in kips/in. (kN/mm), of the isolation system at the design displacement in the horizontal direction under consideration as prescribed by Eq. 9.13.9.5.1-1

K_{Dmin} = minimum effective stiffness, in kips/in. (kN/mm), of the isolation system at the design displacement in the horizontal direction under consideration as prescribed by Eq. 9.13.9.5.1-2

K_{max} = maximum effective stiffness, in kips/in. (kN/mm), of the isolation system at the maximum displacement in the horizontal direction under consideration as prescribed by Eq. 9.13.9.5.1-3

K_{min} = minimum effective stiffness, in kips/in. (kN/mm), of the isolation system at the maximum displacement in the horizontal direction under consideration, as prescribed by Eq. 9.13.9.5.1-4

k = the distribution exponent given in Section 9.5.3.4

k = the stiffness of the building as determined in Section 9.5.5.2.1.1 (lb/ft or N/m)

k_{eff} = effective stiffness of an isolator unit, as prescribed by Eq. 9.13.9.3-1

L = the overall length of the building (ft or m) at the base in the direction being analyzed

L = length of bracing member (in. or mm) in Section A.9.8

L = the effect of live load in Section 9.13

L_0 = the overall length of the side of the foundation in the direction being analyzed, Section 9.5.5.2.1.2 (ft or m)

l = the dimension of a diaphragm perpendicular to the direction of application of force. For open-front structures, l is the length from the edge of the diaphragm at the open-front to the vertical resisting elements parallel to the direction of the applied force. For a cantilevered diaphragm, l is the length of the cantilever.

M_f = the foundation overturning design moment as defined in Section 9.5.3.6 (ft-kip or kN-m)

M_0, M_{01} = the overturning moment at the foundation-soil interface as determined in Sections 9.5.5.2.3 and 9.5.5.3.2 (ft-lb or N-m)

M_f = the foundation overturning design moment as defined in Section 9.5.3.6

M_t = the torsional moment resulting from the location of the building masses, Section 9.5.3.5.2

M_{ta} = the accidental torsional moment as determined in Section 9.5.3.5.2

M_x = the building overturning design moment at Level x as defined in Section 9.5.3.6 or 9.5.4.7

m = a subscript denoting the mode of vibration under consideration; i.e., $m = 1$ for the fundamental mode

N = number of stories, Section 9.5.3.3

N = standard penetration resistance, ASTM D1536-84.

$\overline{N}$ = average field standard penetration resistance for the top 100 ft (30 m), see Section 9.4.1.2

N_{ch} = average standard penetration resistance for cohesionless soil layers for the top 100 ft (30 m), see Section 9.4.1.2

n = designates the level that is uppermost in the main portion of the building

P_D = required axial strength on a column resulting from application of dead load, D, in Section 9.5 (kip or kN)

P_E = required axial strength on a column resulting from application of the amplified earthquake load, E', in Section 9.5 (kip or kN)

P_L = required axial strength on a column resulting from application of live load, L, in Section 9.5 (kip or kN)

P_n = nominal axial load strength (lb or N) in A.9.8

P_n = the algebraic sum of the shear wall and the minimum gravity loads on the joint surface acting simultaneously with the shear (lb or N)

P_u^* = required axial strength on a brace (kip or kN) in Section A.9.8

P_x = the total unfactored vertical design load at, and above, Level x for use in Section 9.5.3.7.2

PI = plasticity index, ASTM D4318–93.

Q_E = the effect of horizontal seismic (earthquake-induced) forces, Section 9.5.2.7

Q_v = the load equivalent to the effect of the horizontal and vertical shear strength of the vertical segment, Section A.9.8

R = the response modification coefficient as given in Table 9.5.2.2

R_p = the component response modification factor as defined in Section 9.6.1.3

r = the characteristic length of the foundation as defined in Section 9.5.5.2.1

r_a = the characteristic foundation length as defined by Eq. 9.5.5.2.1.2-2 (ft or m)

r_m = the characteristic foundation length as defined by Eq. 9.5.5.2.1.2-3 (ft or m)

r_x = the ratio of the design story shear resisted by the most heavily loaded single element in the story, in direction x, to the total story shear

S_S = the mapped maximum considered earthquake, 5% damped, spectral response acceleration at short periods as defined in Section 9.4.1.2

S_1 = the mapped maximum considered earthquake, 5% damped, spectral response acceleration at a period of 1 sec as defined in Section 9.4.1.2

S_{DS} = the design, 5% damped, spectral response acceleration at short periods as defined in Section 9.4.1.2

S_{D1} = the design, 5% damped, spectral response acceleration at a period of 1 sec as defined in Section 9.4.1.2

S_{MS} = the maximum considered earthquake, 5% damped, spectral response acceleration at short periods adjusted for site class effects as defined in Section 9.4.1.2

S_{M1} = the maximum considered earthquake, 5% damped, spectral response acceleration at a period of 1 sec adjusted for site class effects as defined in Section 9.4.1.2

$\overline{S}_u$ = average undrained shear strength in top 100 ft (30 m); see Section 9.4.1.2, ASTM D2166-91 or ASTM D2850-87.

s_h = spacing of special lateral reinforcement (in. or mm)

T = the fundamental period of the building as determined in Section 9.5.3.2.1

T, T_1 = the effective fundamental period (sec) of the building as determined in Sections 9.5.5.2.1.1 and 9.5.5.3.1

T_a = the approximate fundamental period of the building as determined in Section 9.5.3.3

T_D = effective period, in seconds (sec), of the seismically isolated structure at the design displacement in the direction under consideration as prescribed by Eq. 9.13.3.3.2

T_m = the modal period of vibration of the m^{th} mode of the building as determined in Section 9.5.4.5

T_M = effective period, in seconds (sec), of the seismically isolated structure at the maximum displacement in the direction under consideration as prescribed by Eq. 9.13.3.3.4

T_p = the fundamental period of the component and its attachment, Section 9.6.3.3

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

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

T_M = effective period, in seconds (sec), of the seismically isolated structure at the maximum displacement in the direction under consideration as prescribed by Eq. 9.13.3.3.4

T_m = the modal period of vibration (sec) of the m^{th} mode of the building as determined in Section 9.5.4.5

T_4 = net tension in steel cable due to dead load, prestress, live load, and seismic load, Section A.9.8.8

V = the total design lateral force or shear at the base, Section 9.5.3.2

V_b = the total lateral seismic design force or shear on elements of the isolation system or elements below the isolation system as prescribed by Eq. 9.13.3.4.1

V_s = the total lateral seismic design force or shear on elements above the isolation system as prescribed by Eq. 9.13.3.4.2

V_t = the design value of the seismic base shear as determined in Section 9.5.4.8

V_u = required shear strength (lb or N) due to factored loads in Section 9.6

V_x = the seismic design shear in story x as determined in Section 9.5.3.5 or 9.5.4.8

V_1 = the portion of the seismic base shear, V, contributed by the fundamental mode, Section 9.5.5.3 (kip or kN)

ΔV = the reduction in V as determined in Section 9.5.5.2 (kip or kN)

ΔV_1 = the reduction in V_1 as determined in Section 9.5.5.3 (kip or kN)

v_s = the average shear wave velocity for the soils beneath the foundation at large strain levels, Section 9.5.5.2 (ft/s or m/s)

$\bar{v}_s$ = average shear wave velocity in top 100 ft (30 m), see Section 9.4.1.2

v_{so} = the average shear wave velocity for the soils beneath the foundation at small strain levels, Section 9.5.5.2 (ft/s or m/s)

W = the total gravity load of the building as defined in Section 9.5.3.2. For calculation of seismic-isolated building period, W is the total seismic dead load weight of the building as defined in Sections 9.5.5.2 and 9.5.5.3 (kip or kN).

W = the effective gravity load of the building as defined in Sections 9.5.5.2 and 9.5.5.3 (kip or kN)

W_c = the gravity load of a component of the building

W_D = the energy dissipated per cycle at the story displacement for the design earthquake, Section 9.13.3.2

W_m = the effective modal gravity load determined in accordance with Eq. 9.5.4.5-2.

W_p = component operating weight (lb or N)

w = the width of a diaphragm or shear wall in the direction of application of force. For sheathed diaphragms, the width shall be defined as the dimension between the outside faces of the tension and compression chords.

w = moisture content (in percent), ASTM D2216-92 [3]

w_i, w_n, w_x = the portion of W that is located at or assigned to Level i, n, or x, respectively

x = the level under consideration

x = 1 designates the first level above the base

y = elevations difference between points of attachment in Section 9.6

y = the distance, in ft (mm), between the center of rigidity of the isolation system rigidity and the element of interest measured perpendicular to the direction of seismic loading under consideration, Section 9.13

z = the level under consideration; $x = 1$ designates the first level above the base

α = the relative weight density of the structure and the soil as determined in Section 9.5.5.2.1

α = angle between diagonal reinforcement and longitudinal axis of the member (degree or rad)

β = ratio of shear demand to shear capacity for the story between Level x and $x - 1$

β = the fraction of critical damping for the coupled structure-foundation system, determined in Section 9.5.5.2.1

β_D = effective damping of the isolation system at the design displacement as prescribed by Eq. 9.13.9.5.2-1

β_M = effective damping of the isolation system at the maximum displacement as prescribed by Eq. 9.13.5.2-2

β_0 = the foundation damping factor as specified in Section 9.5.5.2.1

β_{eff} = effective damping of the isolation system as prescribed by Eq. 9.13.9.3-2

γ = the average unit weight of soil (lb/ft^3 or kg/m^3)

Δ = the design story drift as determined in Section 9.5.3.7.1

Δ_a = the allowable story drift as specified in Section 9.5.2.8

Δ_m = the design modal story drift determined in Section 9.5.4.6

Δ^+ = maximum positive displacement of an isolator unit during each cycle of prototype testing

Δ^- = maximum negative displacement of an isolator unit during each cycle of prototype testing

δ_{max} = the maximum displacement at Level x, considering torsion, Section 9.5.3.5.2

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x, Section 9.5.3.5.2

δ_x = the deflection of Level x at the center of the mass at and above Level x, Eq. 9.5.3.7.1

δ_{xe} = the deflection of Level x at the center of the mass at and above Level x determined by an elastic analysis, Section 9.5.3.7.1

δ_{xem} = the modal deflection of Level x at the center of the mass at and above Level x determined by an elastic analysis, Section 9.5.4.6

δ_{xm} = the modal deflection of Level x at the center of the mass at and above Level x as determined by Eqs. 9.5.4.6-3 and 9.13.3.2-1

δ_x, δ_{x1} = the deflection of Level x at the center of the mass at and above Level x, Eqs. 9.5.5.2.3-1 and 9.5.5.3.2-1 (in. or mm)

θ = the stability coefficient for P-delta effects as determined in Section 9.5.3.7.2

τ = the overturning moment reduction factor, Eq. 9.5.3.6

ρ = a reliability coefficient based on the extent of structural redundance present in a building as defined in Section 9.5.2.7

ρ_s = spiral reinforcement ratio for precast, prestressed piles in Sections A.9.7.4.4.5 and A.9.7.5.4.4

ρ_x = a reliability coefficient based on the extent of structural redundancy present in the seismic force-resisting system of a building in the x direction

λ = time effect factor

ϕ = the capacity reduction factor

ϕ = the strength reduction factor or resistance factor

ϕ_{im} = the displacement amplitude at the i^{th} level of the building for the fixed-base condition when vibrating in its m^{th} mode, Section 9.5.4.5

Ω_0 = overstrength factor as defined in Table 9.5.2.2

ΣE_D = total energy dissipated, in kip-ins. (kN-mm), in the isolation system during a full cycle of response at the design displacement, D_D

ΣE_M = total energy dissipated, in kip-ins. (kN-mm), in the isolation system during a full cycle of response at the maximum displacement, D_M

$\Sigma|F_D^+|_{max}$ = sum, for all isolator units, of the maximum absolute value of force, in kips (kN), at a positive displacement equal to D_D

$\Sigma|F_D^+|_{min}$ = sum, for all isolator units, of the minimum absolute value of force, in kips (kN), at a positive displacement equal to D_D

$\Sigma|F_D^-|_{max}$ = sum, for all isolator units, of the maximum absolute value of force, in kips (kN), at a negative displacement equal to D_D

$\Sigma|F_D^-|_{min}$ = sum, for all isolator units, of the minimum absolute value of force, in kips (kN), at a negative displacement equal to D_D

$\Sigma|F_M^+|_{max}$ = sum, for all isolator units, of the maximum absolute value of force, in kips (kN), at a positive displacement equal to D_M

$\Sigma|F_M^+|_{min}$ = sum, for all isolator units, of the minimum absolute value of force, in kips (kN), at a positive displacement equal to D_M

$\Sigma|F_M^-|_{max}$ = sum, for all isolator units, of the maximum absolute value of force, in kips (kN), at a negative displacement equal to D_M

$\Sigma|F_M^-|_{min}$ = sum, for all isolator units, of the minimum absolute value of force, in kips (kN), at a negative displacement equal to D_M

SECTION 9.3

This Section is intentionally left blank.

9.4.1 Procedures for Determining Maximum Considered Earthquake and Design Earthquake Ground Motion Accelerations and Response Spectra. Ground motion accelerations, represented by response spectra and coefficients derived from these spectra, shall be determined in accordance with the general procedure of Section 9.4.1.2 or the site-specific procedure of Section 9.4.1.3. The general procedure in which spectral response acceleration parameters for the maximum considered earthquake ground motions are derived using Figure 9.4.1.1a through 9.4.1.1j, modified by site coefficients to include local site effects and scaled to design values, are permitted to be used for any structure except as specifically indicated in these provisions. The site-specific procedure also is permitted to be used for any structure and shall be used where specifically required by these provisions.

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

9.4.1.2 General Procedure for Determining Maximum Considered Earthquake and Design Spectral Response Accelerations. The mapped maximum considered earthquake spectral response acceleration at short periods (S_S) and at 1-sec (S_1) shall be determined respectively from Spectral Acceleration Maps 1 through 32.

For buildings and structures included in the scope of this Standard as specified in Section 9.1.2.1, the Site Class shall be determined in accordance with Section 9.4.1.2.1. The maximum considered earthquake spectral response accelerations adjusted for Site Class effects, S_{MS} and S_{M1}, shall be determined in accordance with Section 9.4.1.2.4 and the design spectral response accelerations, S_{DS} and S_{D1}, shall be determined in accordance with Section 9.4.1.2.5. The general response spectrum, when required by these provisions, shall be determined in accordance with Section 9.4.1.2.6.

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

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

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

TABLE 9.4.1.2
SITE CLASSIFICATION

Site Class	$\overline{v}_s$	$\overline{N}$ or $\overline{N}_{ch}$	$\overline{s}_u$
A Hard rock	>5000 ft/s (>1500 m/s)	not applicable	not applicable
B Rock	2500 to 5000 ft/s (760 to 1500 m/s)	not applicable	not applicable
C Very dense soil and soft rock	1200 to 2500 ft/s (370 to 760 m/s)	> 50	> 2000 psf (> 100 kPa)
D Stiff soil	600 to 1200 ft/s (180 to 370 m/s)	15 to 50	1000 to 2000 psf (50 to 100 kPa)
E Soil	<600 ft/s (<180 m/s)	<15	<1000 psf (<50 kPa)
	Any profile with more than 10 ft of soil having the following characteristics: —Plasticity index PI > 20, —Moisture content $w \geq 40\%$, and —Undrained shear strength $\overline{s}_u < 500$ psf		
F Soils requiring site-specific evaluation		1. Soils vulnerable to potential failure or collapse 2. Peats and/or highly organic clays 3. Very high plasticity clays 4. Very thick soft/medium clays	

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

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

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

F = Soils requiring site-specific evaluations:

1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.

   Exception: For structures having fundamental periods of vibration equal to or less than 0.5-sec, site-specific evaluations are not required to determine spectral accelerations for liquefiable soils. Rather, the Site Class may be determined in accordance with Section 9.4.1.2.2 and the corresponding values of F_a and F_v determined from Tables 9.4.1.2.4a and 9.4.1.2.4b.

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

   Exception: When the soil properties are not known in sufficient detail to determine the Site Class, Class D shall be used. Site Class E shall be used when the authority having jurisdiction determines that Site Class E is present at the site or in the event that Site E is established by geotechnical data.

The following standards are referenced in the provisions for determining the seismic coefficients:

[1] ASTM. "Test Method for Penetration Test and Split-Barrel Sampling of Soils." *ASTM D1586-84*, 1984.

[2] ASTM. "Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils." *ASTM D4318-93*, 1993.

[3] ASTM. "Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock." *ASTM D2216-92*, 1992.

[4] ASTM. "Test Method for Unconfined Compressive Strength of Cohesive Soil." *ASTM D2166-91*, 1991.

[5] ASTM. "Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression." *ASTM D2850-87*, 1987.

This page intentionally left blank.

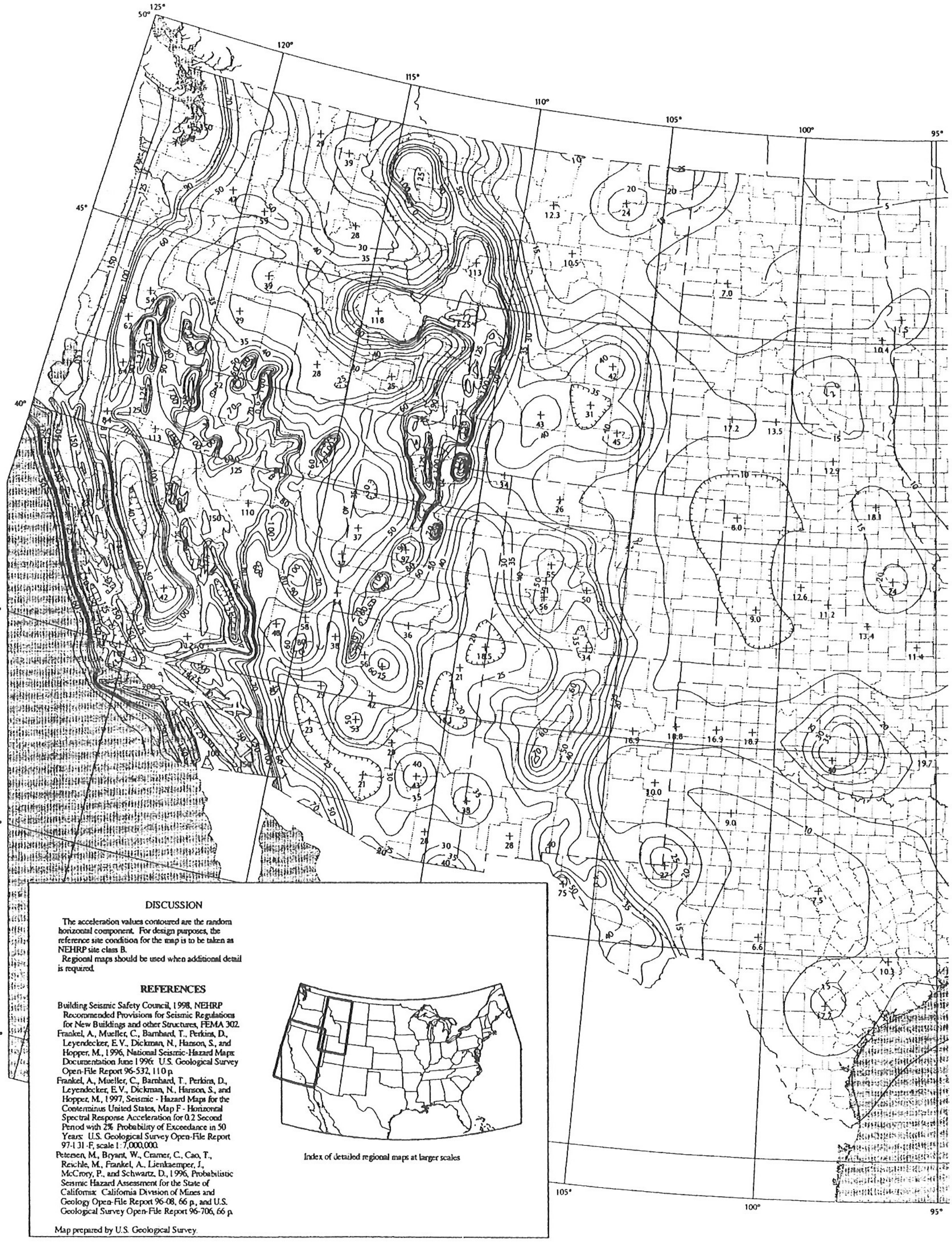


FIGURE 9.4.1.1(a)
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR CONTERMINOUS UNITED STATES. OF 0.2 s SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

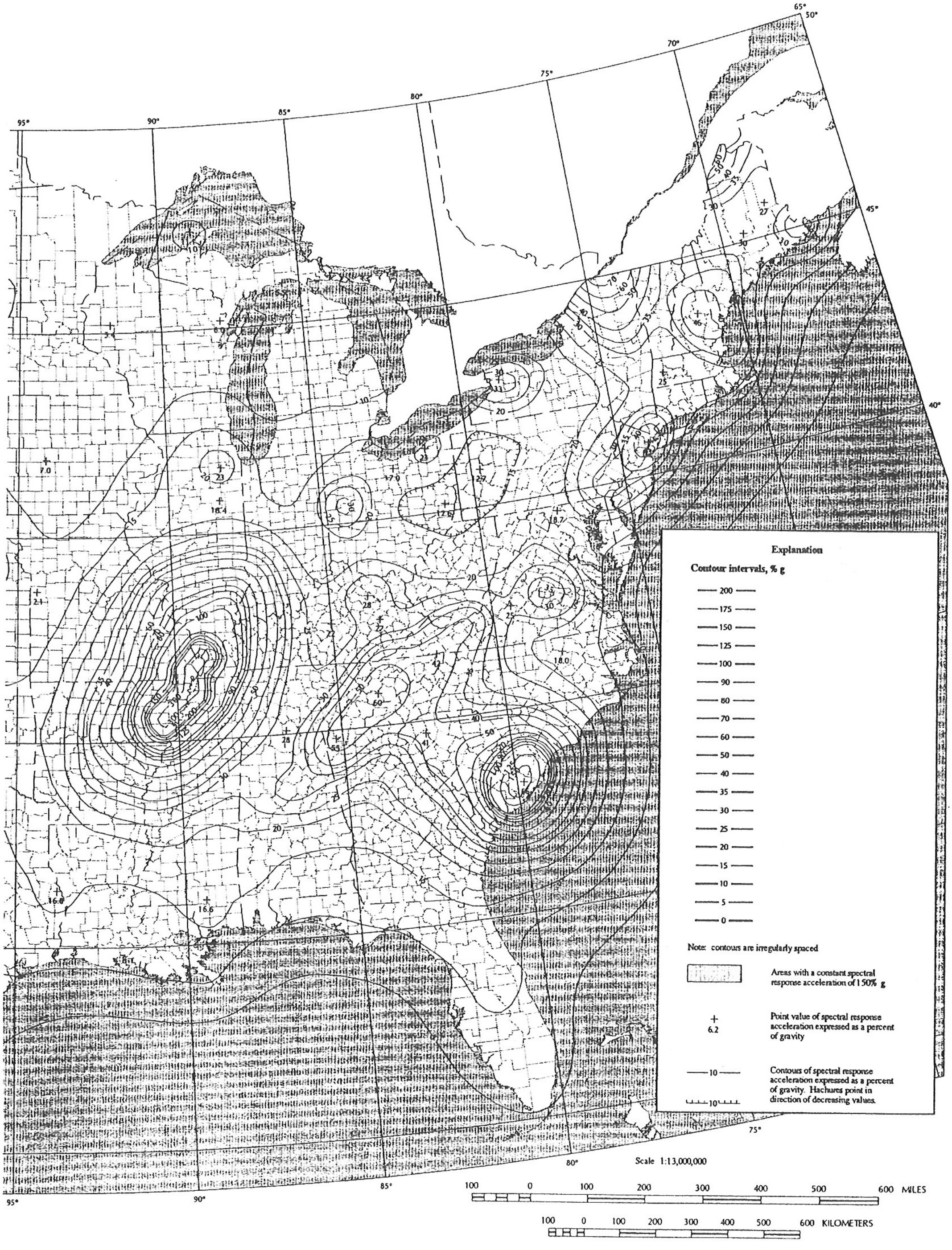


FIGURE 9.4.1.1(a) – continued
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR CONTERMINOUS UNITED STATES. OF 0.2 s SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

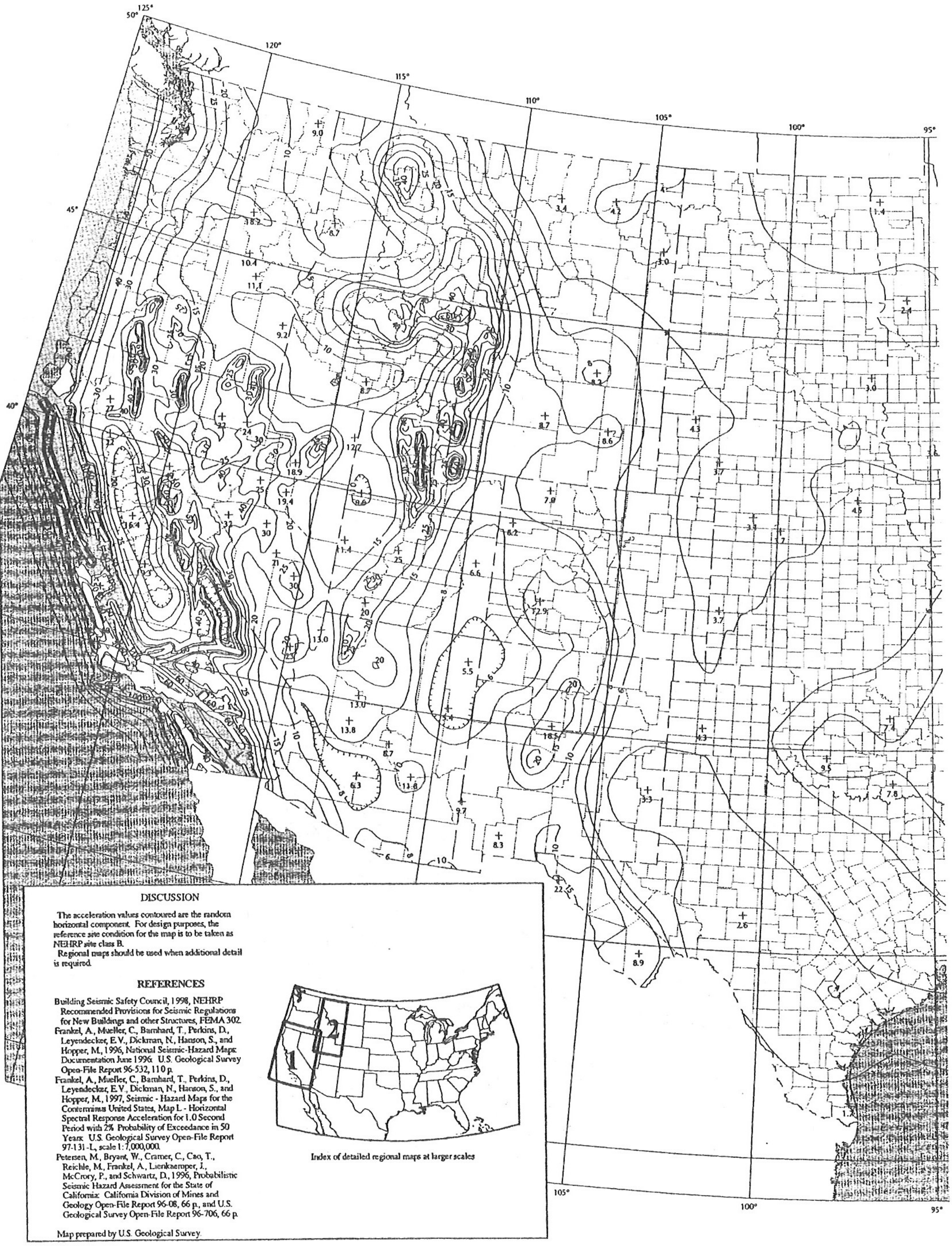


FIGURE 9.4.1.1(b)
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR CONTERMINOUS UNITED STATES OF 1.0 s SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

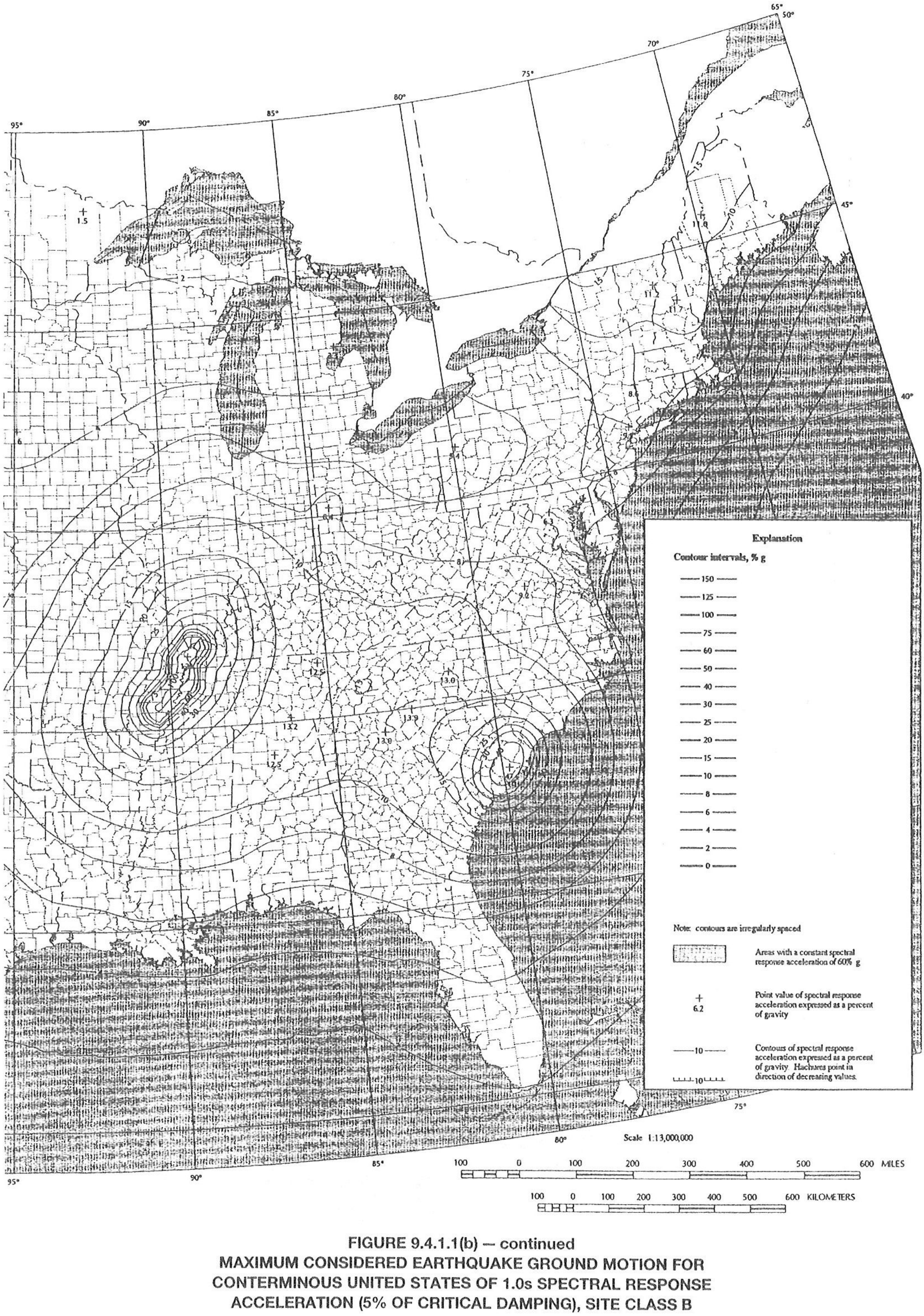
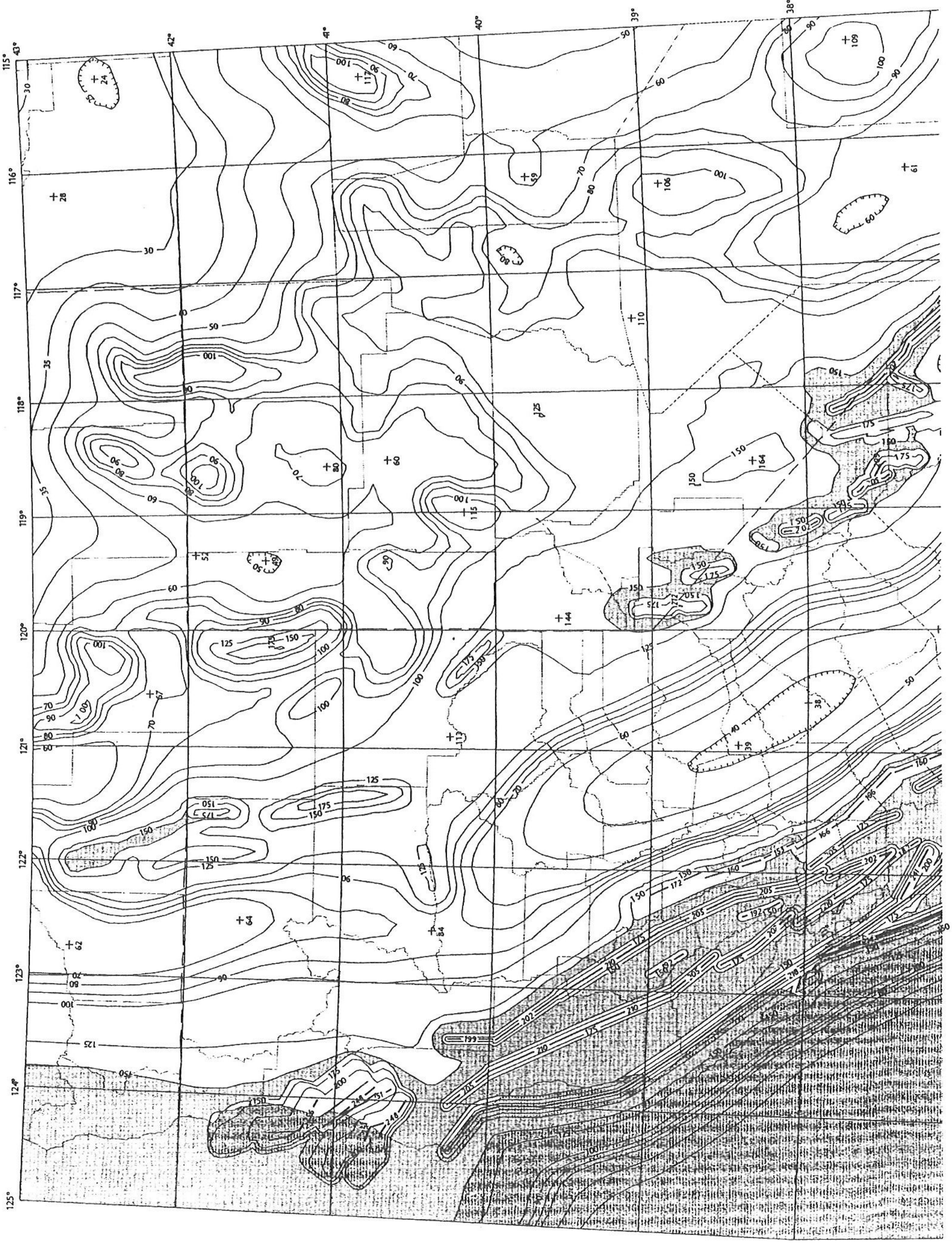


FIGURE 9.4.1.1(b) — continued
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR CONTERMINOUS UNITED STATES OF 1.0s SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



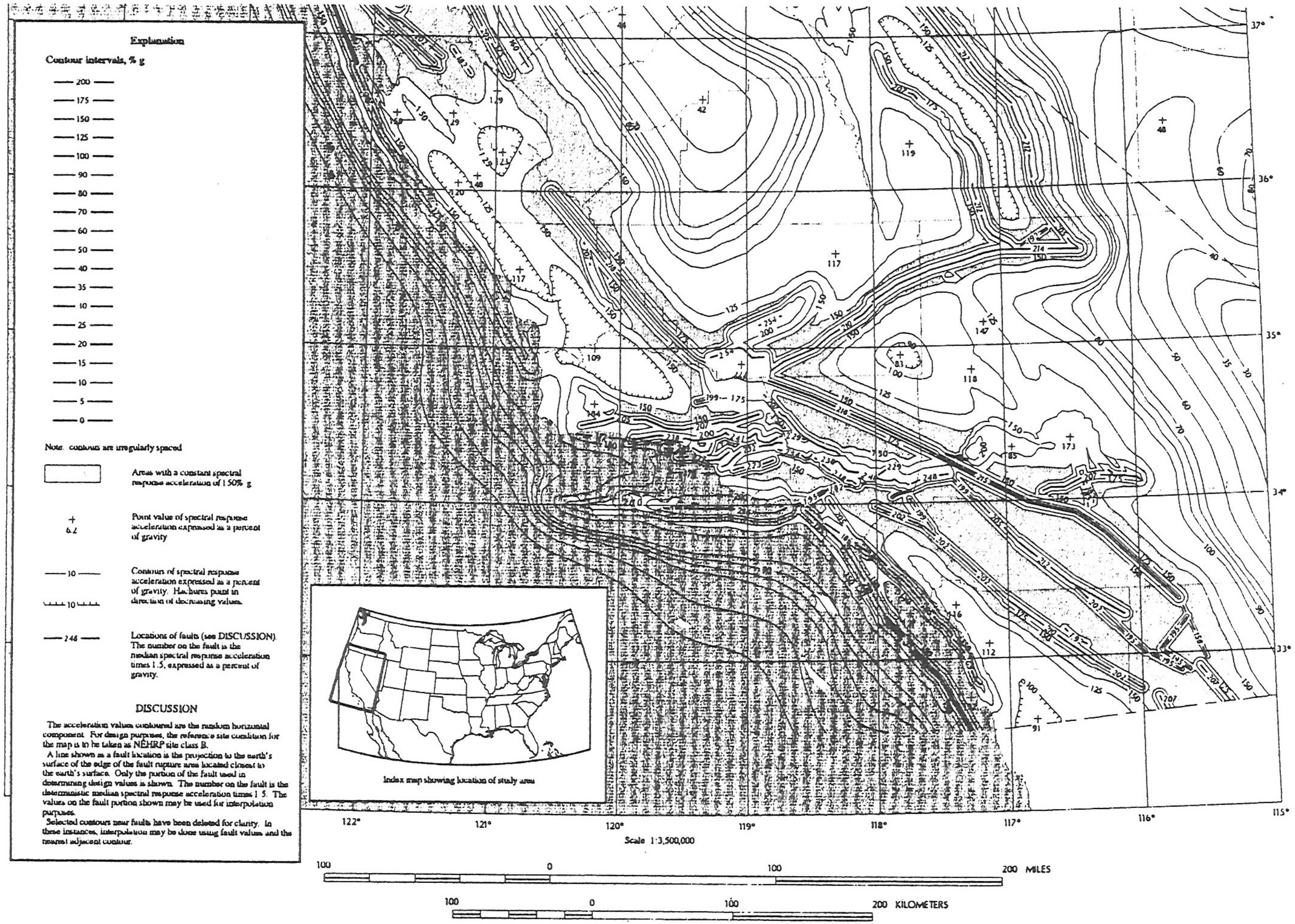
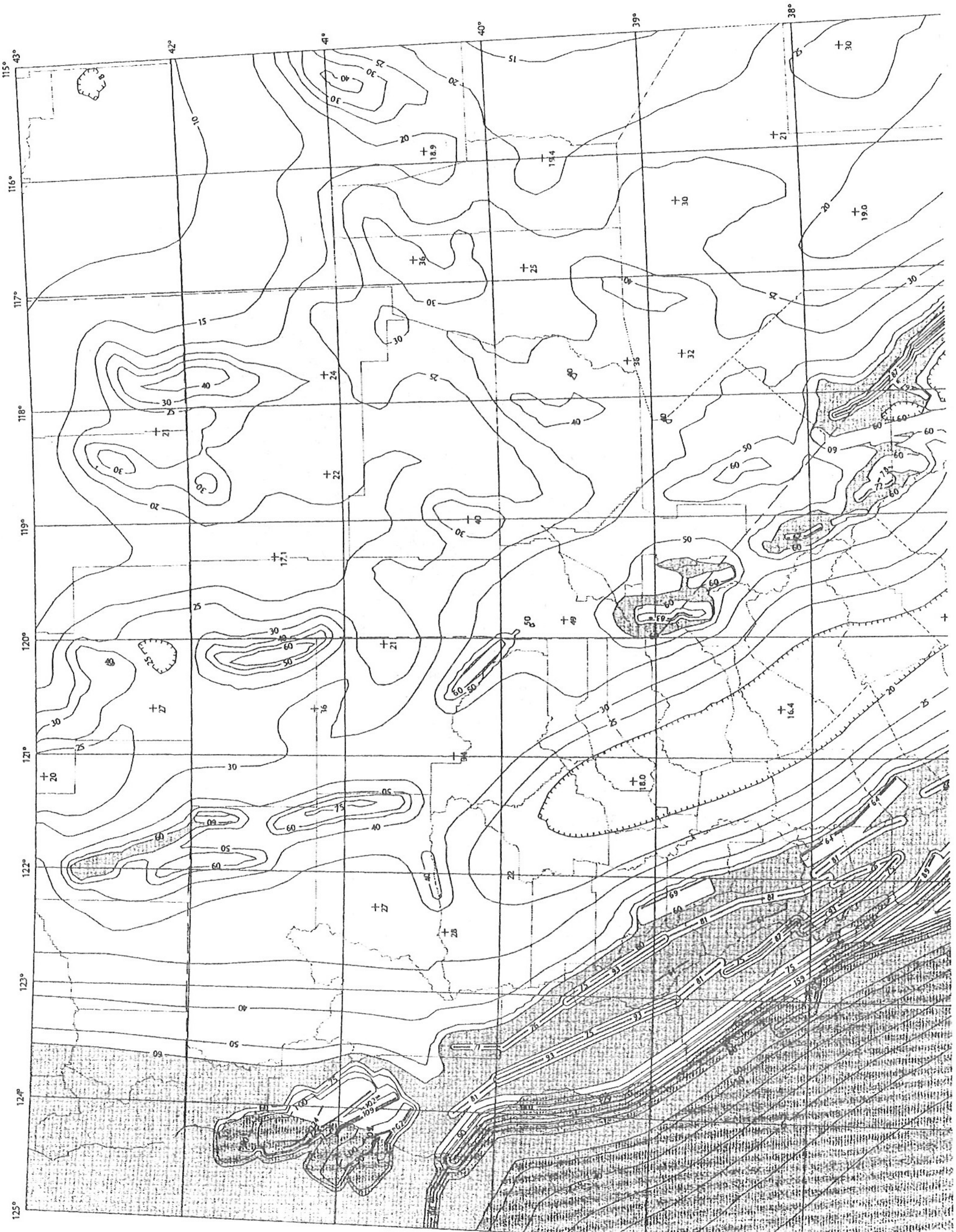


FIGURE 9.4.1.1(c) — continued
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 1 OF 0.2 s SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B


43°
42°
41°
40°
39°
38°
115°
116°
117°
118°
119°
120°
121°
122°
123°
124°
125°

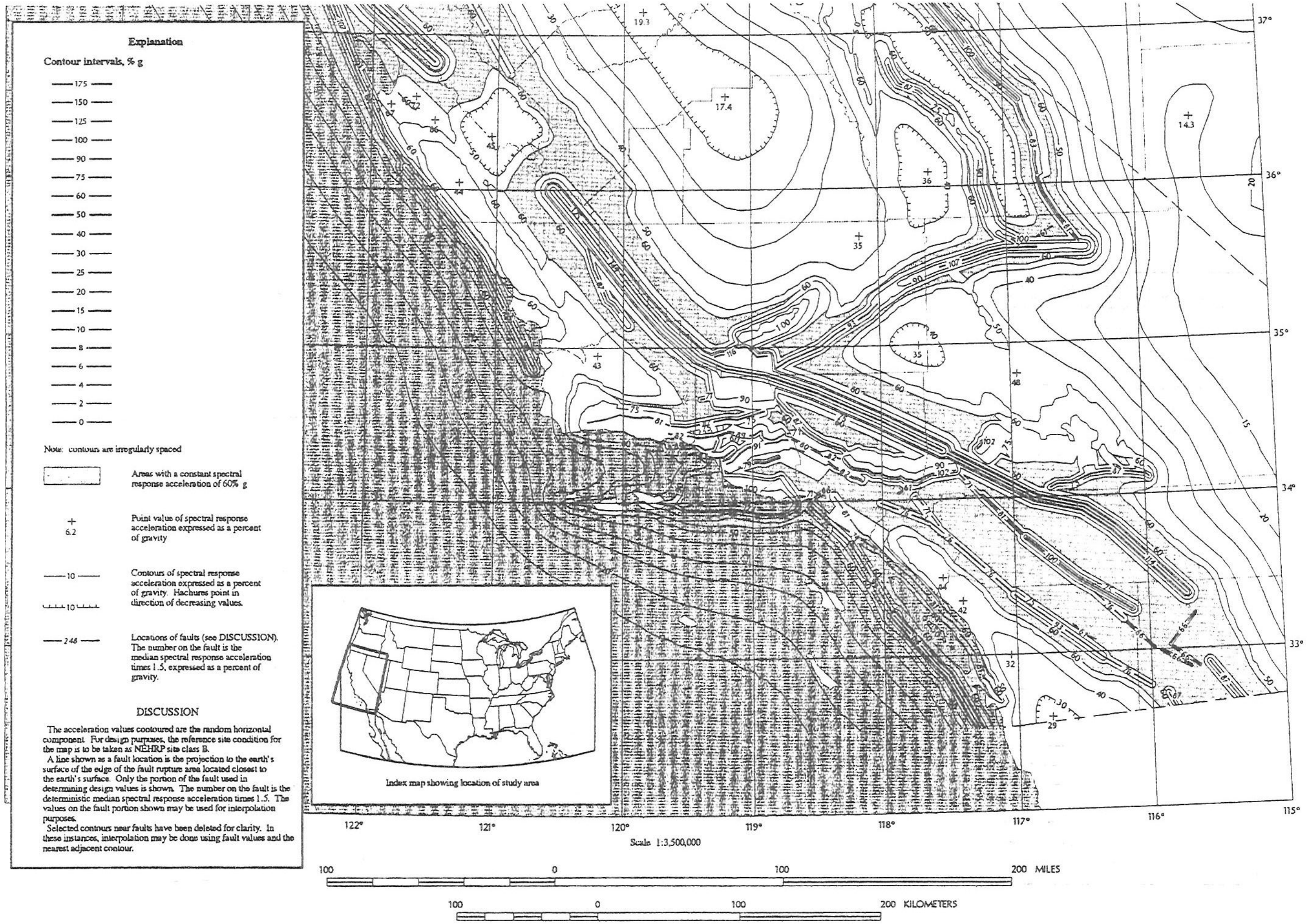
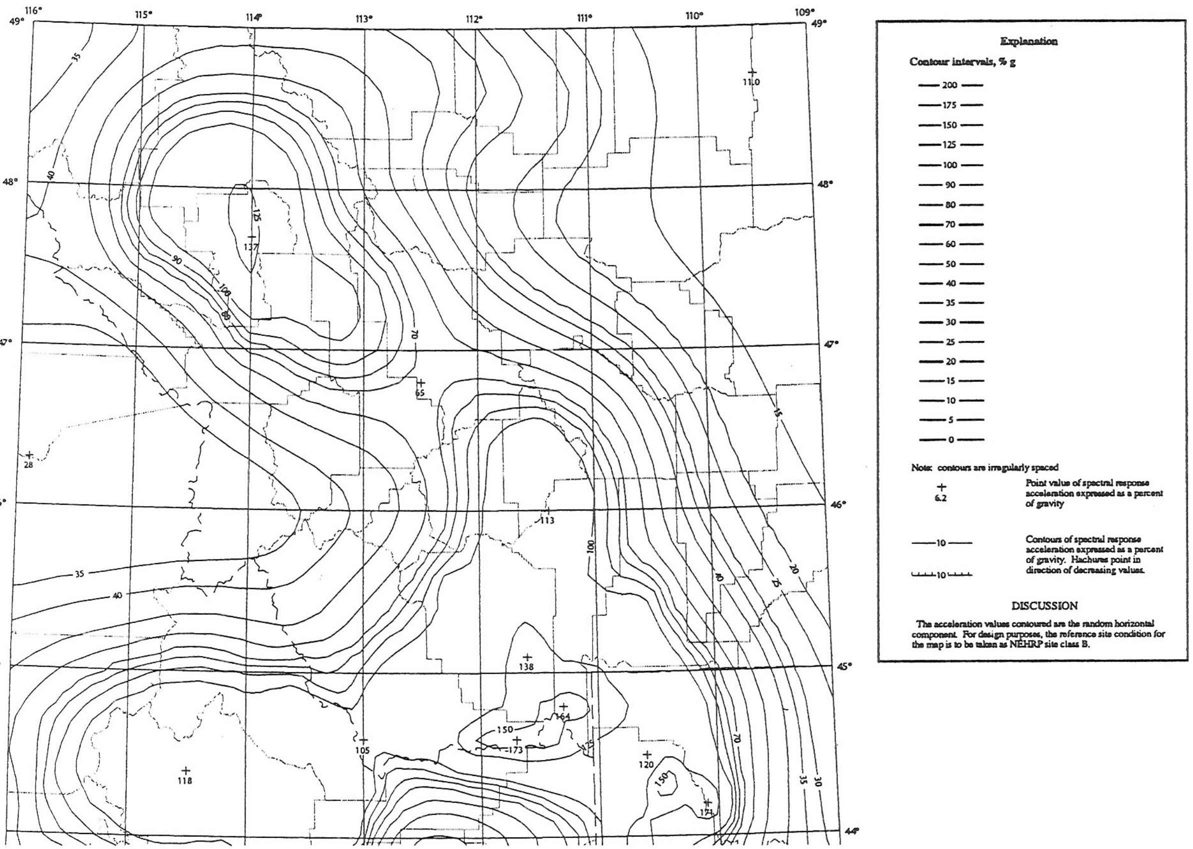


FIGURE 9.4.1.1(d) — continued
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 1 OF 1.0 s SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



116°
115°
114°
113°
112°
111°
110°
109°
49°
48°
47°
46°
45°
44°
Explanation
Contour intervals, % g
200
175
150
125
100
90
80
70
60
50
40
35
30
25
20
15
10
5
0
Note: contours are irregularly spaced
6.2
Point value of spectral response acceleration expressed as a percent of gravity
10
Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.
DISCUSSION
The acceleration values contoured are the random horizontal component. For design purposes, the reference site condition for the map is to be taken as NEHRP site class B.

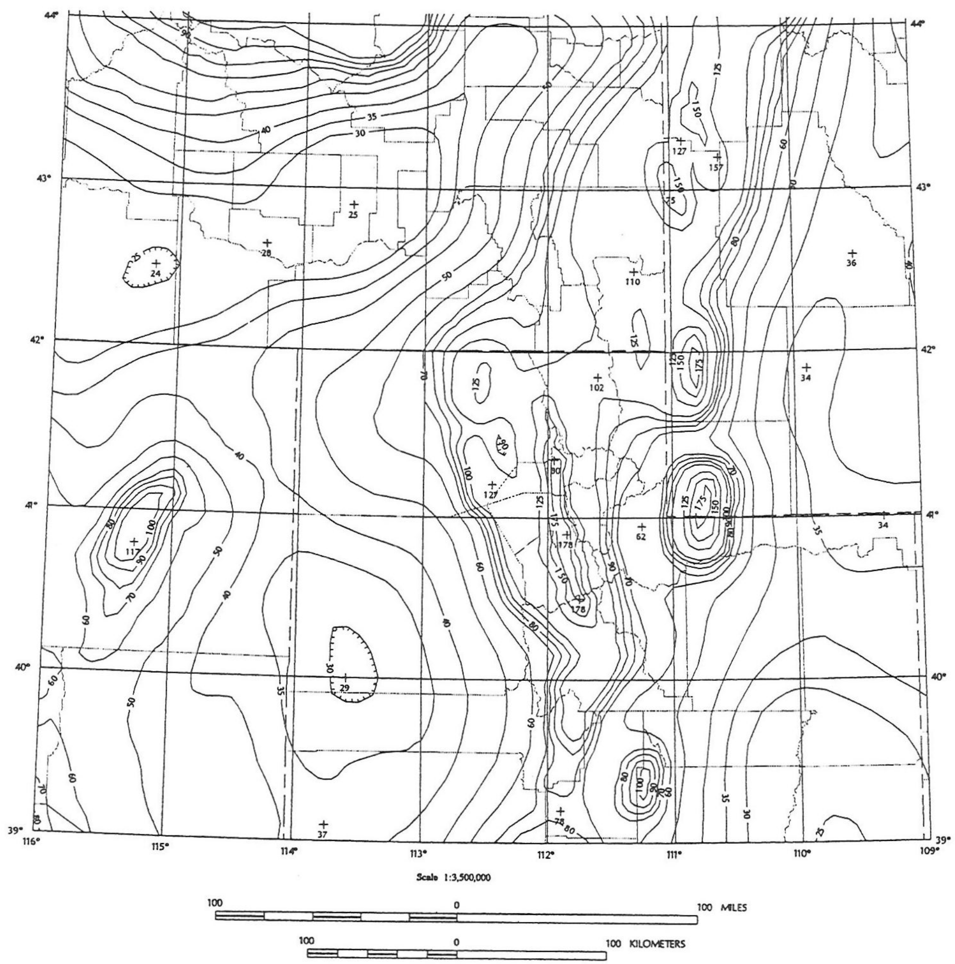
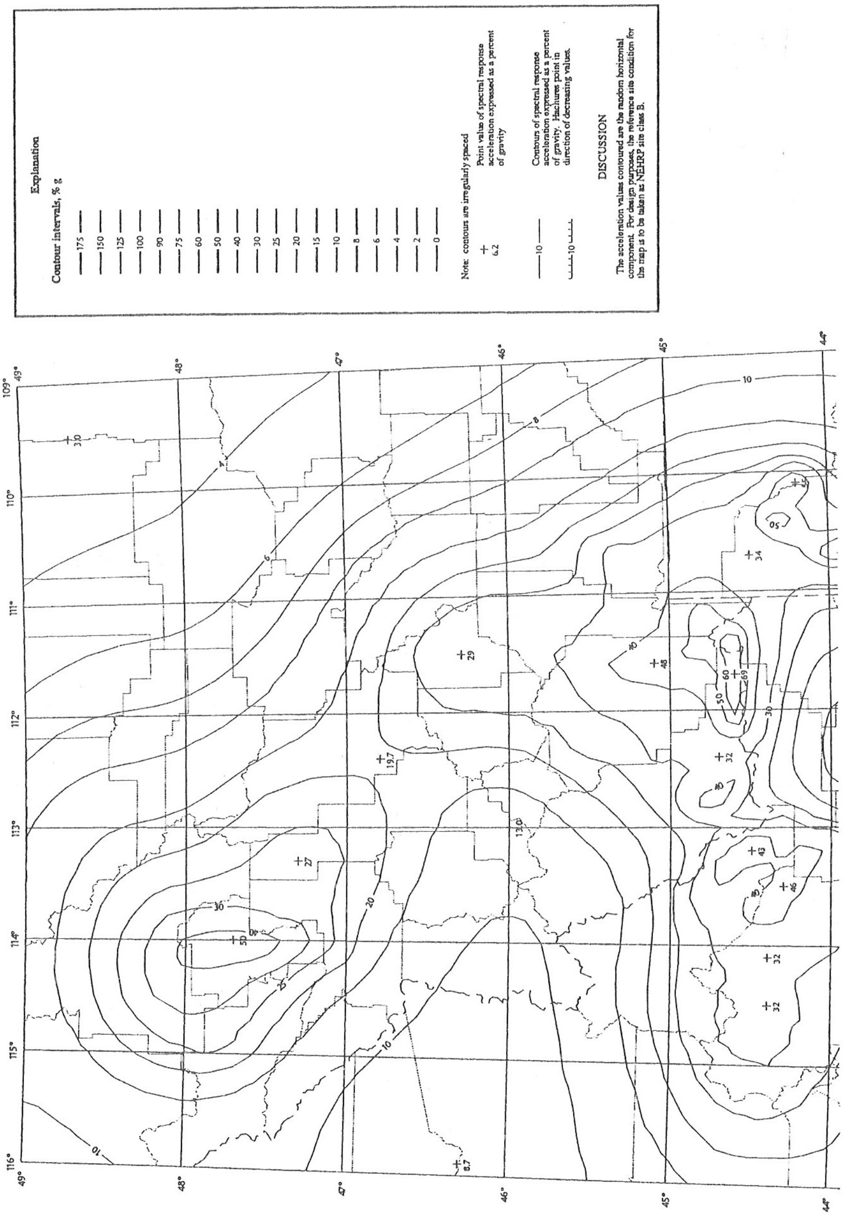


FIGURE 9.4.1.1(e) — continued
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 2 OF 0.2 s SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B


Explanation
Contour intervals, % g
175
150
125
100
90
75
60
50
40
30
25
20
15
10
8
6
4
2
0
Note: contours are irregularly spaced
6.2
Point value of spectral response acceleration expressed as a percent of gravity
10
Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.
DISCUSSION
The acceleration values contoured are the random horizontal component. For design purposes, the reference site condition for the map is to be taken as NEHRP site class B.

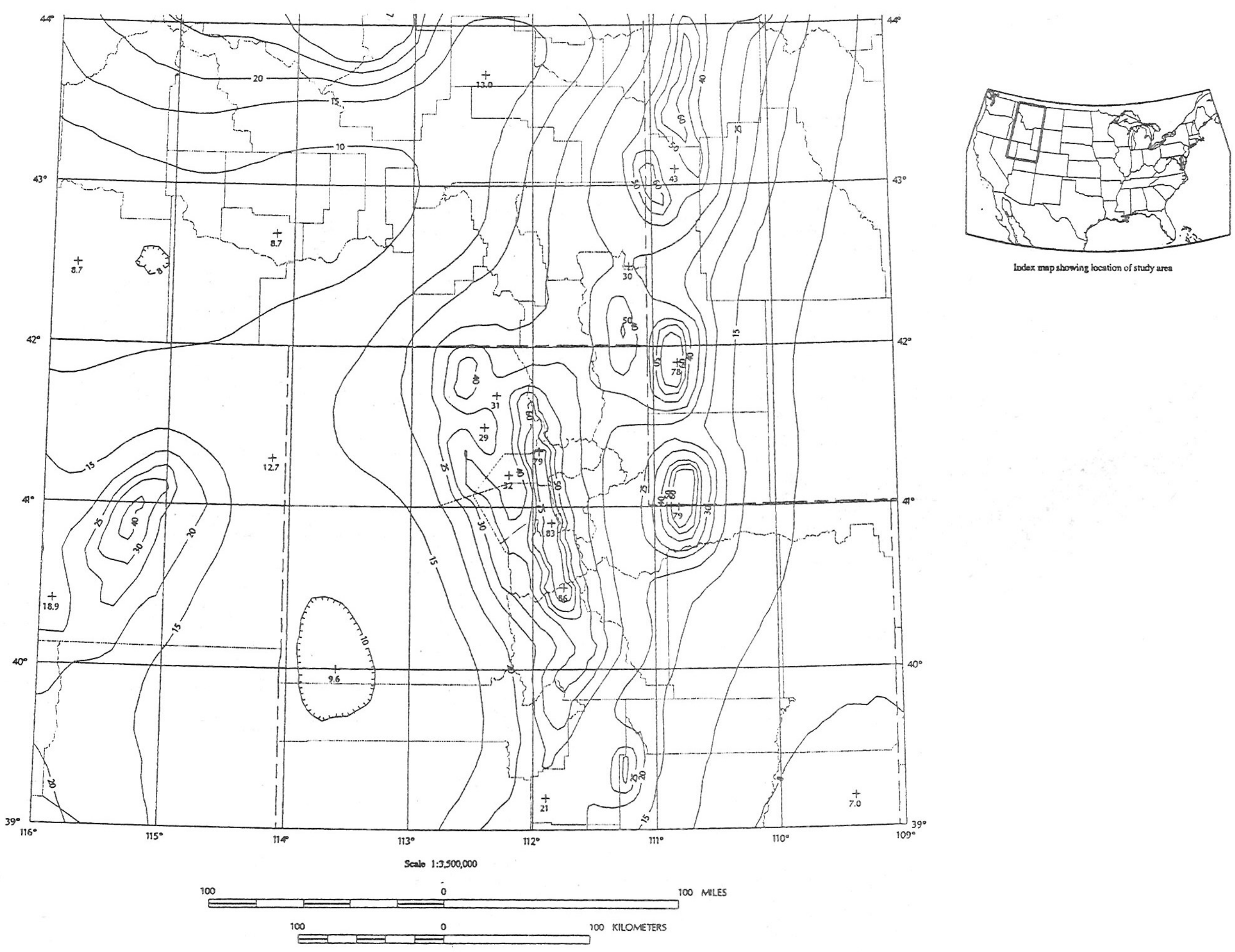


FIGURE 9.4.1.1(f) — continued
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 2 OF 1.0 s SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

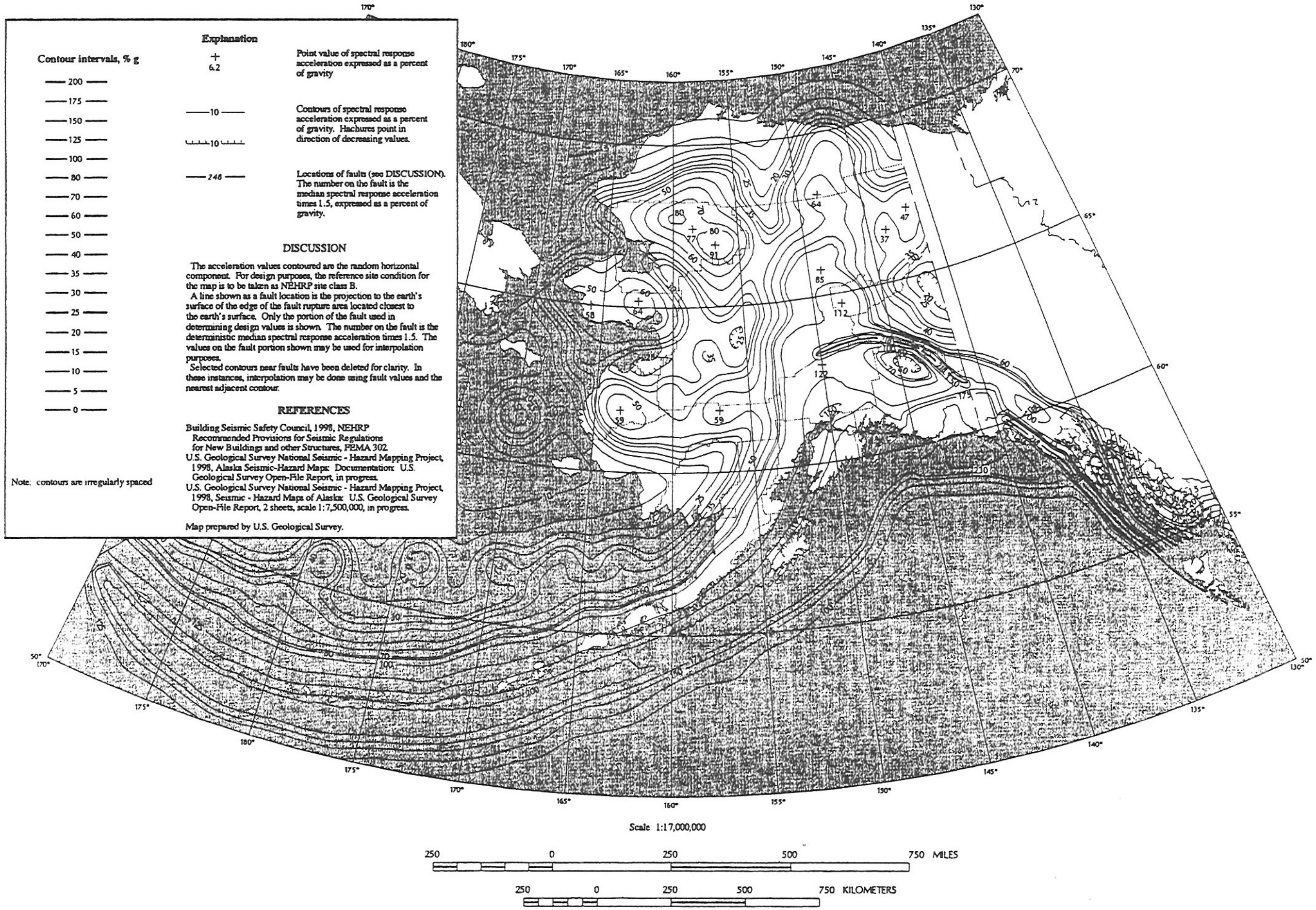


FIGURE 9.4.1.1(g)
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR ALASKA OF 0.2 s SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

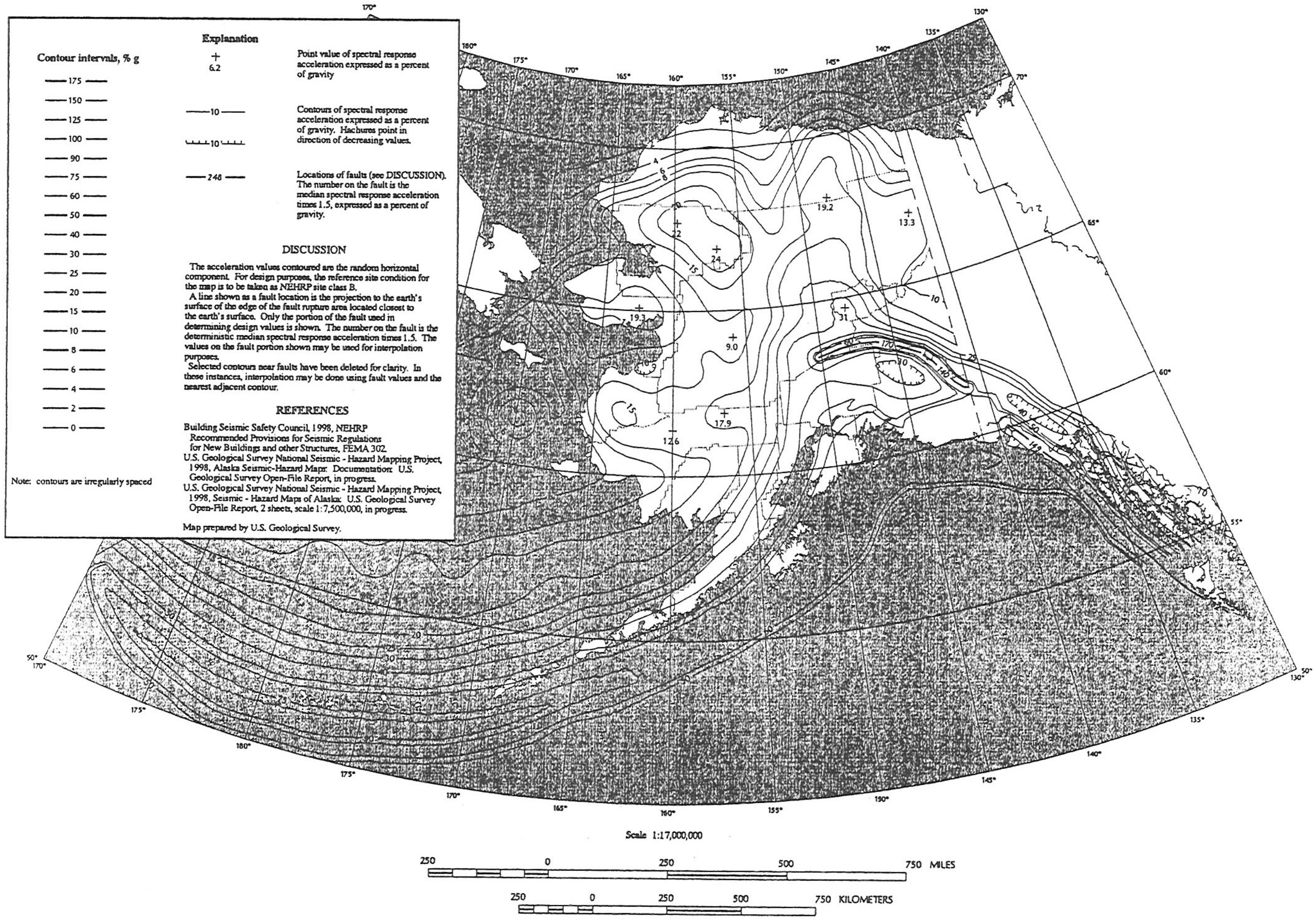


FIGURE 9.4.1.1(g) — continued
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR ALASKA OF 1.0 s SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

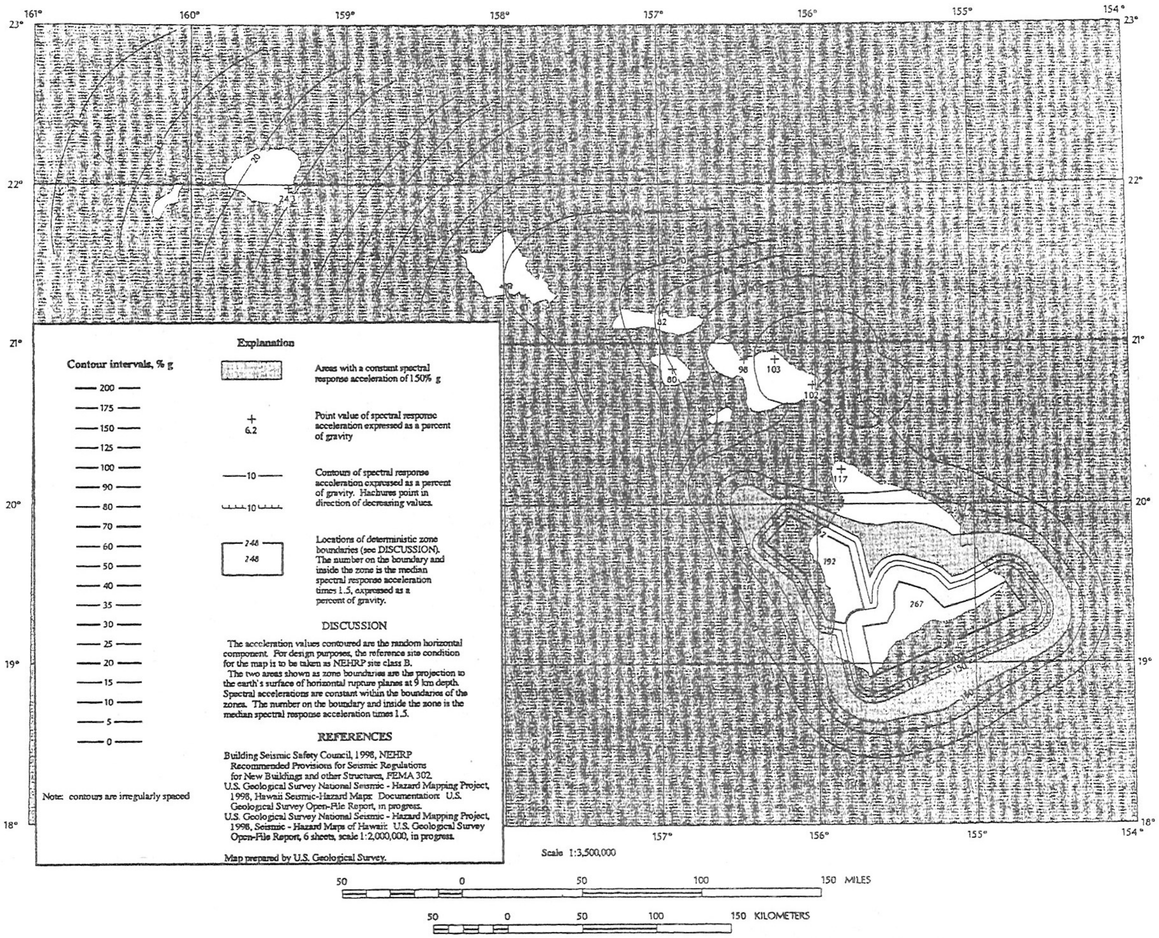


FIGURE 9.4.1.1(h)
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR HAWAII OF 0.2 s SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

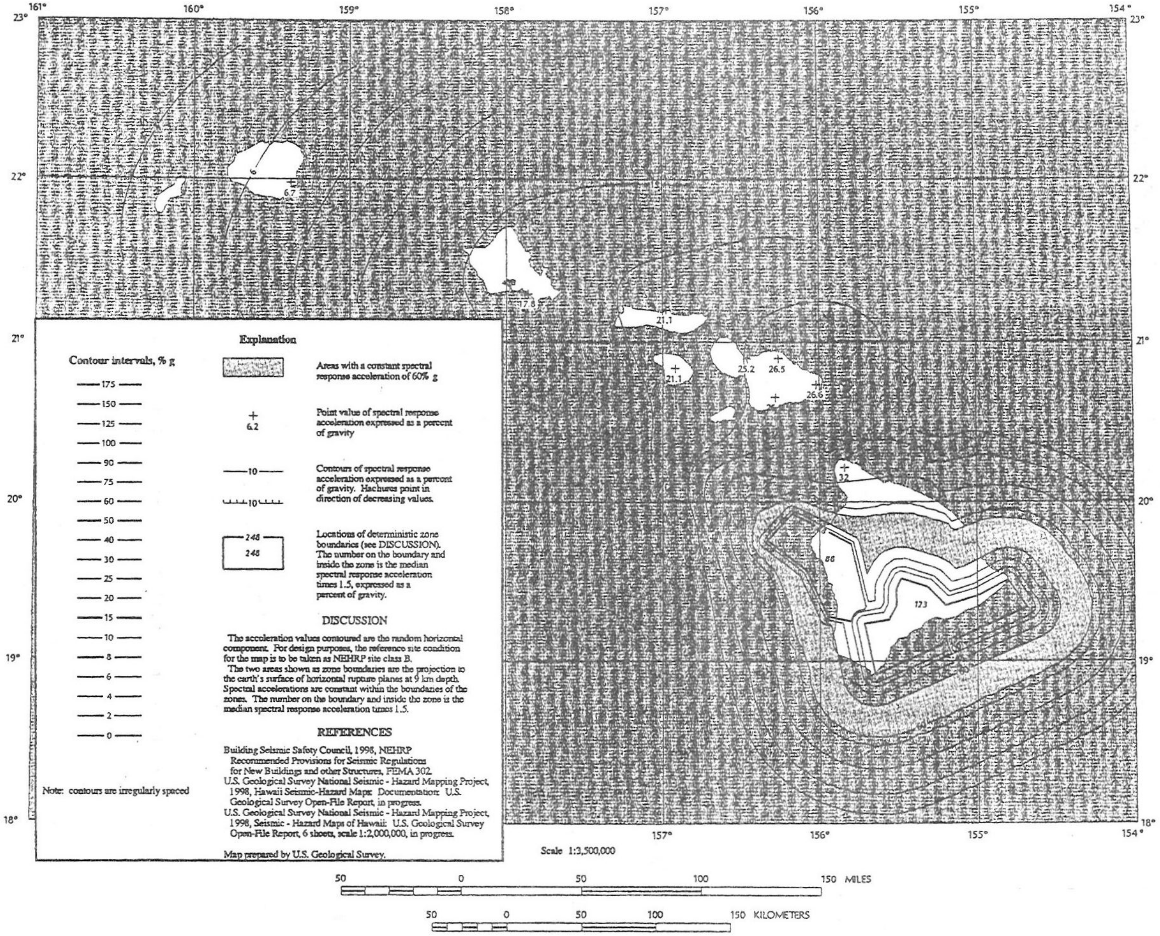
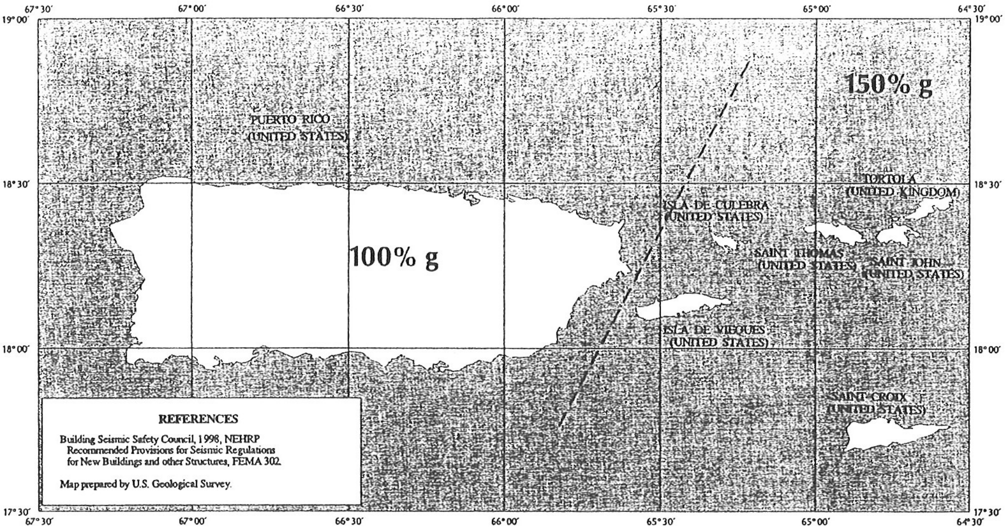
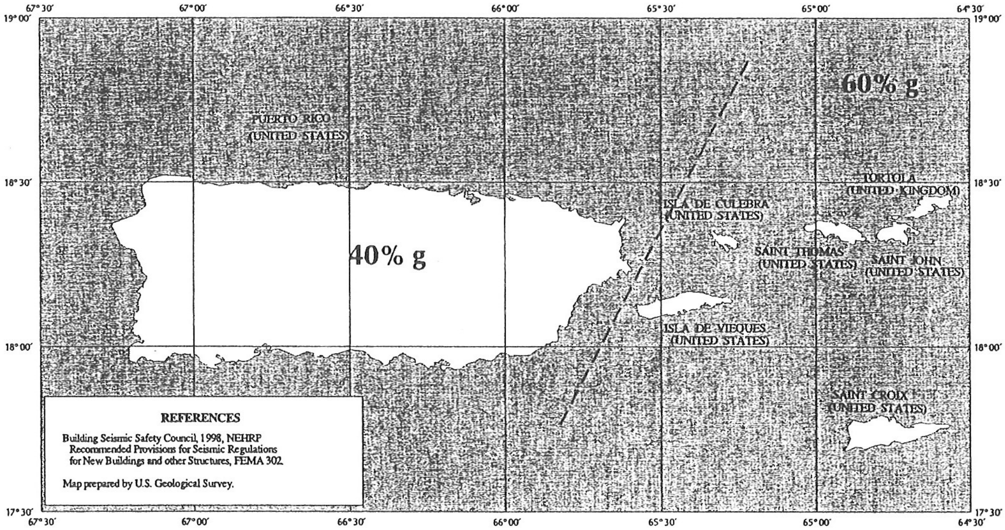


FIGURE 9.4.1.1(h) — continued
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR HAWAII OF 1.0 s SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)



1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)

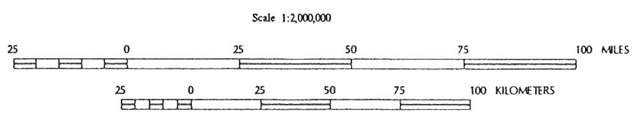
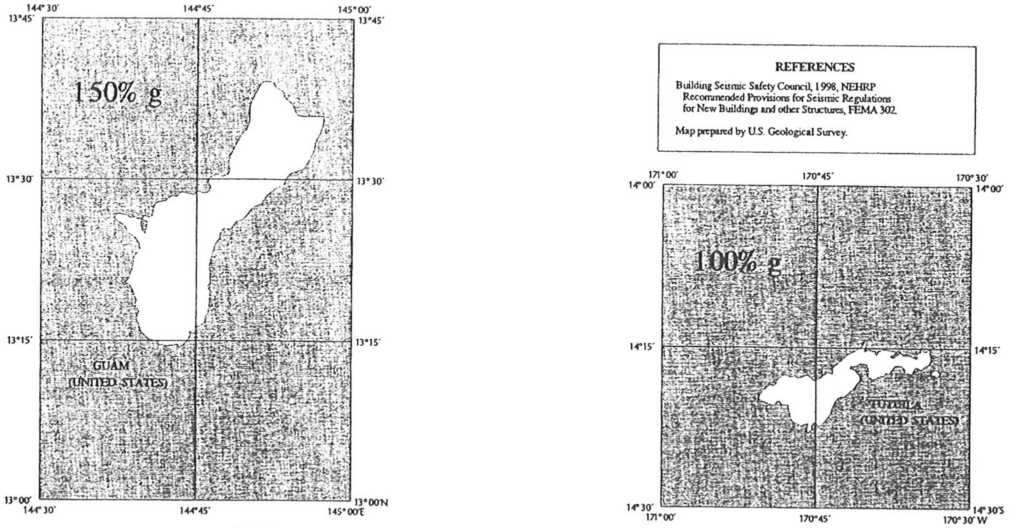
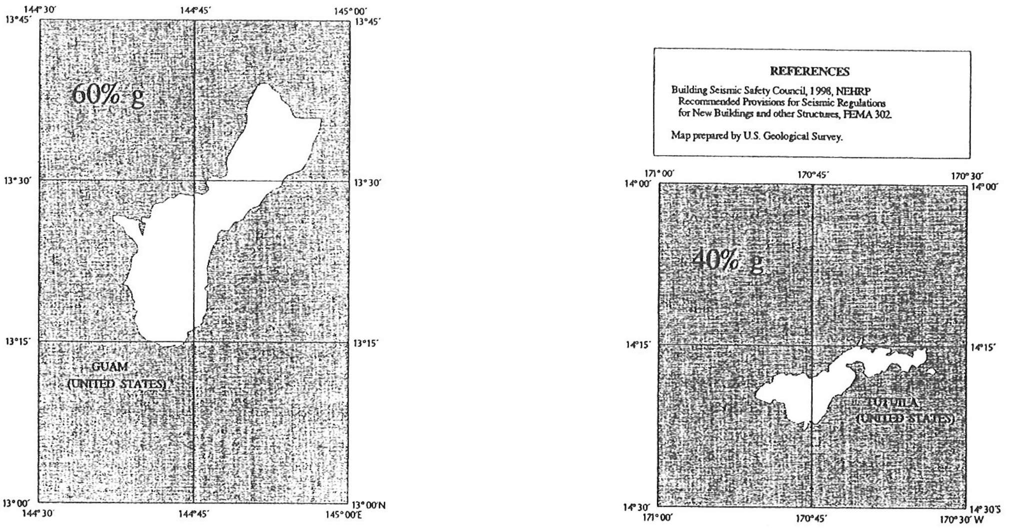


FIGURE 9.4.1.1(i)
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR PUERTO RICO, CULEBRA, VIEQUES, ST. THOMAS, ST. JOHN, AND ST. CROIX OF 0.2 AND 1.0 s SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)



1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)

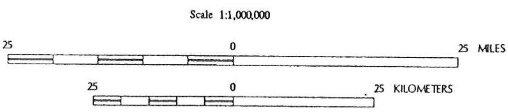


FIGURE 9.4.1.1(j)
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR GUAM AND TUTUILLA OF 0.2 AND 1.0 S SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

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

Step 1: Check for the four categories of Site Class F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific evaluation.

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

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

a. The $\overline{v}_s$ method: Determine $\overline{v}_s$ for the top 100 ft (30 m) of soil. Compare the value of $\overline{v}_s$ with those given in Section 9.4.1.2 and Table 9.4.1.2 and assign the corresponding Site Class.
$\overline{v}_s$ for rock, Site Class B, shall be measured on-site or estimated by a geotechnical engineer or engineering geologist/seismologist for competent rock with moderate fracturing and weathering. $\overline{v}_s$ for softer and more highly fractured and weathered rock shall be measured on-site or shall be classified as Site Class C.
The classification of hard rock, Site Class A, shall be supported by on-site measurements of $\overline{v}_s$ or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of at least 100 ft (30 m), surficial measurements of v_s are not prohibited from being extrapolated to assess $\overline{v}_s$.
The rock categories, Site Classes A and B, shall not be assigned to a site if there is more than 10 ft (3 m) of soil between the rock surface and the bottom of the spread footing or mat foundation.

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

c. The $\overline{s}_u$ method: For cohesive soil layers, determine $\overline{s}_u$ for the top 100 ft (30 m) of soil. For cohesionless soil layers, determine $\overline{N}_{ch}$ for the top 100 ft (30 m) of soil. Cohesionless soil is defined by a $PI < 20$ where cohesive soil is defined by a $PI > 20$. Compare the values of $\overline{s}_u$ and $\overline{N}_{ch}$ with those given in Section 9.4.1.2 and Table 9.4.1.2 and assign the corresponding Site Class. When the $\overline{N}_{ch}$ and $\overline{s}_u$ criteria differ, assign the category with the softer soil (Site Class E soil is softer than D).

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

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

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

$\overline{v}_s$ is

$$\overline{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \qquad \textbf{(Eq. 9.4.1.2-1)}$$

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

N_i is the standard penetration resistance, ASTM D1586-84 not to exceed 100 blows/ft as directly measured in the field without corrections.

$\overline{N}$ is:

$$\overline{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad \textbf{(Eq. 9.4.1.2-2)}$$

$\overline{N}_{ch}$ is:

$$\overline{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \qquad \textbf{(Eq. 9.4.1.2-3)}$$

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

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

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

$\bar{s}_u$ is

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \quad \textbf{(Eq. 9.4.1.2-4)}$$

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

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

PI is the plasticity index, ASTM D4318-93

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

9.4.1.2.4 Site Coefficients and Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameters. The maximum considered earthquake spectral response acceleration for short periods (S_{MS}) and at 1-sec (S_{M1}), adjusted for site class effects, shall be determined by Eqs. 9.4.1.2.4-1 and 9.4.1.2.4-2, respectively.

$$S_{MS} = F_a S_s \quad \textbf{(Eq. 9.4.1.2.4-1)}$$

$$S_{M1} = F_v S_1 \quad \textbf{(Eq. 9.4.1.2.4-2)}$$

where

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

S_S = the mapped maximum considered earthquake spectral response acceleration at short periods as determined in accordance with Section 9.4.1

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

9.4.1.2.5 Design Spectral Response Acceleration Parameters. Design earthquake spectral response acceleration at short periods, S_{DS}, and at 1-sec period, S_{D1}, shall be determined from Eqs. 9.4.1.2.5-1 and 9.4.1.2.5-2, respectively.

$$S_{DS} = \frac{2}{3} S_{MS} \quad \textbf{(Eq. 9.4.1.2.5-1)}$$

$$S_{D1} = \frac{2}{3} S_{M1} \quad \textbf{(Eq. 9.4.1.2.5-2)}$$

9.4.1.2.6 General Procedure Response Spectrum. Where a design response spectrum is required by these provisions and site-specific procedures are not used, the design response spectrum curve shall be developed as indicated in Figure 9.4.1.2.6 and as follows:

1. For periods less than or equal to T_0, the design spectral response acceleration, S_a, shall be taken as given by Eq. 9.4.1.2.6-1:

$$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right)$$

(Eq. 9.4.1.2.6-1)

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

	Mapped Maximum Considered Earthquake Spectral Response Acceleration at Short Periods				
Site Class	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*a*	*a*	*a*	*a*	*a*

Note: Use straight-line interpolation for intermediate values of S_S.

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

TABLE 9.4.1.2.4b
VALUES OF F_v AS A FUNCTION OF SITE CLASS AND MAPPED 1-SECOND PERIOD MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL ACCELERATION

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration at 1-Second Periods				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	*a*	*a*	*a*	*a*	*a*

Note: Use straight-line interpolation for intermediate values of S_1.
[a] Site-specific geotechnical investigation and dynamic site response analyses shall be performed except that for structures with periods of vibration equal to or less than 0.5-seconds, values of F_v for liquefiable soils may be assumed equal to the values for the site class determined without regard to liquefaction in Step 3 of Section 9.4.1.2.2.

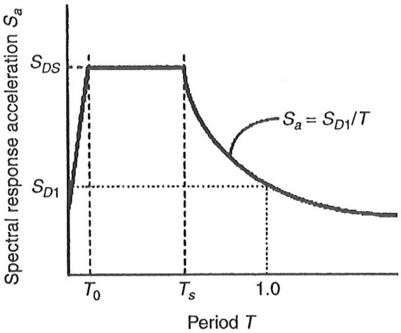


FIGURE 9.4.1.2.6
DESIGN RESPONSE SPECTRUM

2. For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response acceleration, S_a, shall be taken as equal to S_{DS}.
3. For periods greater than T_S, the design spectral response acceleration, S_a, shall be taken as given by Eq. 9.4.1.2.6-2:

$$S_a = \frac{S_{D1}}{T} \quad \textbf{(Eq. 9.4.1.2.6-2)}$$

where

S_{DS} = the design spectral response acceleration at short periods
S_{D1} = the design spectral response acceleration at 1-sec period, in units of g-sec
T = the fundamental period of the structure (sec)
$T_0 = 0.2S_{D1}/S_{DS}$ and
$T_S = S_{D1}/S_{DS}$.

9.4.1.3 Site-Specific Procedure for Determining Ground Motion Accelerations. A site-specific study shall account for the regional seismicity and geology, the expected recurrence rates and maximum magnitudes of events on known faults and source zones, the location of the site with respect to these near source effects, if any, and the characteristics of subsurface site conditions.

9.4.1.3.1 Probabilistic Maximum Considered Earthquake. When site-specific procedures are utilized, the maximum considered earthquake ground motion shall be taken as that motion represented by a 5% damped acceleration response spectrum having a 2% probability of exceedance within a 50-year period. The maximum considered earthquake spectral response acceleration at any period S_{aM} shall be taken from that spectrum.

Exception: Where the spectral response ordinates or a 5% damped spectrum having a 2% probability of exceedance within a 50-year period at periods of 1.0- or 0.2-sec exceed the corresponding ordinate of the deterministic limit of Section 9.4.1.3.2, the maximum considered earthquake ground motion shall be taken as the lesser of the probabilistic maximum considered earthquake ground motion or the deterministic maximum considered earthquake ground motion of Section 9.4.1.3.3 but shall not

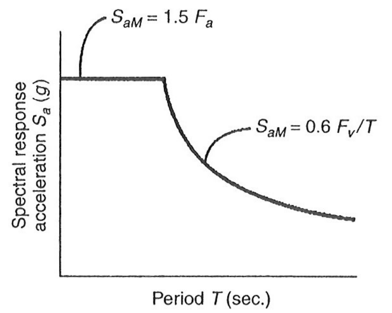


FIGURE 9.4.1.3.2
DETERMINISTIC LIMIT ON MAXIMUM CONSIDERED EARTHQUAKE RESPONSE SPECTRUM

be taken less than the deterministic limit ground motion of Section 9.4.1.3.2.

9.4.1.3.2 Deterministic Limit on Maximum Considered Earthquake Ground Motion. The deterministic limit on maximum considered earthquake ground motion shall be taken as the response spectrum determined in accordance with Figure 9.4.1.3.2, where F_a and F_v are determined in accordance with Section 9.4.1.2.4, with the value of S_S taken as 1.5 g and the value of S_1 taken as 0.6 g.

9.4.1.3.3 Deterministic Maximum Considered Earthquake Ground Motion. The deterministic maximum considered earthquake ground motion response spectrum shall be calculated as 150% of the median spectral response accelerations (S_{aM}) at all periods resulting from a characteristic earthquake on any known active fault within the region.

9.4.1.3.4 Site-Specific Design Ground Motion. Where site-specific procedures are used to determine the maximum considered earthquake ground motion response spectrum, the design spectral response acceleration at any period shall be determined from Eq. 9.4.1.3.4:

$$S_a = \frac{2}{3} S_{aM} \qquad \textbf{(Eq. 9.4.1.3.4)}$$

and shall be greater than or equal to 80% of the S_a determined by the general response spectrum in Section 9.4.1.2.6.

9.4.1.3.5 Design Acceleration Parameters. Where the site-specific procedure is used to determine the design ground motion in accordance with Section 9.4.1.3.4, the parameter S_{DS} shall be taken as the spectral acceleration, S_a, obtained from the site-specific spectra at a period of 0.2 sec, except that it shall not be taken as less than 90% of the peak spectral acceleration, S_a, at any period. The parameter S_{D1} shall be taken as the greater of the spectral acceleration, S_a, at a period of 1 sec or two times the spectral acceleration, S_a, at a period of 2 sec. The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively.

9.4.2 Seismic Design Category. Structures shall be assigned a Seismic Design Category in accordance with Section 9.4.2.1.

9.4.2.1 Determination of Seismic Design Category. All structures shall be assigned to a Seismic Design Category based on their Seismic Use Group and the design spectral response acceleration coefficients, S_{DS} and S_{D1}, determined in accordance with Section 9.4.1.2.5. Each building and structure shall be assigned to the most severe Seismic Design Category in accordance with Table 9.4.2.1a or 9.4.2.1b, irrespective of the fundamental period of vibration of the structure, T.

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

Exception: Detached one- and two-family dwellings of light-frame construction.

9.4.3 Quality Assurance. The performance required of structures in Seismic Design Categories C, D, E, or F requires that special attention be paid to quality assurance during construction. Refer to A.9.3 for supplementary provisions.

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

	Seismic Use Group		
Value of S_{DS}	I	II	III
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D[a]	D[a]	D[a]

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

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

Value of S_{D1}	Seismic Use Group		
	I	II	III
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D[a]	D[a]	D[a]

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

SECTION 9.5 STRUCTURAL DESIGN CRITERIA, ANALYSIS, AND PROCEDURES

9.5.1 This Section Has Been Intentionally Left Blank.

9.5.2 Structural Design Requirements.

9.5.2.1 Design Basis. The seismic analysis and design procedures to be used in the design of structures and their components shall be as prescribed in this Section. The structure shall include complete lateral and vertical force-resisting systems capable of providing adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand. The design ground motions shall be assumed to occur along any horizontal direction of a structure. The adequacy of the structural systems shall be demonstrated through the construction of a mathematical model and evaluation of this model for the effects of design ground motions. The design seismic forces, and their distribution over the height of the structure, shall be established in accordance with one of the applicable procedures indicated in Section 9.5.2.5 and the corresponding internal forces and deformations in the members of the structure shall be determined. An approved alternative procedure shall not be used to establish the seismic forces and their distribution unless the corresponding internal forces and deformations in the members are determined using a model consistent with the procedure adopted.

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

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

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

9.5.2.2 Basic Seismic Force-Resisting Systems. The basic lateral and vertical seismic force-resisting system shall conform to one of the types indicated in Table 9.5.2.2. Each type is subdivided by the types of vertical element used to resist lateral seismic forces. The structural system used shall be in accordance with the Seismic Design Category and height limitations indicated in Table 9.5.2.2. The appropriate response modification, coefficient, R, system overstrength factor, Ω_0, and the deflection amplification factor (C_d) indicated in Table 9.5.2.2 shall be used in determining the base shear, element design forces, and design story drift. Special framing requirements are indicated in Section 9.5.2.6 and Sections 9.8, 9.9, 9.10, 9.11, and 9.12 for structures assigned to the various Seismic Design Categories.

Seismic force-resisting systems that are not contained in Table 9.5.2.2 shall be permitted if analytical and test data are submitted that establish the dynamic characteristics and demonstrate the lateral force resistance and energy dissipation capacity to be equivalent to the structural systems listed in Table 9.5.2.2 for equivalent response modification coefficient, R, system overstrength coefficient, Ω_0, and deflection amplification factor, C_d, values.

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

9.5.2.2.2 Combinations of Framing Systems. Different seismic force-resisting systems are permitted along the two orthogonal axes of the structure. Combinations of seismic force-resisting systems shall comply with the requirements of this Section.

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

Basic Seismic Force-Resisting System	Response Modification Coefficient, R^a	System Over-strength Factor, $W_0{}^g$	Deflection Amplification Factor, $C_d{}^b$	Structural System Limitations and Building Height (ft) Limitations[c] — Seismic Design Category: A&B	C	D[d]	E[e]	F[e]
Bearing Wall Systems								
Ordinary steel concentrically braced frames	4	2	$3\frac{1}{2}$	NL	NL	35[k]	35[k]	NP[k]
Special reinforced concrete shear walls	5	$2\frac{1}{2}$	5	NL	NL	160	160	100
Ordinary reinforced concrete shear walls	4	$2\frac{1}{2}$	4	NL	NL	NP	NP	NP
Detailed plain concrete shear walls	$2\frac{1}{2}$	$2\frac{1}{2}$	2	NL	NP	NP	NP	NP
Ordinary plain concrete shear walls	$1\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{1}{2}$	NL	NP	NP	NP	NP
Special reinforced masonry shear walls	5	$2\frac{1}{2}$	$3\frac{1}{2}$	NL	NL	160	160	100
Intermediate reinforced masonry shear walls	$3\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{4}$	NL	NL	NP	NP	NP
Ordinary reinforced masonry shear walls	2	$2\frac{1}{2}$	$1\frac{3}{4}$	NL	160	NP	NP	NP
Detailed plain masonry shear walls	2	$2\frac{1}{2}$	$1\frac{3}{4}$	NL	NP	NP	NP	NP
Ordinary plain masonry shear walls	$1\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{1}{4}$	NL	NP	NP	NP	NP
Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	6	3	4	NL	NL	65	65	65
Light-framed walls with shear panels of all other materials	2	$2\frac{1}{2}$	2	NL	NL	35	NP	NP
Light-framed wall systems using flat strap bracing	4	2	$3\frac{1}{2}$	NL	NL	65	65	65
Building Frame Systems								
Steel eccentrically braced frames, moment resisting, connections at columns away from links	8	2	4	NL	NL	160	160	100
Steel eccentrically braced frames, non-moment resisting, connections at columns away from links	7	2	4	NL	NL	160	160	100
Special steel concentrically braced frames	6	2	5	NL	NL	160	160	100
Ordinary steel concentrically braced frames	5	2	$4\frac{1}{2}$	NL	NL	35[k]	35[k]	NP[k]
Special reinforced concrete shear walls	6	$2\frac{1}{2}$	5	NL	NL	160	160	100
Ordinary reinforced concrete shear walls	5	$2\frac{1}{2}$	$4\frac{1}{2}$	NL	NL	NP	NP	NP
Detailed plain concrete shear walls	3	$2\frac{1}{2}$	$2\frac{1}{2}$	NL	NP	NP	NP	NP
Ordinary plain concrete shear walls	2	$2\frac{1}{2}$	2	NL	NP	NP	NP	NP
Composite eccentrically braced frames	8	2	4	NL	NL	160	160	100
Composite concentrically braced frames	5	2	$4\frac{1}{2}$	NL	NL	160	160	100
Ordinary composite braced frames	3	2	3	NL	NL	NP	NP	NP
Composite steel plate shear walls	$6\frac{1}{2}$	$2\frac{1}{2}$	$5\frac{1}{2}$	NL	NL	160	160	100
Special composite reinforced concrete shear walls with steel elements	6	$2\frac{1}{2}$	5	NL	NL	160	160	100
Ordinary composite reinforced concrete shear walls with steel elements	5	$2\frac{1}{2}$	$4\frac{1}{4}$	NL	NL	NP	NP	NP
Special reinforced masonry shear walls	$5\frac{1}{2}$	$2\frac{1}{2}$	4	NL	NL	160	160	100
Intermediate reinforced masonry shear walls	4	$2\frac{1}{2}$	4	NL	NL	NP	NP	NP
Ordinary reinforced masonry shear walls	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{4}$	NL	160	NP	NP	NP

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

Basic Seismic Force-Resisting System	Response Modification Coefficient, R^a	System Over-strength Factor, W_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations and Building Height (ft) Limitations[c] — Seismic Design Category: A&B	C	D^d	E^e	F^e
Detailed plain masonry shear walls	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{4}$	NL	160	NP	NP	NP
Ordinary plain masonry shear walls	$1\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{1}{4}$	NL	NP	NP	NP	NP
							65	
Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	$6\frac{1}{2}$	$2\frac{1}{2}$	$4\frac{1}{4}$	NL	NL	65	65	65
Light-framed walls with shear panels of all other materials	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{2}$	NL	NL	35	NP	NP
Moment Resisting Frame Systems								
Special steel moment frames	8	3	$5\frac{1}{2}$	NL	NL	NL	NL	NL
Special steel truss moment frames	7	3	$5\frac{1}{2}$	NL	NL	160	100	NP
Intermediate steel moment frames	4.5	3	4	NL	NL	35[h]	NP[h, i]	NP[h, i]
Ordinary steel moment frames	3.5	3	3	NL	NL	NP[h, i]	NP[h, i]	NP[h, i]
Special reinforced concrete moment frames	8	3	$5\frac{1}{2}$	NL	NL	NL	NL	NL
Intermediate reinforced concrete moment frames	5	3	$4\frac{1}{2}$	NL	NL	NP	NP	NP
Ordinary reinforced concrete moment frames	3	3	$2\frac{1}{2}$	NL	NP	NP	NP	NP
Special composite moment frames	8	3	$5\frac{1}{2}$	NL	NL	NL	NL	NL
Intermediate composite moment frames	5	3	$4\frac{1}{2}$	NL	NL	NP	NP	NP
Composite partially restrained moment frames	6	3	$5\frac{1}{2}$	160	160	100	NP	NP
Ordinary composite moment frames	3	3	$2\frac{1}{2}$	NL	NP	NP	NP	NP
Special masonry moment frames	$5\frac{1}{2}$	3	5	NL	NL	160	160	100
Dual Systems with Special Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces								
Steel eccentrically braced frames, moment resisting connections, at columns away from links	8	$2\frac{1}{2}$	4	NL	NL	NL	NL	NL
Steel eccentrically braced frames, non-moment resisting connections, at columns away from links	7	$2\frac{1}{2}$	4	NL	NL	NL	NL	NL
Special steel concentrically braced frames	8	$2\frac{1}{2}$	$6\frac{1}{2}$	NL	NL	NL	NL	NL
Special reinforced concrete shear walls	8	$2\frac{1}{2}$	$6\frac{1}{2}$	NL	NL	NL	NL	NL
Ordinary reinforced concrete shear walls	7	$2\frac{1}{2}$	6	NL	NL	NP	NP	NP
Composite eccentrically braced frames	8	$2\frac{1}{2}$	4	NL	NL	NL	NL	NL
Composite concentrically braced frames	6	$2\frac{1}{2}$	5	NL	NL	NL	NL	NL
Composite steel plate shear walls	8	$2\frac{1}{2}$	$6\frac{1}{2}$	NL	NL	NL	NL	NL
Special composite reinforced concrete shear walls with steel elements	8	$2\frac{1}{2}$	$6\frac{1}{2}$	NL	NL	NL	NL	NL

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

Basic Seismic Force-Resisting System	Response Modification Coefficient, R[a]	System Over-strength Factor, W_0[g]	Deflection Amplification Factor, C_d[b]	Structural System Limitations and Building Height (ft) Limitations[c] — Seismic Design Category: A&B	C	D[d]	E[e]	F[e]
Ordinary composite reinforced concrete shear walls with steel elements	7	$2\frac{1}{2}$	6	NL	NL	NP	NP	NP
Special reinforced masonry shear walls	7	3	$6\frac{1}{2}$	NL	NL	NL	NL	NL
Intermediate reinforced masonry shear walls	6	$2\frac{1}{2}$	5	NL	NL	NL	NL	NL
Ordinary steel concentrically braced frames	6	$2\frac{1}{2}$	5	NL	NL	NL	NL	NL
Dual Systems with Intermediate Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces								
Special steel concentrically braced frames[f]	$4\frac{1}{2}$	$2\frac{1}{2}$	$4\frac{1}{2}$	NL	NL	35	NP	NP[h, i]
Special reinforced concrete shear walls	6	$2\frac{1}{2}$	5	NL	NL	160	100	100
Ordinary reinforced masonry shear walls	3	3	$2\frac{1}{2}$	NL	160	NP	NP	NP
Intermediate reinforced masonry shear walls	5	3	$4\frac{1}{2}$	NL	NL	NP	NP	NP
Composite concentrically braced frames	5	$2\frac{1}{2}$	$4\frac{1}{2}$	NL	NL	160	100	NP
Ordinary composite braced frames	4	$2\frac{1}{2}$	3	NL	NL	NP	NP	NP
Ordinary composite reinforced concrete shear walls with steel elements	5	3	$4\frac{1}{2}$	NL	NL	NP	NP	NP
Ordinary steel concentrically braced frames	5	$2\frac{1}{2}$	$4\frac{1}{2}$	NL	NL	160	100	NP
Ordinary reinforced concrete shear walls	$5\frac{1}{2}$	$2\frac{1}{2}$	$4\frac{1}{2}$	NL	NL	NP	NP	NP
Inverted Pendulum Systems and Cantilevered Column Systems								
Special steel moment frames	$2\frac{1}{2}$	2	$2\frac{1}{2}$	NL	NL	NL	NL	NL
Ordinary steel moment frames	$1\frac{1}{4}$	2	$2\frac{1}{2}$	NL	NL	NP	NP	NP
Special reinforced concrete moment frames	$2\frac{1}{2}$	2	$1\frac{1}{4}$	NL	NL	NL	NL	NL
Structural Steel Systems Not Specifically Detailed for Seismic Resistance	3	3	3	NL	NL	NP	NP	NP

[a] Response modification coefficient, R, for use throughout the standard. Note R reduces forces to a strength level, not an allowable stress level.
[b] Deflection amplification factor, C_d, for use in Sections 9.5.3.7.1 and 9.5.3.7.2
[c] NL = Not Limited and NP = Not Permitted. For metric units use 30 m for 100 ft and use 50 m for 160 ft. Heights are measured from the base of the structure as defined in Section 9.2.1.
[d] See Section 9.5.2.2.4.1 for a description of building systems limited to buildings with a height of 240 ft (75 m) or less.
[e] See Sections 9.5.2.2.4 and 9.5.2.2.4.5 for building systems limited to buildings with a height of 160 ft (50 m) or less.
[f] Ordinary moment frame is permitted to be used in lieu of intermediate moment frame in Seismic Design Categories B and C.
[g] The tabulated value of the overstrength factor, W_0, may be reduced by subtracting $\frac{1}{2}$ for structures with flexible diaphragms but shall not be taken as less than 2.0 for any structure.
[h] Steel ordinary moment frames and intermediate moment frames are permitted in single-story buildings up to a height of 60 ft, when the moment joints of field connections are constructed of bolted end plates and the dead load of the roof does not exceed 15 psf.
[i] Steel ordinary moment frames are permitted in buildings up to a height of 35 ft where the dead load of the walls, floors, and roofs does not exceed 15 psf.
[k] Steel ordinary concentrically braced frames are permitted in single-story buildings up to a height of 60 ft when the dead load of the roof does not exceed 15 psf and in penthouse structures.

9.5.2.2.2.1 ***R*** **and** Ω_0 **Factors.** The response modification, coefficient, R, in the direction under consideration at any story shall not exceed the lowest response modification coefficient, R, for the seismic force-resisting system in the same direction considered above that story excluding penthouses. For other than dual systems, where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of R used in that direction shall not be greater than the least value of any of the systems utilized in the same direction. If a system other than a dual system with a response modification coefficient, R, with a value of less than 5 is used as part of the seismic force-resisting system in any direction of the structure, the lowest such value shall be used for the entire structure. The system overstrength factor, Ω_0, in the direction under consideration at any story shall not be less than the largest value of this factor for the seismic force-resisting system in the same direction considered above that story.

Exceptions:

1. The limit does not apply to supported structural systems with a weight equal to or less than 10% of the weight of the structure.
2. Detached one- and two-family dwellings of light-frame construction.

9.5.2.2.2.2 Combination Framing Detailing Requirements. The detailing requirements of Section 9.5.2.6 required by the higher response modification coefficient, R, shall be used for structural components common to systems having different response modification coefficients.

9.5.2.2.3 Seismic Design Categories B and C. The structural framing system for structures assigned to Seismic Design Categories B and C shall comply with the structure height and structural limitations in Table 9.5.2.2.

9.5.2.2.4 Seismic Design Categories D and E. The structural framing system for a structure assigned to Seismic Design Categories D and E shall comply with Section 9.5.2.2.3 and the additional provisions of this Section.

9.5.2.2.4.1 Increased Building Height Limit. The height limits in Table 9.5.2.2 are permitted to be increased to 240 ft (75 m) in buildings that have steel braced frames or concrete cast-in-place shear walls and that meet the requirements of this Section. In such buildings the braced frames or cast-in-place special reinforced concrete shear walls in any one plane shall resist no more than 60% of the total seismic forces in each direction, neglecting torsional effects. The seismic force in any braced frame or shear wall in any one plane resulting from torsional effects shall not exceed 20% of the total seismic force in that braced frame or shear wall.

9.5.2.2.4.2 Interaction Effects. Moment resisting frames that are enclosed or adjoined by more rigid elements not considered to be part of the seismic force-resisting system shall be designed so that the action or failure of those elements will not impair the vertical load and seismic force-resisting capability of the frame. The design shall provide for the effect of these rigid elements on the structural system at structure deformations corresponding to the design story drift (Δ) as determined in Section 9.5.3.7. In addition, the effects of these elements shall be considered when determining whether a structure has one or more of the irregularities defined in Section 9.5.2.3.

9.5.2.2.4.3 Deformational Compatibility. Every structural component not included in the seismic force-resisting system in the direction under consideration shall be designed to be adequate for the vertical load-carrying capacity and the induced moments and shears resulting from the design story drift (Δ) as determined in accordance with Section 9.5.3.7; see also Section 9.5.2.8.

Exception: Reinforced concrete frame members not designed as part of the seismic force-resisting system shall comply with Section 21.9 of Ref. 9.9-1.

When determining the moments and shears induced in components that are not included in the seismic force-resisting system in the direction under consideration, the stiffening effects of adjoining rigid structural and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used.

9.5.2.2.4.4 Special Moment Frames. A special moment frame that is used but not required by Table 9.5.2.2 shall not be discontinued and supported by a more rigid system with a lower response modification coefficient (R) unless the requirements of Sections 9.5.2.6.2.4 and 9.5.2.6.4.2 are met. Where a special moment frame is required by Table 9.5.2.2, the frame shall be continuous to the foundation.

9.5.2.2.5 Seismic Design Category F. The framing systems of structures assigned to Seismic Design Category F shall conform to the requirements of Section 9.5.2.2.4 for Seismic Design Categories D and E and to the additional requirements and limitations of this Section. The increased height limit of Section 9.5.2.2.4.1 for braced frame or shear wall systems shall be reduced from 240 ft (75 m) to 160 ft (50 m).

9.5.2.3 Structure Configuration. Structures shall be classified as regular or irregular based on the criteria in this Section. Such classification shall be based on the plan and vertical configuration.

9.5.2.3.1 Diaphragm Flexibility. A diaphragm shall be considered flexible for the purposes of distribution of story shear and torsional moment when the computed maximum in-plane deflection of the diaphragm itself under lateral load is more than two times the average drift of adjoining vertical elements of the lateral force-resisting system of the associated story under equivalent tributary lateral load. The loadings used for this calculation shall be those prescribed by Section 9.5.5.

9.5.2.3.2 Plan Irregularity. Structures having one or more of the irregularity types listed in Table 9.5.2.3.2 shall be designated as having plan structural irregularity. Such structures assigned to the Seismic Design Categories listed in Table 9.5.2.3.2 shall comply with the requirements in the sections referenced in that table.

9.5.2.3.3 Vertical Irregularity. Structures having one or more of the irregularity types listed in Table 9.5.2.3.3 shall be designated as having vertical irregularity. Such structures assigned to the Seismic

TABLE 9.5.2.3.2
PLAN STRUCTURAL IRREGULARITIES

	Irregularity Type and Description	Reference Section	Seismic Design Category Application
1a.	Torsional Irregularity Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	9.5.2.6.4.2 9.5.5.5.2	D, E, and F C, D, E, and F
1b.	Extreme Torsional Irregularity Extreme Torsional Irregularity is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	9.5.2.6.4.2 9.5.5.5.2 9.5.2.6.5.1	D C and D E and F
2.	Re-entrant Corners Plan configurations of a structure and its lateral force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15% of the plan dimension of the structure in the given direction.	9.5.2.6.4.2	D, E, and F
3.	Diaphragm Discontinuity Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one-story to the next.	9.5.2.6.4.2	D, E, and F
4.	Out-of-Plane Offsets Discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	9.5.2.6.4.2 9.5.2.6.2.11	D, E, and F B, C, D, E, or F
5.	Nonparallel Systems The vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force-resisting system.	9.5.2.6.3.1	C, D, E, and F

TABLE 9.5.2.3.3
VERTICAL STRUCTURAL IRREGULARITIES

	Irregularity Type and Description	Reference Section	Seismic Design Category Application
1a.	Stiffness Irregularity: Soft Story A soft story is one in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	9.5.2.5.1	D, E, and F
1b.	Stiffness Irregularity: Extreme Soft Story An extreme soft story is one in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	9.5.2.5.1 9.5.2.6.5.1	D, E and F
2.	Weight (Mass) Irregularity Mass irregularity shall be considered to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	9.5.2.5.1	D, E, and F
3.	Vertical Geometric Irregularity Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force-resisting system in any story is more than 130% of that in an adjacent story.	9.5.2.5.1	D, E, and F
4.	In-Plane Discontinuity in Vertical Lateral Force-Resisting Elements In-plane discontinuity in vertical lateral force-resisting elements shall be considered to exist where an in-plane offset of the lateral force-resisting elements is greater than the length of those elements or there exists a reduction in stiffness of the resisting element in the story below.	9.5.2.5.1 and 9.5.2.6.2.11	B, C, D, E, and F
5.	Discontinuity in Lateral Strength: Weak Story A weak story is one in which the story lateral strength is less than 80% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	9.5.2.6.2.2 9.5.2.5.3 9.5.2.6.5.1	B, C, D, E, and F D, E, and F E and F

Design Categories listed in Table 9.5.2.3.3 shall comply with the requirements in the sections referenced in that table.

Exceptions:

1. Vertical structural irregularities of Types 1a, 1b, or 2 in Table 9.5.2.3.3 do not apply where no story drift ratio under design lateral seismic force is greater than 130% of the story drift ratio of the next story above. Torsional effects need not be considered in the calculation of story drifts. The story drift ratio relationship for the top 2 stories of the structure are not required to be evaluated.
2. Irregularities Types 1a, 1b, and 2 of Table 9.5.2.3.3 are not required to be considered for 1-story buildings in any Seismic Design Category or for 2-story buildings in Seismic Design Categories A, B, C, or D.

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

9.5.2.4.1 Seismic Design Categories A, B, and C. For structures in Seismic Design Categories A, B, and C, the value of ρ is 1.0.

9.5.2.4.2 Seismic Design Category D. For structures in Seismic Design Category D, ρ shall be taken as the largest of the values of ρ_x calculated at each story "x" of the structure in accordance with Eq. 9.5.2.4.2-1 as follows:

$$\rho_x = 2 - \frac{20}{r_{\max_x}\sqrt{A_x}} \qquad \textbf{(Eq. 9.5.2.4.2-1)}$$

where

r_{max_x} = the ratio of the design story shear resisted by the single element carrying the most shear force in the story to the total story shear, for a given direction of loading. For braced frames, the value of r_{max_x} is equal to the lateral force component in the most heavily loaded brace element divided by the story shear. For moment frames, r_{max_x} shall be taken as the maximum of the sum of the shears in any two adjacent columns in the plane of a moment frame divided by the story shear. For columns common to two bays with moment resisting connections on opposite sides at the level under consideration, 70% of the shear in that column may be used in the column shear summation. For shear walls, r_{max_x} shall be taken equal to shear in the most heavily loaded wall or wall pier multiplied by $10/l_w$ (the metric coefficient is $3.3/l_w$) where l_w is the wall or wall pier length in ft (m) divided by the story shear and where the ratio $10/l_w$ need not be taken greater than 1.0 for buildings of light-frame construction. For dual systems, r_{max_x} shall be taken as the maximum value as defined above considering all lateral-load–resisting elements in the story. The lateral loads shall be distributed to elements based on relative rigidities considering the interaction of the dual system. For dual systems, the value of ρ need not exceed 80% of the value calculated above.

A_x = the floor area in ft^2 of the diaphragm level immediately above the story

The value of ρ need not exceed 1.5, which may be used for any structure. The value of ρ shall not be taken as less than 1.0.

Exception: For structures with lateral-force-resisting systems in any direction consisting solely of special moment frames, the lateral-force-resisting system shall be configured such that the value of ρ calculated in accordance with this Section does not exceed 1.25.

The metric equivalent of Eq. 9.5.2.4.2 is:

$$\rho_x = 2 - \frac{6.1}{r_{\max_x}\sqrt{A_x}} \qquad \textbf{(Eq. 9.5.2.4.2)}$$

where A_x is in m^2.

9.5.2.4.3 Seismic Design Categories E and F. For structures in Seismic Design Categories E and F, the value of r shall be calculated as indicated in Section 9.5.2.4.2, above.

Exception: For structures with lateral force-resisting systems in any direction consisting solely of special moment frames, the lateral-force-resisting system shall be configured such that the value of ρ calculated in accordance with Section 9.5.2.4.2 does not exceed 1.1.

9.5.2.5 Analysis Procedures. A structural analysis conforming to one of the types permitted in Section 9.5.2.5.1 shall be made for all structures. Application of loading shall be as indicated in Section 9.5.2.5.2.1, and as required by the selected analysis procedure. All members of the structure's seismic force-resisting system and their connections shall have adequate strength to resist the forces, Q_E, predicted by the analysis, in combination with other loads, as required by Section 9.5.2.7. Drifts predicted by the analysis shall be within the limits specified by Section 9.5.2.8. If a nonlinear analysis is performed, component deformation demands shall not exceed limiting values as indicated in Section 9.5.7.3.2.

Exception: For structures designed using the index force analysis procedure of Section 9.5.3 or the simplified analysis procedure of Section 9.5.4, drift need not be evaluated.

9.5.2.5.1 Analysis Procedures. The structural analysis required by Section 5.2.5 shall consist of one of the types permitted in Table 9.5.2.5.1, based on the structure's Seismic Design Category, structural system, dynamic properties and regularity, or with the approval of the authority having jurisdiction, an alternative generally accepted procedure shall be permitted to be used.

9.5.2.5.2 Application of Loading. The directions of application of seismic forces used in the design shall be those which will produce the most critical load effects. It shall be permitted to satisfy this requirement using the procedures of Section 9.5.2.5.2.1 for Seismic Design Category A and B, Section 9.5.2.5.2.2 for Seismic Design Category C, and Section 9.5.2.5.2.3 for Seismic Design Categories D, E, and F. All structural components and their connections shall be provided with strengths sufficient to resist the effects of the seismic forces prescribed herein. Loads shall be combined as prescribed in Section 9.5.2.7.

9.5.2.5.2.1 Seismic Design Categories A and B. For structures assigned to Seismic Design Category A and B, the design seismic forces are permitted to be applied separately in each of two orthogonal directions and orthogonal interaction effects may be neglected.

9.5.2.5.2.2 Seismic Design Category C. Loading applied to structures assigned to Seismic Design

TABLE 9.5.2.5.1
PERMITTED ANALYTICAL PROCEDURES

Seismic Design Category	Structural Characteristics	Index Force Analysis Section 9.5.3	Simplified Analysis Section 9.5.4	Equivalent Lateral Force Analysis Section 9.5.5	Modal Response Spectrum Analysis Section 9.5.6	Linear Response History Analysis Section 9.5.7	Nonlinear Response History Analysis Section 9.5.8
A	All structures	P	P	P	P	P	P
B, C	SUG-1 buildings of light-framed construction not exceeding three stories in height	NP	P	P	P	P	P
	Other SUG-1 buildings not exceeding two stories in height	NP	P	P	P	P	P
	All other structures	NP	NP	P	P	P	P
D, E, F	SUG-1 buildings of light-framed construction not exceeding three stories in height	NP	P	P	P	P	
	Other SUG-1 buildings not exceeding two stories in height	NP	P	P	P	P	P
	Regular structures with T< 3.5 T_s and all structures of light-frame construction	NP	NP	P	P	P	P
	Irregular structures with T <3.5 T_s and having only plan irregularities type 2, 3, 4, or 5 of Table 9.5.2.3.2 or vertical irregularities type 4 or 5 of Table 9.5.2.3.3	NP	NP	P	P	P	P
	All other structures	NP	NP	NP	P	P	P

Notes: P — indicates permitted, NP — indicates not permitted

Category C shall, as a minimum, conform to the requirements of Section 9.5.2.5.2.1 for Seismic Design Categories A and B and the requirements of this Section. Structures that have plan structural irregularity Type 5 in Table 9.5.2.3.2 shall be analyzed for seismic forces using a three-dimensional representation and either of the following procedures:

a. The structure shall be analyzed using the equivalent lateral force analysis procedure of Section 9.5.5, the modal response spectrum analysis procedure of Section 9.5.6, or the linear response history analysis procedure of Section 9.5.7, as permitted under Section 9.5.2.5.1, with the loading applied independently in any two orthogonal directions and the most critical load effect due to direction of application of seismic forces on the structure may be assumed to be satisfied if components and their foundations are designed for the following combination of prescribed loads: 100% of the forces for one direction plus 30% of the forces for the perpendicular direction; the combination requiring the maximum component strength shall be used.

b. The structure shall be analyzed using the linear response history analysis procedure of Section 9.5.7 or the nonlinear response history analysis procedure of Section 9.5.8, as permitted by Section 9.5.2.5.1, with orthogonal pairs of ground motion acceleration histories applied simultaneously.

9.5.2.5.2.3 Seismic Design Categories D, E, and F. Structures assigned to Seismic Design Categories D, E, and F shall, as a minimum, conform to the requirements of Section 9.5.2.5.2.2 In addition, any column or wall that forms part of two or more intersecting seismic force-resisting systems and is subjected to axial load due to seismic forces acting along either principal plan axis equaling or exceeding 20% of the axial design strength of the column or wall shall be designed for the most critical load effect due to application of seismic forces in any direction. Either of the procedures of Section 9.5.2.5.2a or b shall be permitted to be used to satisfy this requirement. Two-dimensional analyses shall be permitted for structures with flexible diaphragms.

9.5.2.6 Design and Detailing Requirements. The design and detailing of the components of the seismic force-resisting system shall comply with the requirements of this Section. Foundation design shall conform to the applicable requirements of Section 9.7. The materials and the systems composed of those materials shall conform to the requirements and limitations of Sections 9.8 through 9.12 for the applicable category.

9.5.2.6.1 Seismic Design Category A. The design and detailing of structures assigned to Category A shall comply with the requirements of this Section.

9.5.2.6.1.1 Load Path Connections. All parts of the structure between separation joints shall be interconnected to form a continuous path to the seismic force-resisting system, and the connections shall be capable of transmitting the seismic force (F_p) induced by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure with elements having a design strength capable of transmitting a seismic force of 0.133 times the short period design spectral response acceleration coefficient, S_{DS}, times the weight of the smaller portion or 5% of the portion's weight, whichever is greater. This connection force does not apply to the overall design of the lateral-force-resisting system. Connection design forces need not exceed the maximum forces that the structural system can deliver to the connection.

A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder, or truss to its support. The connection shall have a minimum strength of 5% of the dead plus live load reaction. One means to provide the strength is to use connecting elements such as slabs.

9.5.2.6.1.2 Anchorage of Concrete or Masonry Walls. Concrete and masonry walls shall be anchored to the roof and all floors and members that provide lateral support for the wall or which are supported by the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be capable of resisting the horizontal forces specified in Section 9.5.2.6.1.1 but not less than a minimum strength level, horizontal force of 280 lbs/ linear ft (4.09 kN/m) of wall substituted for E in the load combinations of Section 2.3.2 or 2.4.1.

9.5.2.6.2 Seismic Design Category B. Structures assigned to Seismic Design Category B shall conform to the requirements of Section 9.5.2.6.1 for Seismic Design Category A and the requirements of this Section.

9.5.2.6.2.1 *P*-Delta Effects. *P*-delta effects shall be included where required by Section 9.5.5.7.2.

9.5.2.6.2.2 Openings. Where openings occur in shear walls, diaphragms, or other plate-type elements, reinforcement at the edges of the openings shall be designed to transfer the stresses into the structure. The edge reinforcement shall extend into the body of the wall or diaphragm a distance sufficient to develop the force in the reinforcement. The extension must be sufficient in length to allow dissipation or transfer of the force without exceeding the shear and tension capacity of the diaphragm or the wall.

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

9.5.2.6.2.4 Discontinuities in Vertical System. Structures with a discontinuity in lateral capacity, vertical irregularity Type 5 as defined in Table 9.5.2.3.3, shall not be more than 2 stories or 30 ft (9 m) in height where the "weak" story has a calculated strength of less than 65% of the story above.

Exception: The limit does not apply where the "weak" story is capable of resisting a total seismic force equal to Ω_0 times the design force prescribed in Section 9.5.3.

9.5.2.6.2.5 Nonredundant Systems. The design of a structure shall consider the potentially adverse effect that the failure of a single member, connection, or component of the seismic force-resisting system will have on the stability of the structure; see Section 1.4.

9.5.2.6.2.6 Collector Elements. Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces.

9.5.2.6.2.7 Diaphragms. The deflection in the plane of the diaphragm, as determined by engineering analysis, shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection which will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

Floor and roof diaphragms shall be designed to resist F_p where F_p is the larger of:

1. The portion of the design seismic force at the level of the diaphragm that depends on the diaphragm for transmission to the vertical elements of the seismic force-resisting system, or
2. $F_p = 0.2 S_{DS} I w_p + V_{px}$ **(Eq. 9.5.2.6.2.7)**

   where

   F_p = the seismic force induced by the parts

   I = occupancy importance factor (Table 9.1.4)

   S_{DS} = the short period site design spectral response acceleration coefficient, Section 9.4.1

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

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

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

At diaphragm discontinuities, such as openings and re-entrant corners, the design shall ensure that the dissipation or transfer of edge (chord) forces combined with other forces in the diaphragm is within shear and tension capacity of the diaphragm.

9.5.2.6.2.8 Anchorage of Concrete or Masonry Walls. Exterior and interior bearing walls and their anchorage shall be designed for a force normal to the surface equal to 40% of the short period design spectral response acceleration, S_{DS}, times the occupancy importance factor, I, multiplied by the weight of wall (W_c) associated with the anchor, with a minimum force of 10% of the weight of the wall. Interconnection of wall elements and connections to supporting framing systems shall have sufficient ductility, rotational capacity, or sufficient strength to resist shrinkage, thermal changes, and differential foundation settlement when combined with seismic forces. The connections shall also satisfy Section 9.5.2.6.1.2.

The anchorage of concrete or masonry walls to supporting construction shall provide a direct connection capable of resisting the greater of the force 0.4 $S_{DS} I W_c$ as given above or 400 $S_{DS} I$ lbs/linear ft (5.84 $S_{DS} I$ kN/m) of wall or the force specified in Section 9.5.2.6.1.2. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 ft (1219 mm).

9.5.2.6.2.9 Inverted Pendulum-Type Structures. Supporting columns or piers of inverted pendulum-type structures shall be designed for the bending moment calculated at the base determined using the procedures given in Section 9.5.3 and varying uniformly to a moment at the top equal to one-half the calculated bending moment at the base.

9.5.2.6.2.10 Anchorage of Nonstructural Systems. When required by Section 9.6, all portions or components of the structure shall be anchored for the seismic force, F_p, prescribed therein.

9.5.2.6.2.11 Elements Supporting Discontinuous Walls or Frames. Columns, beams, trusses, or slabs supporting discontinuous walls or frames of structures having plan irregularity Type 4 of Table 9.5.2.3.2 or vertical irregularity Type 4 of

Table 9.5.2.3.3 shall have the design strength to resist the maximum axial force that can develop in accordance with the special seismic loads of Section 9.5.2.7.1.

9.5.2.6.3 Seismic Design Category C. Structures assigned to Category C shall conform to the requirements of Section 9.5.2.6.2 for Category B and to the requirements of this Section.

9.5.2.6.3.1 Collector Elements. Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces. Collector elements, splices, and their connections to resisting elements shall resist the special seismic loads of Section 9.5.2.7.1.

Exception: In structures or portions thereof braced entirely by light-frame shear walls, collector elements, splices, and connections to resisting elements need only be designed to resist forces in accordance with Section 9.5.2.5.4.

The quantity $\Omega_0 E$ in Eq. 9.5.2.7.1-1 need not exceed the maximum force that can be transferred to the collector by the diaphragm and other elements of the lateral force-resisting system.

9.5.2.6.3.2 Anchorage of Concrete or Masonry Walls. Concrete or masonry walls shall be anchored to all floors, roofs, and members that provide out-of-plane lateral support for the wall or that are supported by the wall. The anchorage shall provide a positive direct connection between the wall and floor, roof, or supporting member capable of resisting horizontal forces specified in this Section for structures with flexible diaphragms or with Section 9.6.1.3 (using a_p of 1.0 and R_p of 2.5) for structures with diaphragms that are not flexible.

Anchorage of walls to flexible diaphragms shall have the strength to develop the out-of-plane force given by Eq. 9.5.2.6.3.2:

$$F_p = 0.8 S_{DS} I W_p \qquad \textbf{(Eq. 9.5.2.6.3.2)}$$

where

F_p = the design force in the individual anchors
S_{DS} = the design spectral response acceleration at short periods per Section 9.4.1.2.5
I = the occupancy importance factor per Section 9.1.4
W_p = the weight of the wall tributary to the anchor

Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragms. Added chords may be used to form subdiaphragms to transmit the anchorage forces to the main continuous cross ties. The maximum length-to-width ratio of the structural subdiaphragm shall be $2\frac{1}{2}$ to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components. Connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm.

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

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

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

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

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

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

9.5.2.6.4 Seismic Design Category D. Structures assigned to Category D shall conform to the requirements of Section 9.5.2.6.3 for Category C and to the requirements of this Section.

9.5.2.6.4.1 Collector Elements. In addition to the requirements of Section 9.5.2.6.3.1, collector elements, splices, and their connections to resisting elements shall resist the forces determined in accordance with Section 9.5.2.6.4.4.

9.5.2.6.4.2 Plan or Vertical Irregularities. When the ratio of the strength provided in any story to the strength required is less than two-thirds of that ratio for the story immediately above, the potentially adverse effect shall be analyzed and the strengths shall be adjusted to compensate for this effect.

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

9.5.2.6.4.3 Vertical Seismic Forces. The vertical component of earthquake ground motion shall be considered in the design of horizontal cantilever and horizontal prestressed components. The load combinations used in evaluating such components shall include E as defined by Eqs. 9.5.2.7-1 and 9.5.2.7-2. Horizontal cantilever structural components shall be designed for a minimum net upward force of 0.2 times the dead load in addition to the applicable load combinations of Section 9.5.2.7.

9.5.2.6.4.4 Diaphragms. The deflection in the plane of the diaphragm shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached elements to maintain structural integrity under the individual loading and continue to support the prescribed loads. Floor and roof diaphragms shall be designed to resist design seismic forces determined in accordance with Eq. 9.5.2.6.4.4 as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad \textbf{(Eq. 9.5.2.6.4.4)}$$

where

F_{px} = the diaphragm design force
F_i = the design force applied to Level i
w_i = the weight tributary to Level i
w_{px} = the weight tributary to the diaphragm at Level x

The force determined from Eq. 9.5.2.6.4.4 need not exceed $0.4S_{DS}Iw_{px}$ but shall not be less than $0.2S_{DS}Iw_{px}$. When the diaphragm is required to transfer design seismic force from the vertical resisting elements above the diaphragm to other vertical resisting elements below the diaphragm due to offsets in the placement of the elements or to changes in relative lateral stiffness in the vertical elements, these forces shall be added to those determined from Eq. 9.5.2.6.4.4.

9.5.2.6.5 Seismic Design Categories E and F. Structures assigned to Seismic Design Categories E and F shall conform to the requirements of Section 9.5.2.6.4 for Seismic Design Category D and to the requirements of this Section.

9.5.2.6.5.1 Plan or Vertical Irregularities. Structures having plan irregularity Type 1b of Table 9.5.2.3.1 or vertical irregularities Type 1b or 5 of Table 9.5.2.3.3 shall not be permitted.

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

for load combination 5 in Section 2.3.2 or load combination 4 in Section 2.4.1:

$$E = \rho Q_E + 0.2S_{DS}D \qquad \textbf{(Eq. 9.5.2.7-1)}$$

for load combination 6 in Section 2.3.2 or load combination 3 in 2.4.1:

$$E = \rho Q_E - 0.2S_{DS}D \qquad \textbf{(Eq. 9.5.2.7-2)}$$

where

E = the effect of horizontal and vertical earthquake-induced forces
S_{DS} = the design spectral response acceleration at short periods obtained from Section 9.4.1.2.5
D = the effect of dead load, D
Q_E = the effect of horizontal seismic (earthquake-induced) forces
ρ = the reliability factor

9.5.2.7.1 Special Seismic Load. Where specifically indicated in this Standard, the special seismic load of Eq. 9.5.2.7.1-1 shall be used to compute E for use in load combination 5 in Section 2.3.2 or load combination 3 in 2.4.1 and the special seismic load of Eq. 9.5.2.7.1-2 shall be used to compute E in load combination 7 in Section 2.3.2 or load combination 5 in Section 2.4.1:

$$E = \Omega_o Q_E + 0.2 S_{DS} D \quad \textbf{(Eq. 9.5.2.7.1-1)}$$

$$E = \Omega_o Q_E - 0.2 S_{DS} D \quad \textbf{(Eq. 9.5.2.7.1-2)}$$

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

Where allowable stress design methodologies are used with the special load of this Section applied in load combinations 3 or 5 of Section 2.4.1, allowable stresses are permitted to be determined using an allowable stress increase of 1.2. This increase shall not be combined with increases in allowable stresses or load combination reductions otherwise permitted by this standard or the material reference standard except that combination with the duration of load increases permitted in Ref. 9-12.1 is permitted.

9.5.2.8 Deflection, Drift Limits, and Building Separation. The design story drift (Δ) as determined in Section 9.5.5.7 or 9.5.6.6, shall not exceed the allowable story drift (Δ_a) as obtained from Table 9.5.2.8 for any story. For structures with significant torsional deflections, the maximum drift shall include torsional effects. All portions of the structure shall be designed and constructed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact under total deflection (δ_x) as determined in Section 9.5.3.7.1

9.5.3 Index Force Analysis Procedure for Seismic Design of Buildings. See Section 9.5.2.5.1 for limitations on the use of this procedure. An index force analysis shall consist of the application of static lateral index forces to a linear mathematical model of the *structure*, independently in each of two orthogonal directions. The lateral index forces shall be as given by Eq. 9.5.3-1 and shall be applied simultaneously at each floor level. For purposes of analysis, the *structure* shall be considered to be fixed at the *base*:

$$F_x = 0.01 w_x \quad \textbf{(Eq. 9.5.3-1)}$$

where

F_x = the design lateral force applied at story x
w_x = the portion of the total gravity load of the *structure*, W, located or assigned to Level x
W = the effective seismic weight of the structure, including the total dead load and other loads listed below:

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

TABLE 9.5.2.8
ALLOWABLE STORY DRIFT, Δ_a[a]

Structure	Seismic Use Group		
	I	II	III
Structures, other than masonry shear wall or masonry wall frame structures, four stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$[b]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[c]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
Masonry wall frame structures	$0.013h_{sx}$	$0.013h_{sx}$	$0.010h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.
[b] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 9.5.2.8 is not waived.
[c] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

9.5.4 Simplified Analysis Procedure for Seismic Design of Buildings. See Section 9.5.2.5.1 for limitations on the use of this procedure. For purposes of this analysis procedure, a building is considered to be fixed at the base.

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

$$V = \frac{1.2S_{DS}}{R}W \qquad \textbf{(Eq. 9.5.4.1)}$$

where

S_{DS} = the design elastic response acceleration at short period as determined in accordance with Section 9.4.1.2.5

R = the response modification factor from Table 9.5.2.2

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

9.5.4.2 Vertical Distribution. The forces at each level shall be calculated using the following formula:

$$F_x = \frac{1.2S_{DS}}{R}w_x \qquad \textbf{(Eq. 9.5.4.2)}$$

where

w_x = the portion of the effective seismic weight of the structure, W, at level x

9.5.4.3 Horizontal Distribution. Diaphragms constructed of wood structural panels or untopped steel decking are permitted to be considered as flexible.

9.5.4.4 Design Drift. For the purposes of Section 9.5.2.8, the design story drift, Δ, shall be taken as 1% of the story height unless a more exact analysis is provided.

9.5.5 Equivalent Lateral Force Procedure.

9.5.5.1 General. Section 9.5.5 provides required minimum standards for the equivalent lateral force procedure of seismic analysis of structures. An equivalent lateral force analysis shall consist of the application of equivalent static lateral forces to a linear mathematical model of the *structure*. The directions of application of lateral forces shall be as indicated in Section 9.5.2.5.2. The lateral forces applied in each direction shall be the total seismic base shear given by Section 9.5.5.2 and shall be distributed vertically in accordance with the provisions of Section 9.5.5.3. For purposes of analysis, the structure is considered to be fixed at the base. See Section 9.5.2.5 for limitations on the use of this procedure.

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

$$V = C_sW \qquad \textbf{(Eq. 9.5.5.2-1)}$$

where

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

W = the total dead load and applicable portions of other loads as indicated in Section 9.5.3

9.5.5.2.1 Calculation of Seismic Response Coefficient. When the fundamental period of the structure is computed, the seismic design coefficient (C_s) shall be determined in accordance with the following equation:

$$C_s = \frac{S_{DS}}{R/I} \qquad \textbf{(Eq. 9.5.5.2.1-1)}$$

where

S_{DS} = the design spectral response acceleration in the short period range as determined from Section 9.4.1.2.5

R = the response modification factor in Table 9.5.2.2

I = the occupancy importance factor determined in accordance with Section 9.1.4

A soil-structure interaction reduction shall not be used unless Section 9.5.9 or another generally accepted procedure approved by the authority having jurisdiction is used.

Alternatively, the seismic response coefficient, (C_s), need not be greater than the following equation:

$$C_s = \frac{S_{D1}}{T(R/I)} \qquad \textbf{(Eq. 9.5.5.2.1-2)}$$

but shall not be taken less than

$$C_s = 0.044S_{DS}I \qquad \textbf{(Eq. 9.5.5.2.1-3)}$$

nor for buildings and structures in Seismic Design Categories E and F

$$C_s = \frac{0.5S_1}{R/I} \qquad \textbf{(Eq. 9.5.5.2.1-4)}$$

where I and R are as defined above and

S_{D1} = the design spectral response acceleration at a period of 1.0 sec, in units of g-sec, as determined from Section 9.4.1.2.5

T = the fundamental period of the structure (sec) determined in Section 9.5.5.3

S_1 = the mapped maximum considered earthquake spectral response acceleration determined in accordance with Section 9.4.1

A soil-structure interaction reduction is permitted when determined using Section 9.5.9.

For regular structures 5 stories or less in height and having a period, T, of 0.5 sec or less, the seismic response coefficient, C_s shall be permitted to be calculated using values of 1.5 g and 0.6 g, respectively, for the mapped maximum considered earthquake spectral response accelerations S_S and S_1.

9.5.5.3 Period Determination. The fundamental period of the structure (T) in the direction under consideration shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The fundamental period (T) shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 9.5.5.3 and the approximate fundamental period (T_a) determined from Eq. 9.5.5.3-1. As an alternative to performing an analysis to determine the fundamental period (T), it shall be permitted to use the approximate building period, (T_a), calculated in accordance with Section 9.5.5.3.2, directly.

9.5.5.3.1 Upper Limit on Calculated Period. The fundamental building period (T) determined in a properly substantiated analysis shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 9.5.5.3.1 and the approximate fundamental period (T_a) determined in accordance with Section 9.5.5.3.2.

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

TABLE 9.5.5.3.1
COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD

Design Spectral Response Acceleration at 1 Second, S_{D1}	Coefficient C_u
$\geq$ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
0.1	1.7
$\leq$ 0.05	1.7

$$T_a = C_t h_n^{\ x} \qquad \textbf{(Eq. 9.5.5.3.2-1)}$$

where h_n is the height in ft above the base to the highest level of the structure and the coefficients C_t and x are determined from Table 9.5.5.3.2.

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

$$T_a = 0.1N \qquad \textbf{(Eq. 9.5.5.3.2-1a)}$$

where N = number of stories

The approximate fundamental period, T_a, in seconds for masonry or concrete shear wall structures shall be permitted to be determined from

TABLE 9.5.5.3.2
VALUES OF APPROXIMATE PERIOD PARAMETERS C_t AND x

Structure Type	C_t	x
Moment resisting frame systems of steel in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frames from deflecting when subjected to seismic forces	0.028(0.068)[a]	0.8
Moment resisting frame systems of reinforced concrete in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frame from deflecting when subjected to seismic forces	0.016(0.044)[a]	0.9
Eccentrically braced steel frames	0.03(0.07)[a]	0.75
All other structural systems	0.02(0.055)	0.75

[a] —metric equivalents are shown in parentheses

Eq. 9.5.5.3.2-2 as follows:

$$T_a = \frac{0.0019}{\sqrt{C_W}} h_n \quad \textbf{(Eq. 9.5.5.3.2-2)}$$

where h_n is as defined above and C_w is calculated from Eq. 9.5.5.3.2-3 as follows:

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

(Eq. 9.5.5.3.2-3)

where

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

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

$$F_x = C_{vx} V \quad \textbf{(Eq. 9.5.5.4-1)}$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \quad \textbf{(Eq. 9.5.5.4-2)}$$

where

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

9.5.5.5 Horizontal Shear Distribution and Torsion. The seismic design story shear in any story (V_x) (kip or kN) shall be determined from the following equation:

$$V_x = \sum_{i=x}^{n} F_i \quad \textbf{(Eq. 9.5.5.5)}$$

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

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

9.5.5.5.2 Torsion. Where diaphragms are not flexible, the design shall include the torsional moment (M_t) (kip or kN) resulting from the location of the structure masses plus the accidental torsional moments (M_{ta}) (kip or kN) caused by assumed displacement of the mass each way from its actual location by a distance equal to 5% of the dimension of the structure perpendicular to the direction of the applied forces. Where earthquake forces are applied concurrently in two orthogonal directions, the required 5% displacement of the center of mass need not be applied in both of the orthogonal directions at the same time, but shall be applied in the direction that produces the greater effect.

Structures of Seismic Design Categories C, D, E, and F, where Type 1 torsional irregularity exists as defined in Table 9.5.2.3.2, shall have the effects accounted for by multiplying M_{ta} at each level by a torsional amplification factor (A_x) determined from the following equation:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \quad \textbf{(Eq. 9.5.5.5.2)}$$

where

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

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

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

9.5.5.6 Overturning. The structure shall be designed to resist overturning effects caused by the seismic forces determined in Section 9.5.4.4. At any story, the increment of overturning moment in the story under consideration shall be distributed to the various vertical elements of the lateral force-resisting system in the same proportion as the distribution of the horizontal shears to those elements.

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

$$M_x = \sum_{i=x}^{n} F_i(h_i - h_x) \qquad \textbf{(Eq. 9.5.5.6)}$$

where

F_i = the portion of the seismic base shear (V) induced at Level i
h_i and h_x = the height (in ft or m) from the base to Level i or x

The foundations of structures, except inverted pendulum-type structures, shall be permitted to be designed for 75% of the foundation overturning design moment (M_f) (kip·ft or kN·m) at the foundation–soil interface determined using the equation for the overturning moment at Level x (M_x) (kip·ft or kN·m).

9.5.5.7 Drift Determination and *P*-Delta Effects. Story drifts and, where required, member forces and moments due to P-delta effects shall be determined in accordance with this Section. Determination of story drifts shall be based on the application of the design seismic forces to a mathematical model of the physical structure. The model shall include the stiffness and strength of all elements that are significant to the distribution of forces and deformations in the structure and shall represent the spatial distribution of the mass and stiffness of the structure. In addition, the model shall comply with the following:

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

9.5.5.7.1 Story Drift Determination. The design story drift (Δ) shall be computed as the difference of the deflections at the top and bottom of the story under consideration. Where allowable stress design is used, Δ shall be computed using code-specified earthquake forces without reduction.

Exception: For structures of Seismic Design Categories C, D, E, and F having plan irregularity Types 1a or 1b of Table 9.5.2.3.2, the design story drift, D, shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the story under consideration.

The deflections of Level x at the center of the mass (δ_x) (in. or mm) shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad \textbf{(Eq. 9.5.5.7.1)}$$

where

C_d = the deflection amplification factor in Table 9.5.2.2
δ_{xe} = the deflections determined by an elastic analysis
I = the importance factor determined in accordance with Section 9.1.4

The elastic analysis of the seismic force-resisting system shall be made using the prescribed seismic design forces of Section 9.5.3.4. For the purpose of this Section, the value of the base shear, V, used in Eq. 9.5.3.2 need not be limited by the value obtained from Eq. 9.5.5.2.1-3.

For determining compliance with the story drift limitation of Section 9.5.2.8, the deflections at the center of mass of Level x (δ_x) (in. or mm) shall be calculated as required in this Section. For the purposes of this drift analysis only, the upper-bound limitation specified in Section 9.5.3.3 on the computed fundamental period, T, in seconds, of the building does not apply for computing forces and displacements.

Where applicable, the design story drift (Δ) (in. or mm) shall be increased by the incremental factor relating to the P-delta effects as determined in Section 9.5.5.7.2.

When calculating drift, the redundancy coefficient, ρ, is not used.

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

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad \textbf{(Eq. 9.5.5.7.2-1)}$$

where

P_x = the total vertical design load at and above Level x. (kip or kN); when computing P_x, no individual load factor need exceed 1.0
Δ = the design story drift as defined in Section 9.5.3.7.1 occurring simultaneously with V_x, (in. or mm)

V_x = the seismic shear force acting between Levels x and $x-1$, (kip or kN)
h_{sx} = the story height below Level x, (in. or mm)
C_d = the deflection amplification factor in Table 9.5.2.2

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

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \qquad \textbf{(Eq. 9.5.5.7.2-2)}$$

where β is the ratio of shear demand to shear capacity for the story between Level x and $x-1$. This ratio may be conservatively taken as 1.0.

When the stability coefficient (θ) is greater than 0.10 but less than or equal to θ_{max}, the incremental factor related to P-delta effects (a_d) shall be determined by rational analysis. To obtain the story drift for including the P-delta effect, the design story drift determined in Section 9.5.5.7.1 shall be multiplied by $1.0/(1-\theta)$.

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

When the P-delta effect is included in an automated analysis, Eq. 9.5.5.7.2-2 must still be satisfied, however, the value of θ computed from Eq. 9.5.5.7.2-1 using the results of the P-delta analysis may be divided by $(1+\theta)$ before checking Eq. 9.5.5.7.2-2.

9.5.6 Modal Analysis Procedure.

9.5.6.1 General. Section 9.5.6 provides required standards for the modal analysis procedure of seismic analysis of structures. See Section 9.5.2.5 for requirements for use of this procedure. The symbols used in this method of analysis have the same meaning as those for similar terms used in Section 9.5.3, with the subscript m denoting quantities in the m^{th} mode.

9.5.6.2 Modeling. A mathematical model of the structure shall be constructed that represents the spatial distribution of mass and stiffness throughout the structure.

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

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

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

9.5.6.4 Periods. The required periods, mode shapes, and participation factors of the structure in the direction under consideration shall be calculated by established methods of structural analysis for the fixed-base condition using the masses and elastic stiffnesses of the seismic force-resisting system.

9.5.6.5 Modal Base Shear. The portion of the base shear contributed by the m^{th} mode (V_m) shall be determined from the following equations:

$$V_m = C_{sm} W_m \qquad \textbf{(Eq. 9.5.6.5-1)}$$

$$W_m = \frac{\left(\sum_{i=1}^{n} w_i \phi_{im}\right)^2}{\sum_{i=1}^{n} w_i \phi_{im}^2} \qquad \textbf{(Eq. 9.5.6.5-2)}$$

where

C_{sm} = the modal seismic design coefficient determined below
W_m = the effective modal gravity load
w_i = the portion of the total gravity load of the structure at Level i
ϕ_{im} = the displacement amplitude at the i^{th} level of the structure when vibrating in its m^{th} mode

The modal seismic design coefficient (C_{sm}) shall be determined in accordance with the following equation:

$$C_{sm} = \frac{S_{am}}{R/I} \qquad \textbf{(Eq. 9.5.6.5-3)}$$

where

S_{am} = the design spectral response acceleration at period T_m determined from either the general design response spectrum of Section 9.4.1.2.6 or a site-specific response spectrum per Section 9.4.1.3

R = the response modification factor determined from Table 9.5.2.2

I = the occupancy importance factor determined in accordance with Section 9.1.4

T_m = the modal period of vibration (in seconds) of the m^{th} mode of the structure

Exception: When the general design response spectrum of Section 9.4.1.2.6 is used for structures where any modal period of vibration (T_m) exceeds 4.0 sec, the modal seismic design coefficient (C_{sm}) for that mode shall be determined by the following equation:

$$C_{sm} = \frac{4S_{D1}}{(R/I)T_m^2} \qquad \textbf{(Eq. 9.5.6.5-4)}$$

The reduction due to soil-structure interaction as determined in Section 9.5.5.3 is permitted to be used.

9.5.6.6 Modal Forces, Deflections, and Drifts. The modal force (F_{xm}) at each level shall be determined by the following equations:

$$F_{xm} = C_{vxm} V_m \qquad \textbf{(Eq. 9.5.6.6-1)}$$

and

$$C_{vxm} = \frac{w_x \phi_{xm}}{\sum_{i=1}^{n} w_i \phi_{im}} \qquad \textbf{(Eq. 9.5.6.6-2)}$$

where

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

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

w_i and w_x = the portion of the total gravity load of the structure (W) located or assigned to Level i or x

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

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

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

$$\delta_{xm} = \frac{C_d \delta_{xem}}{I} \qquad \textbf{(Eq. 9.5.6.6-3)}$$

and

$$\delta_{xem} = \left(\frac{g}{4\pi^2}\right)\left(\frac{T_m^2 F_{xm}}{w_x}\right) \qquad \textbf{(Eq. 9.5.6.6-4)}$$

where

C_d = the deflection amplification factor determined from Table 9.5.2.2

δ_{xem} = the deflection of Level x in the m^{th} mode at the center of the mass at Level x determined by an elastic analysis

g = the acceleration due to gravity (ft^2/sec)

I = the occupancy importance factor determined in accordance with Section 9.1.4

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

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

w_x = the portion of the total gravity load of the structure (W) located or assigned to Level x

The modal drift in a story (Δ_m) shall be computed as the difference of the deflections (δ_{xm}) at the top and bottom of the story under consideration.

9.5.6.7 Modal Story Shears and Moments. The story shears, story overturning moments, and the shear forces and overturning moments in vertical elements of the structural system at each level due to the seismic forces determined from the appropriate equation in Section 9.5.6.6 shall be computed for each mode by linear static methods.

9.5.6.8 Design Values. The design value for the modal base shear (V_t), each of the story shear, moment and drift quantities, and the deflection at each level shall be determined by combining their modal values as obtained from Sections 9.5.6.6 and 9.5.6.7. The combination shall be carried out by taking the square root of the sum of the squares of each of the modal values or where closely spaced periods in the translational and torsional modes result in significant cross-correlation of the modes, the complete quadratic combination (CQC) method, in accordance with ASCE-4, shall be used.

A base shear (V) shall be calculated using the equivalent lateral force procedure in Section 9.5.5. For the purpose of this calculation, a fundamental period of the structure (T), in seconds, shall not exceed the coefficient for upper limit on the calculated period (C_u) times the approximate fundamental period of the structure (T_a). Where the design value for the modal base shear (V_t) is less than 85% of the calculated base shear (V) using the equivalent lateral force procedure, the design story shears, moments, drifts, and floor deflections shall be multiplied by the following modification factor:

$$0.85\frac{V}{V_t} \qquad \textbf{(Eq. 9.5.6.8)}$$

where

V = the equivalent lateral force procedure base shear, calculated in accordance with this section and Section 9.5.5, and

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

9.5.6.9 Horizontal Shear Distribution. The distribution of horizontal shear shall be in accordance with the requirements of Section 9.5.5.5 except that amplification of torsion per Section 9.5.5.5.2 is not required for that portion of the torsion, A_x, included in the dynamic analysis model.

9.5.6.10 Foundation Overturning. The foundation overturning moment at the foundation-soil interface may be reduced by 10%.

9.5.6.11 P-Delta Effects. The P-delta effects shall be determined in accordance with Section 9.5.5.7. The story drifts and base shear used to determine the story shears shall be determined in accordance with Section 9.5.5.7.1.

9.5.7 Linear Response History Analysis Procedure. A linear response history analysis shall consist of an analysis of a linear mathematical model of the *structure* to determine its response, through methods of numerical integration, to suites of ground motion acceleration histories compatible with the design response spectrum for the site. The analysis shall be performed in accordance with the provisions of this Section. For purposes of analysis, the structure shall be permitted to be considered to be fixed at the *base*, or alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness of foundations. See Section 9.5.2.1 for limitations on the use of this procedure.

9.5.7.1 Modeling. Mathematical models shall conform to the requirements of Section 9.5.6.1.

9.5.7.2 Ground Motion. A suite of not less than three appropriate ground motions shall be used in the analysis. Ground motion shall conform to the requirements of this Section.

9.5.7.2.1 Two-Dimensional Analysis. When two-dimensional analyses are performed, each ground motion shall consist of a horizontal acceleration history, selected from an actual recorded event. Appropriate acceleration histories shall be obtained from records of events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the *maximum considered earthquake*. Where the required number of appropriate recorded ground motion records are not available, appropriate simulated ground motion records shall be used to make up the total number required. The ground motions shall be scaled such that the average value of the 5% damped response spectra for the suite of motions is not less than the design response spectrum for the site, determined in accordance with Section 9.4.1.3 for periods ranging from $0.2T$ to $1.5T$ sec where T is the natural period of the *structure* in the fundamental mode for the direction of response being analyzed.

9.5.7.2.2 Three-Dimensional Analysis. When three-dimensional analyses are performed, ground motions shall consist of pairs of appropriate horizontal ground motion acceleration components that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the *maximum considered earthquake*. Where the required number of recorded ground motion pairs are not available, appropriate simulated ground motion pairs shall be used to make up the total number required. For each pair of horizontal ground motion components, the square root of the sum of the squares (SRSS) of the 5% damped response spectrum of the scaled horizontal components shall be constructed. Each pair of motions shall be scaled such that the average value of the SRSS spectra from all horizontal component pairs is not less than 1.3 times the 5% damped design response spectrum determined in accordance with Section 9.4.1.3 for periods ranging from $0.2T$ to $1.5T$ seconds, where T is the natural period of the fundamental mode of the *structure*.

9.5.7.3 Response Parameters. For each ground motion analyzed, the individual response parameters shall be scaled by the quantity I/R, where I is the occupancy importance factor determined in accordance with Section 1.4 and R is the Response Modification Coefficient selected in accordance with Section 9.5.2.2. The maximum value of the base shear, V_i, member forces, Q_{Ei}, and interstory drifts, d_i at each story, scaled as indicated above shall be determined. When the maximum scaled base shear predicted by the analysis, V_i, is less than given by Eq. 9.5.5.1.2-3, or in Seismic Design Categories E and F, Eq. 9.5.5.1.2-4, the scaled member forces, Q_{ei}, shall be additionally scaled by the factor:

$$\frac{V}{V_i} \qquad \textbf{(Eq. 9.5.7.3)}$$

where V is the minimum base shear determined in accordance with Eq. 9.5.5.2.1-3 or for structures in Seismic Design Category E or F, Eq. 9.5.5.2.1-4.

If at least seven ground motions are analyzed, the design member forces, Q_E, used in the load combinations of Section 9.5.2.7, and the design interstory drift, D, used in the evaluation of drift in accordance with Section 5.2.8 shall be permitted to be taken respectively as the average of the scaled Q_{Ei} and D_i values determined from the analyses and scaled as indicated above. If less than seven ground motions are analyzed, the design member forces, Q_E, and the design interstory drift, D, shall be taken as the maximum value of the scaled Q_{Ei} and D_i values determined from the analyses.

Where these provisions require the consideration of the special load combinations of Section 9.5.2.7, the value of $\Omega_0 Q_E$ need not be taken larger than the maximum of the unscaled value, Q_{Ei}, obtained from the suite of analyses.

9.5.8 Nonlinear Response History Analysis. A nonlinear response history analysis shall consist of an analysis of a mathematical model of the *structure* that directly accounts for the nonlinear hysteretic behavior of the structure's components to determine its response through methods of numerical integration to suites of ground motion acceleration histories compatible with the design response spectrum for the site. The analysis shall be performed in accordance with this Section. See Section 9.5.2.1 for limitations on the use of this procedure.

9.5.8.1 Modeling. A mathematical model of the structure shall be constructed that represents the spatial distribution of mass throughout the structure. The hysteretic behavior of elements shall be modeled consistent with suitable laboratory test data and shall account for all significant yielding, strength degradation, stiffness degradation, and hysteretic pinching indicated by such test data. Strength of elements shall be based on expected values considering material overstrength, strain hardening, and also hysteretic strength degradation. Linear properties, consistent with the provisions of Section 9.5.6.1, shall be permitted to be used for those elements demonstrated by the analysis to remain within their linear range of response. The structure shall be assumed to have a fixed base, or alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness and load-carrying characteristics of the foundations consistent with site-specific soils data and rational principles of engineering mechanics.

For regular structures with independent orthogonal seismic force-resisting systems, independent two-dimensional models shall be permitted to be constructed to represent each system. For structures having plan irregularities Types 1a, 1b, 4 or 5 of Table 9.5.2.3.2, or structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis at each level of the structure shall be used. Where the diaphragms are not rigid compared to the vertical elements of the lateral-force-resisting system, the model should include representation of the diaphragm's flexibility and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response.

9.5.8.2 Ground Motion and Other Loading. Ground motion shall conform to the provisions of Section 9.5.7.2. The structure shall be analyzed for the effects of these ground motions simultaneously with the effects of dead load in combination with not less than 25% of the required live loads.

9.5.8.3 Response Parameters. For each ground motion analyzed, individual response parameters consisting of the maximum value of the individual member forces, Q_{Ei}, member inelastic deformations, D_i and interstory drifts, D_i at each story shall be determined.

If at least seven ground motions are analyzed, the design values of member forces, Q_E, member inelastic deformations, D and interstory drift, D shall be permitted to be taken respectively as the average of the scaled Q_{Ei}, g_i, and D_i values determined from the analyses. If less than seven ground motions are analyzed, the design member forces, Q_E, design member inelastic deformations, g and the design interstory drift, D, shall be taken as the maximum value of the scaled Q_{Ei}, g_i, and D_i values determined from the analyses.

9.5.8.3.1 Member Strength. The adequacy of members to resist the combination of load effects of Section 9.5.2.7 need not be evaluated.

Exception: Where this Standard requires the consideration of the special seismic loads of Section 9.5.2.7.1. In such evaluations, the maximum value of Q_{Ei} obtained from the suite of analyses shall be taken in place of the quantity $\Omega_0 Q_E$.

9.5.8.3.2 Member Deformation. The adequacy of individual members and their connections to withstand the estimated design deformation values, g_i, as predicted by the analyses shall be evaluated based on laboratory test data for similar components. The effects of gravity and other loads on member deformation capacity shall be considered in these evaluations. Member deformation shall not exceed two-thirds of a value that results in loss of ability to carry gravity loads or that results in deterioration of member strength to less than the 67% of the peak value.

9.5.8.3.3 Interstory Drift. The design interstory drift obtained from the analyses shall not exceed 125% of the drift limit specified in Section 9.5.2.8.

9.5.8.4 Design Review. A design review of the seismic force-resisting system and the structural analysis shall be performed by an independent team of *registered design professionals* in the appropriate disciplines and others experienced in seismic analysis methods and the theory and application of nonlinear seismic analysis and structural behavior under extreme cyclic loads. The design review shall include, but not be limited to, the following:

1. Review of any site-specific seismic criteria employed in the analysis including the development of site-specific spectra and ground motion time histories
2. Review of acceptance criteria used to demonstrate the adequacy of structural elements and systems to withstand the calculated force and deformation demands, together with that laboratory and other data used to substantiate these criteria.
3. Review of the preliminary design including the selection of structural system and the configuration of structural elements.
4. Review of the final design of the entire structural system and all supporting analyses.

9.5.9 Soil-Structure Interaction.

9.5.9.1 General. If the option to incorporate the effects of soil-structure interaction is exercised, the requirements of this Section shall be used in the determination of the design earthquake forces and the corresponding displacements of the structure. The use of these provisions will decrease the design values of the base shear, lateral forces, and overturning moments but may increase the computed values of the lateral displacements and the secondary forces associated with the P-delta effects.

The provisions for use with the equivalent lateral force procedure are given in Section 9.5.9.2. and those for use with the modal analysis procedure are given in Section 9.5.9.3.

9.5.9.2 Equivalent Lateral Force Procedure. The following requirements are supplementary to those presented in Section 9.5.5.

9.5.9.2.1 Base Shear. To account for the effects of soil-structure interaction, the base shear (V) determined from shall be reduced to:

$$\tilde{V} = V - \Delta V \quad \textbf{(Eq. 9.5.9.2.1-1)}$$

The reduction (ΔV) shall be computed as follows and shall not exceed $0.3V$:

$$V = \left[C_s - \tilde{C}_s\left(\frac{0.05}{\tilde{B}}\right)^{0.4}\right]\overline{W} \leq 0.3V$$

(Eq. 9.5.9.2.1-2)

where

C_s = the seismic design coefficient computed from Eqs. 9.5.5.2.1-1 and 9.5.5.2.1-2 using the fundamental natural period of the fixed-base structure (T or T_a) as specified in Section 9.5.3.3

$\tilde{C}_s$ = the value of C_s computed from Eqs. 9.5.5.2.1-1 and 9.5.5.2.1-2 using the fundamental natural period of the flexibly supported structure (T) defined in Section 9.5.5.2.1.1

$\tilde{B}$ = the fraction of critical damping for the structure-foundation system determined in Section 9.5.5.2.1.2, and

$\overline{W}$ = the effective gravity load of the structure, which shall be taken as 0.7W, except that for structures where the gravity load is concentrated at a single level, it shall be taken equal to W

9.5.9.2.1.1 Effective Building Period. The effective period (T) shall be determined as follows:

$$\tilde{T} = T\sqrt{1 + \frac{\overline{k}}{K_y}\left(1 + \frac{K_y\overline{h}^2}{K_\theta}\right)}$$

(Eq. 9.5.9.2.1.1-1)

where

T = the fundamental period of the structure as determined in Section 9.5.5.3

k = the stiffness of the structure when fixed at the base, defined by the following:

$$\overline{k} = 4\pi^2\left(\frac{\overline{W}}{gT^2}\right)$$

(Eq. 9.5.9.2.1.1-2)

$\overline{h}$ = the effective height of the structure which shall be taken as 0.7 times the total height (h_n) except that for structure where the gravity load is effectively concentrated at a single level, it shall be taken as the height to that level

K_y = the lateral stiffness of the foundation defined as the horizontal force at the level of the foundation necessary to produce a unit deflection at that level, the force and the deflection being measured in the direction in which the structure is analyzed

K_θ = the rocking stiffness of the foundation defined as the moment necessary to produce a unit average rotation of the foundation, the moment and rotation being measured in the direction in which the structure is analyzed, and

g = the acceleration of gravity

The foundation stiffnesses (K_y and K_θ) shall be computed by established principles of foundation mechanics using soil properties that are compatible with the soil strain levels associated with the design earthquake motion. The average shear modulus (G) for the soils beneath the foundation at large strain levels and the associated shear wave velocity (v_s) needed in these computations shall be determined from Table 9.5.9.2.1.1a where

v_{so} = the average shear wave velocity for the soils beneath the foundation at small strain levels (10^{-3}% or less)

$G_o = \gamma v_{so}^2/g$ = the average shear modulus for the soils beneath the foundation at small strain levels

γ = the average unit weight of the soils

Alternatively, for structures supported on mat foundations that rest at or near the ground surface or are embedded in such a way that the side wall contact with the soil are not considered to remain effective during the design ground motion, the effective period of the structure is permitted to be determined from:

$$\tilde{T} = T\sqrt{1 + \frac{25\alpha r_a \bar{h}}{v_s^2 T^2}\left(1 + \frac{1.12 r_a \bar{h}^2}{\alpha_\theta r_m^3}\right)} \quad \textbf{(Eq. 9.5.9.2.1.1-3)}$$

TABLE 9.5.9.2.1.1a
VALUES OF G/G_o AND v_s/v_{so}

	Spectral Response Acceleration, S_{D1}			
	≤0.10	≤0.15	≤0.20	≥0.30
Value of G/G_o	0.81	0.64	0.49	0.42
Value of v_s/v_{so}	0.9	0.8	0.7	0.65

TABLE 9.5.9.2.1.1b
VALUES OF α_θ

r_m/v_sT	α_θ
<0.05	1.0
0.15	0.85
0.35	0.7
0.5	0.6

where

α = the relative weight density of the structure and the soil defined by

$$\alpha = \frac{\bar{W}}{\gamma A_o \bar{h}} \quad \textbf{(Eq. 9.5.9.2.1.1-4)}$$

r_a and r_m = characteristic foundation lengths defined by

$$r_a = \sqrt{\frac{A_o}{\pi}} \quad \textbf{(Eq. 9.5.9.2.1.1-5)}$$

and

$$r_m = \sqrt[4]{\frac{4I_o}{\pi}} \quad \textbf{(Eq. 9.5.9.2.1.1-6)}$$

where

A_o = the area of the load-carrying foundation

I_o = the static moment of inertia of the load-carrying foundation about a horizontal centroidal axis normal to the direction in which the structure is analyzed

α_θ = dynamic foundation stiffness modifier for rocking as determined from Table 9.5.9.2.1.1b

where

r_m = characteristic foundation length as determined by Eq. 9.5.9.2.1.1-6

v_s = shear wave velocity

T = fundamental period as determined in Section 9.5.5.3

9.5.9.2.1.2 Effective Damping. The effective damping factor for the structure-foundation system $\tilde{\beta}$ shall be computed as follows:

$$\tilde{\beta} = \beta_o + \frac{0.05}{(\tilde{T}/T)^3} \quad \textbf{(Eq. 9.5.9.2.1.2-1)}$$

where

β_o = the foundation damping factor as specified in Figure 9.5.9.2.1.2

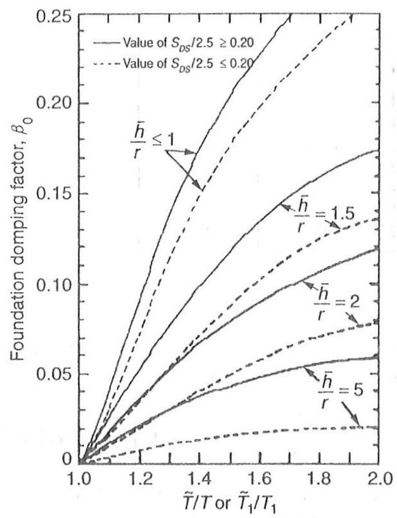


FIGURE 9.5.9.2.1.2
FOUNDATION DAMPING FACTOR

The values of β_o corresponding to $A_v = 0.15$ in Figure 9.5.9.2.1.2 shall be determined by averaging the results obtained from the solid lines and the dashed lines.

The quantity r in Figure 9.5.9.2.1.2 is a characteristic foundation length that shall be determined as follows:

For $\bar{h}/L_o \leq 0.5$,

$$r = r_a = \sqrt{\frac{A_o}{\pi}} \qquad \textbf{(Eq. 9.5.9.2.1.2-2)}$$

For $\bar{h}/L_o \geq 1$,

$$r = r_m = \sqrt[4]{\frac{4I_o}{\pi}} \qquad \textbf{(Eq. 9.5.9.2.1.2-3)}$$

where

L_o = the overall length of the side of the foundation in the direction being analyzed

A_o = the area of the load-carrying foundation, and

I_o = the static moment of inertia of the load-carrying foundation about a horizontal centroidal axis normal to the direction in which the structure is analyzed

For intermediate values of $\bar{h}/L_o$, the value of r shall be determined by linear interpolation.

Exception: For structures supported on point bearing piles and in all other cases where the foundation soil consists of a soft stratum of reasonably uniform properties underlain by a much stiffer, rock-like deposit with an abrupt increase in stiffness, the factor β_o in Eq. 9.5.9.2.1.2-1 shall be replaced by β_o' if $4D_s/v_sT < 1$ where D_s is the total depth of the stratum. β_o' shall be determined as follows:

$$\beta_o' = \left(\frac{4D_s}{V_s\tilde{T}}\right)^2 \beta_o \qquad \textbf{(Eq. 9.5.9.2.1.2-4)}$$

The value of $\tilde{\beta}$ computed from Eq. 9.5.9.2.1.2-1, both with or without the adjustment represented by Eq. 9.5.9.2.1.2-4, shall in no case be taken as less than $\tilde{\beta} = 0.05$ or greater than $\tilde{\beta} = 0.20$.

9.5.9.2.2 Vertical Distribution of Seismic Forces. The distribution over the height of the structure of the reduced total seismic force ($\tilde{V}$) shall be considered to be the same as for the structure without interaction.

9.5.9.2.3 Other Effects. The modified story shears, overturning moments, and torsional effects about a vertical axis shall be determined as for structures without interaction using the reduced lateral forces.

The modified deflections ($\tilde{\delta}_x$) shall be determined as follows:

$$\tilde{\delta}_x = \frac{\tilde{V}}{V}\left[\frac{M_o h_x}{K_\theta} + \delta_x\right] \qquad \textbf{(Eq. 9.5.9.2.3-1)}$$

where

M_o = the overturning moment at the base determined in accordance with Section 9.5.3.6 using the unmodified seismic forces and not including the reduction permitted in the design of the foundation

h_x = the height above the base to the level under consideration

δ_x = the deflections of the fixed-base structure as determined in Section 9.5.3.7.1 using the unmodified seismic forces

The modified story drifts and P-delta effects shall be evaluated in accordance with the provisions of Section 9.5.3.7 using the modified story shears and deflections determined in this Section.

9.5.9.3 Modal Analysis Procedure. The following provisions are supplementary to those presented in Section 9.5.6.

9.5.9.3.1 Modal Base Shears. To account for the effects of soil-structure interaction, the base shear corresponding to the fundamental mode of vibration (V_1) shall be reduced to:

$$\tilde{V}_1 = V_1 - \Delta V_1 \qquad \textbf{(Eq. 9.5.9.3.1-1)}$$

The reduction (ΔV_1) shall be computed in accordance with Eq. 9.5.9.2.1-2 with $\overline{W}$ taken as equal to the gravity load $\overline{W}_1$ defined by Eq. 9.5.6.5-2, C_s computed from Eq. 9.5.6.5-3 using the fundamental period of the fixed-base structure (T_1), and $\tilde{C}_s$ computed from Eq. 9.5.6.5-3 using the fundamental period of the elastically supported structure ($\tilde{T}_1$).

The period $\tilde{T}_1$ shall be determined from Eq. 9.5.9.2.1.1-1, or from Eq. 9.5.9.2.1.1-3 when applicable, taking $T = \tilde{T}_1$, evaluating $\overline{k}$ from Eq. 9.5.9.2.1.1-2 with $\overline{W} = \overline{W}_1$, and computing $\overline{h}$ as follows:

$$\overline{h} = \frac{\sum_{i=1}^{n} w_i \varphi_{i1} h_i}{\sum_{i=1}^{n} w_i \varphi_{i1}} \qquad \textbf{(Eq. 9.5.9.3.1-2)}$$

The above designated values of $\overline{W}$, $\overline{h}$, T, and $\tilde{T}$ also shall be used to evaluate the factor α from Eq. 9.5.9.2.1.1-4 and factor β_o from Figure 9.5.9.2.1.2. No reduction shall be made in the shear components contributed by the higher modes of vibration. The reduced base shear ($\tilde{V}_1$) shall in no case be taken less than $0.7V_1$.

9.5.9.3.2 Other Modal Effects. The modified modal seismic forces, story shears, and overturning moments shall be determined as for structures without interaction using the modified base shear ($\tilde{V}_1$) instead of V_1. The modified modal deflections ($\tilde{\delta}$) shall be determined as follows:

$$\tilde{\delta}_{xl} = \frac{\tilde{V}_1}{V_1}\left[\frac{M_{01}h_x}{K_\theta} + \delta_{x1}\right] \qquad \textbf{(Eq. 9.5.9.3.2-1)}$$

and

$$\tilde{\delta}_{xm} = \delta_{xm} \qquad \textbf{(Eq. 9.5.9.3.2-2)}$$

for $m = 2, 3, \ldots$

where

M_{o1} = the overturning base moment for the fundamental mode of the fixed-base structure, as determined in Section 9.5.6.7 using the unmodified modal base shear V_1

δ_{xm} = the modal deflections at Level x of the fixed-base structure as determined in Section 9.5.6.6 using the unmodified modal shears, V_m

The modified modal drift in a story ($\tilde{\Delta}_m$) shall be computed as the difference of the deflections ($\tilde{\delta}_{xm}$) at the top and bottom of the story under consideration.

9.5.9.3.3 Design Values. The design values of the modified shears, moments, deflections, and story drifts shall be determined as for structures without interaction by taking the square root of the sum of the squares of the respective modal contributions. In the design of the foundation, it shall be permitted to reduce the overturning moment at the foundation-soil interface determined in this manner by 10% as for structures without interaction.

The effects of torsion about a vertical axis shall be evaluated in accordance with the provisions of Section 9.5.6.5 and the P-delta effects shall be evaluated in accordance with the provisions of Section 9.5.6.7.2 using the story shears and drifts determined in Section 9.5.9.3.2.